METRO VANCOUVER

REGIONAL WATER SUPPLY SYSTEM LIFELINE STUDY: SEISMIC VULNERABILITY ASSESSMENT

FEBRUARY 10, 2022

FINAL

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EXECUTIVE SUMMARY

Metro Vancouver retained WSP Canada Inc. (WSP) to conduct a regional-scale seismic vulnerability study of their water transmission network. The assessment included 26 reservoirs (at 21 sites), 19 pump stations at (17 sites) and approximately 500 km of transmission mains. The main objective of the study is to identify the geographical areas, pipe segments and facilities that are potentially vulnerable to damage from strong seismic shaking. Similar reginal-scale study was undertaken in 1993 by Kennedy/Jenks Consultants Engineers and Scientists and EQE Engineering (EQE, 1993). An update to this study was required to account for the changes to the design seismic hazard levels, the seismic performance expectations, the seismic retrofits/upgrades undertaken since the initial study, the improved understanding from past earthquakes and the changes to the design methods. This study intends to assist Metro Vancouver to identify the supply points in its transmission network that are most likely to be resilient to a major seismic event, which can be used to provide a basic level of service to member jurisdictions after a large seismic event.

Seismic Hazard Maps (Section 3)

The seismic assessment was based on the sixth-general seismic hazard models that will be included in the 2020 National Building Code of Canada (NBC). To aid the seismic assessment, the following two sets of seismic hazard maps were developed:

- *Liquefaction hazard maps:* The areas with high, medium and low likelihood of liquefaction and seismic-induced landslide are shown in maps included in Appendix B. Certain watermains may also be impacted by landslides due to their proximity to steep terrains. Site-specific analyses were undertaken at some of these locations to estimate the extent and magnitude of ground displacement.
- *Ground motion amplification maps:* Additional maps were developed to illustrate the potential amplification of earthquake motions as they propagate through the overlying soil layers. The seismic Site Class was used as the indicator of ground motion amplification. Along with the peak ground acceleration (PGA), this was used to estimate the peak ground velocity (PGV) for a given site. PGV is considered a better indicator of the pipe damage since it correlates well with the transient ground strains.

Watermain Vulnerability (Section 4)

The permanent ground displacements (PGD) and transient ground displacements (TGD) are key factors that impact the seismic vulnerability of buried watermains. Transmission mains exposed to such hazards are identified and evaluated by overlaying the pipes on the seismic hazard maps. The pipe sections are divided into smaller segments to account for the variations in seismic hazard (liquefaction potential, ground motion amplification and magnitude and direction of ground displacement relative to the pipe) and pipe properties (e.g., diameter, wall thickness).

The damages caused by TGD was estimated using fragility relationships included in ALA (2001). PGD induced damages are the most dominant form of damage and those were estimated using a framework that included the site-specific studies and pipe-soil interaction analysis. Metro Vancouver's transmission network mainly consists of welded steel pipes with lap welded joints. The empirical fragility relationships are largely based on the performance of small diameter (less than 300 mm) brittle pipes. Thus, the existing fragility relationships for large diameter welded steel pipes are not reliable and do not account for key factors such as the direction of ground movement, pipe characteristics and soil parameters. For this reason, the relative vulnerability of larger diameter lap welded steel pipes was estimated using a novel approach that included pipe-soil interaction analysis. Also, the study has identified 71 water crossings, and they were categorised into four groups based on the vulnerability and consequences. The vulnerability of these transmission mains was evaluated separately based on the available site-specific studies and our judgement. The number of failures estimated for each transmission main are given in Appendix C of this report and key observations are summarized below:

Summary of Estimated Damage for Watermains

	Total	Failure Rate
Total Number of Failures	267	0.54 failures/km
TGD Induced failures	18	0.04 failures/km
PGD induced failures (all)	249	1.98 failures/km (within PGD areas)
PGD induced failures (water crossings)	97	
Total number of breaks	129	
Total number of leaks	138	

Approximately 75 percent of the failures can be attributed PGD induced failures. Also, approximately one-half of PGD failures can be attributed to failures occurring near water crossings.

A total of 129 breaks have been estimated, which is almost entirely (approximately 96 percent) attributed to PGD. Additionally, approximately 46 percent would occur near/at water crossings. It is possible that number of breaks have been overestimated for steel watermains due to the assumed split between breaks and leaks. There are no reliable statistical data to improve this value and it was considered conservative to overestimate the breaks instead of assigning them as leaks.

A total of 138 leaks have been estimated, and approximately 14 percent is attributed to TGD. The remaining is expected to occur due to PGD. There is considerable uncertainty involved in fragility relationships used to estimate the TGD induced damage rates for larger diameter pipes. The estimated damage rate may require further updates if more refined fragility relationships are developed in the future for those pipes.

Water crossings have been divided into four categories to better represent the site conditions (length/depth of the crossing) and consequences of failures. Approximately three to four breaks are expected in Category 1 water crossings that include major rivers such as Fraser River (North and South Arms), Pitt River, False Creek and Burrard Inlet. We have also included Deas Slough, Annieville and Annacis Channels into this category considering the conditions of these crossings. Several site-specific assessments have been already conducted for these crossings. Two failures have been assigned to Category 2 crossings that includes shallow rivers, stream and creeks (e.g., Coquitlam, Seymour and Brunette rivers). No damages were assumed for Category 3 crossings that includes shallow creeks/sloughs, where the pipe is located below the bottom of the creek/slough and it is unlikely to be impacted by the shallow ground failures that may occur at the banks. Category 4 includes bridge crossings, for which two breaks in the watermain were assumed if the bridge is in liquefiable soil, while two leaks were considered if the bridge is not impacted by soil liquefaction.

The estimated failures are significantly higher than the 1993 EQE study, but generally consistent with past earthquakes such as 1995 Kobe and 2016 Kumamoto earthquakes where seismic shaking intensities and extent of liquefaction are somewhat similar to those expected in a 1 in 2475 year return period earthquake occurring in the Greater Vancouver region.

Excluding the water crossings and pipes less than 2 km long, the following transmission mains were assigned a high vulnerability rating as the estimated average breaks/km exceeds 0.12 (i.e., Serviceability Index less than 20 percent).

Watermains with High Seismic Vulnerabilities

Watermain	**Year of Construction**	**Pipe Material Type** (a)	**Total Number of Breaks**	**Total Number of Leaks**	**Breaks per Kilometer**
Annacis Island Main, No. 2	1961	ST/SP	s.15(1)(l) [redacted]		
Annacis Island Main, No. 3	1973	ST/SP			
Barnston Island Main	1990/1998	ST/SP			
Capilano Main No. 4	1936	ST/SP and C			
Capilano Main No. 5 (Marine Drive to North Shaft and Beach Ave)	1969/1975	ST/SP			

Watermain	Year of Construction	Pipe Material Type (a)	Total Number of Breaks	Total Number of Leaks	Breaks per Kilometer
Capilano Main No. 7	1957	ST/SP	s.15(1)(l) [redacted]		
Coquitlam Main No. 2	1956/1959	ST/SP			
Coquitlam Main No. 3	1987	C, ST/SP			
Haney Main No. 2	1966	ST/SP			
Haney Main No. 3	1990/1994	ST/SP			
Lulu Island Delta Main	1962	ST/SP			
Port Moody Main No. 2	1976	ST/SP			
Queensborough Main No. 1	1949-1952	ST/SP			
Queensborough Main No. 2	2018	DI			
River Road – East Main	1980	ST/SP			
River Road Main – West Main	1978	ST/SP			
Seymour Main No. 2	1948	ST/SP			
Seymour Main No. 3	1925/1948	ST/R			
South Delta Main No. 1 (existing)	1964	ST/SP, AC, PVC			
South Delta Main No. 1 (Replaced)	2017	ST/SP			
South Delta Main No. 2	1992/1996	ST/SP			
Tilbury Main	1979	ST/SP			

(a) ST/SP = lap welded steel; ST/R = rivetted steel, C = Concrete pipes DI = Ductile Iron

Facilities (Section 5)

For all facilities, the seismic demand from the 2020 NBC (for a post-disaster structure) is likely to exceed the demand considered for the original design or seismic retrofitting. If the structure was retrofitted or designed based on the 2005 and 2010 NBC, this increase is marginal to moderate (about 10 to 20 percent). Considering the built-in factor of safety against post-disaster structures (i.e., Importance Factor of 1.5), the expected damage is likely to range from minor to moderate. For structures constructed or upgraded as per the 2015 NBC, a considerable increase in seismic lateral earth pressures and seismic inertial forces are expected, which may cause extensive damage to the structure. Some of the key reservoirs (i.e., Level 1) upgraded prior to 2003 may experience extensive damage despite being designed to the MCE earthquake with a return period of 1 in 10,000. This is partly due to the targeted seismic performance considered in seismic retrofits, which is less stringent than that of a "post-disaster" structure. The following summarizes the Damage Levels selected for facilities according to FEMA (2020).

Estimated Damage Levels at Reservoir Sites

Reservoir	Year of Construction	Year of Seismic Retrofit	Seismic Event for Retrofit or Design	Importance Factor	Damage Level
Prospect	1962	2003	MCE	1.0	s.15(1)(l) [redacted]
Vancouver Heights	1928/1968	1996	MCE	1.0	
Little Mountain	2003	2003	MCE	1.0	
Kersland	1954 and 1958/59	1997	MCE	1.0	
Sasamat	1964	2021	2015 NBC	1.5	
Burnaby Mountain	1971	2017	2010 NBC	1.5	
Westburnco	1967/68	2004	MCE	1.0	
Whalley	1966	2006	MCE	1.0	
Kennedy Park	early 1980's	2012	2005 NBC	1.0	
Hellings Tank	1973	2015	2010 NBC	1.5	
Newton	1976/1986	2010	2005 NBC	1.5	
Sunnyside	1971 & 91	2021	2015 NBC	1.5	
Central Park	1974/1975	1998	MCE	1.0	
Pebble Hill	1971/1977/1990	2019/2020	2015 NBC	1.5	
Clayton	2018	-	2010 NBC	1.5	
Glenmore tank	1989	-	1980 NBC	1.0	
Cape Horn	1980	1980	MCE	1.0	
Grandview	1999	-	1995 NBC	1.0	

Estimated Damage Levels at Reservoir Sites

Reservoir	Year of Construction	Year of Seismic Retrofit	Seismic Event for Retrofit or Design	Importance Factor	Damage Level
Greenwood	1984	-	1980 NBC	1.0	s.15(1)(l)
Maple Ridge	1983	-	1980 NBC?	1.0	
Jericho	2023?	-	2010 NBC	1.5	

Estimated Damage Levels at Pump Station Sites

Pump Stations	Year of Construction	Year of Seismic Retrofit	Seismic Event for Retrofit or Design	Importance Factor	Damage Level
Barnston/Maple Ridge	2018	-	2005 NBC	1.5	s.15(1)(l)
Burnaby Mountain	1967	2011?	2005 NBC?	1.5	
Cape Horn	1980/94	2018	2015 NBC	1.5	
Capilano	2006	-	1995 NBC?	1.0	
Central Park	1974	2004	MCE	1.0	
Cleveland Dam	1973	2017	2015 NBC	1.5	
Grandview	1991	2014	2010 NBC	1.5	
Kersland	1963	-	-	-	
Little Mountain	1969	-	-	-	
Mahon	1984	-	-	-	
Newton No. 1 & 2	1966	2013	2010 NBC	1.5	
Pebble Hill	1972	2019	2015 NBC	1.5	
Sasamat	1964	2014	2010 NBC	1.5	
Vancouver Heights	1958	2004	1995 NBC	1.0	
Westburnco No. 1	1971	2003	MCE	1.0	
Westburnco No. 2	1999	2012	2010 NBC	1.5	
North Delta/ Hellings	1971	-	Pre 1970 NBC	1.0	
Greenwood	2003	-	1995 NBC?	1.0	

Failure of inlet and outlet pipes is identified as the most likely failure scenario that can impact the post-earthquake serviceability of reservoirs. This may occur even if the structure remains undamaged. It is not possible to estimate the likelihood of damage of inlet/outlet pipes without conducting a site-specific assessment. Other forms of damages such as roof failures, shear and flexural failures of the wall and columns may also occur in underground reservoirs and pump stations. Potential geotechnical vulnerabilities should be investigated at s.15(1)(l) to determine the cyclic softening potential of underlying clay-like soils. More recent seismic assessments conducted by Metro Vancouver have identified bearing capacity issues at s.15(1)(l) which also require further investigations.

Future Actions and Recommendations (Section 6)

Several action items have been suggested to improve the predictions of this vulnerability assessment. For water crossings, additional site-specific assessments are required to confirm their Category and quantify the damage. For facilities, it is recommended to undertake a separate study to assess seismic vulnerability of inlet/outlet pipes and their connections. Additional structural analysis is recommended for few select structures to estimate the seismic vulnerabilities of these structures due to the anticipated increase in seismic demand in the 2020 NBC. Priority may be given to critical reservoirs (identified as Level 1 reservoirs) which were retrofitted in the 1990's and early 2000's. Additional geotechnical assessments are also recommended for pump stations and reservoirs that indicated potential cyclic softening or bearing capacity concerns. The "non-structural" vulnerabilities should be assessed based on a site walkthrough.

As per Metro Vancouver's request, the report also includes a brief description of potential seismic resiliency measures such as design considerations, addition of isolation devices, developing an emergency preparedness and response plan and incorporating sufficient redundancy in the network.

The seismic vulnerabilities of watermain and facilities will be used by Metro Vancouver to identify the supply components that are most likely to be resilient to a major earthquake. Taking the impacts into account, the hydraulic analysis of the regional system will guide the selection of the nearest supply points for each member jurisdiction. The results will be also used to develop resiliency and emergency response plans for the water supply system that will address how basic service will be provided to the region after a major seismic event.

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C PIPE DAMAGE RATES

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G TERMS OF REFERENCE

ACRONYMS

AC	Asbestos Cement pipes
ALA	American Lifeline Alliance
ASCE	American Society of Civil Engineers
ASTM	American Society of Testing and Materials
D	pipe diameter
DI	Ductile Iron pipes
DBE	Design Basis Earthquake
EBMUD	East Bay Municipal Utility District
GIS	Geographical Information System
H	burial depth of the pipe measured from the ground surface to the centerline of the pipe
MCE	Maximum Credible Earthquake
MoP	Manual of Practice developed by ASCE that provides guidelines on the seismic design of utilities
NBC	National Building Code
OBE	Operating Basis Earthquake
K_0	lateral earth pressure coefficient corresponding to “at rest” conditions
K_1	an empirical factor defined in ALA (2001) to account for different pipe materials on the estimation of repair rates caused by transient ground displacement.
K_2	an empirical factor defined in ALA (2001) to account for different pipe materials in the estimation of repair rates caused by permanent ground displacement.
K_c	corrosion factor (see Section 4.1.3 and Table 4-1)
K_s	soil settlement factor to account for the built-in stresses in the pipe subjected to long-term settlement of soil (see Section 4.1.6)
K_A	factor to account for the welding type (see Section 4.1.3)
L	pipe length impacted by lateral spreading when the ground displacement occurs along the pipe axis
L_1	pipe length in question (in metres) when the pipe is subject to ground movement perpendicular to the pipe axis (This was selected as 300 m if the pipe was located 300 m from a waterbody, and 600 m if the pipe is located more than 300 m from the waterbody)
L_2	pipe length between two joints (m). The pipe segment length typically ranges from 6 m (20 feet) to 12 m (40 feet)
LADWP	Los Angeles Department of Water and Power
PGA	Peak Ground Acceleration
PGD	Permanent Ground Displacements
PGV	Peak Ground Velocity
PE	Polyethylene pipes
PVC	Polyvinyl Chloride pipes
PRCI	Pipeline Research Council International
PSI	Pipe-Soil Interaction
$P_{p,i}$	probability of failure of the pipe itself for a given range of strain (see Table 4-3).
$P_{c,i}$	probability of failure of a lap welded connection for a given range of strain (see Table 4-4).

$RR_{(TGD)}$	estimated repair rate under transient ground displacements
$RR_{(PGD)}$	estimated repair rate under permanent ground displacements
$S_a(T)$	spectral acceleration for a given period (T)
SPU	Seattle Public Utilities
TGD	transient ground displacements
t	pipe wall thickness
δ	interface friction angle between pipe and soil
ε_y	yield strain of steel
ε_{cp}	compression strain limit in the pipe
$\bar{\gamma}$	average soil density of the surrounding soil
σ_y	yield stress of steel

1 INTRODUCTION

1.1 BACKGROUND

Metro Vancouver has retained WSP Canada Inc. (WSP) to conduct a regional-scale seismic vulnerability study of their drinking water transmission system. The main water sources for Metro Vancouver's drinking water supply system are the Capilano, Seymour and Coquitlam reservoirs. Water from these sources is distributed to member jurisdictions via a network comprised of a series of reservoirs, pump stations and approximately 500 km of transmission mains. The main objective of the study is to identify the pipe segments and facilities that are likely to be resilient to a strong seismic shaking. The results will be used to identify the supply points at which a basic service can be provided to member jurisdictions after a major seismic event.

In broad terms, the proposed work scope includes the following:

- Develop seismic hazard maps to depict the earthquake induced ground displacement hazard and ground motion amplification;
- Evaluate the vulnerability of pipelines from earthquake-induced ground displacements and ground motion amplification;
- Evaluate the seismic vulnerability of facilities; and
- Develop a prioritization strategy and identify action items.

1.2 1993 EQE STUDY

In 1993, Metro Vancouver retained Kennedy/Jenks Consultants Engineers and Scientists and EQE Engineering (EQE, 1993) to undertake a similar regional-scale study to assess the seismic vulnerability of the water transmission network. The study included 27 facilities (8 reservoirs, 10 pumping stations and pump houses, four tanks, and three chlorination and/or treatment stations and buildings, one emergency power building, and one control building). At that time, the transmission system consisted of 432 km of pipelines. The number of facilities and overall pipe length have changed in the current study due to the system upgrades undertaken since 1993.

The 1993 EQE study considered two benchmark events; (i) "Operating Basis Earthquake" (OBE) based on a 100-year event, and (ii) "Design Basis Earthquake" (DBE) based on a 475-year event. The seismic demands for the 1 in 100-year and 1 in 475-year return period events were based on the seismic models included in the 1990 National Building Code of Canada (NBC). The study concluded that the system would remain operable after the OBE event, but would be severely impacted by the DBE event.

The performance of the OBE level earthquake is not discussed in this report because the Metro Vancouver Seismic Design Criteria (2015) requires transmission mains to be designed to a much larger earthquake intensity (either to a return period of 1 in 975, 2475 or 10,000 years) depending on the pipe classification. Maintaining this high-level of performance is consistent with seismic design guidelines adopted by other water distribution agencies and national/provincial design codes.

The key findings in the EQE study related to the DBE event are:

- A total of 173 km (out of 432 km) of pipeline were in areas underlain by liquefiable soil (i.e., 40 percent).
- EQE (1993) estimated approximately 31 watermain failures (over a length of 432 km), making much of the system inoperable. Apart from the water crossing portions of the Capilano Main No. 7 and Seymour No. 3, which were classified as moderate risk, all other water crossings were identified

as high risk. Permanent ground deformation caused by soil liquefaction was attributed to 30 failures while only one failure was attributed to seismic shaking.

- There was a high probability that the transmission system delivering water south of the Burrard Inlet would be inoperable as a result of flow slides/lateral spreading predicted at the Burrard Inlet crossings and failures of the transmission mains originating from Coquitlam Lake.
- All pump stations would likely be inoperable due to non-structural and structural damages or due to power outages.
- All reservoir roofs/columns and perimeter walls were classified as deficient, while some may collapse. However, it was predicted that, in most cases, the damage may not impact the hydraulic integrity of the reservoir.

Following the 1993 EQE study, nearly all reservoirs have since been upgraded with the aim of allowing the facility to operate at near full capacity after an earthquake. The upgrades completed by Metro Vancouver also included the complete replacement of the Little Mountain Reservoir, one of the oldest, largest and one of the most important reservoirs in the system. However, some of these upgrades do not necessarily meet the required performance level as a result of increased seismic demands in more recent codes.

Generally, the pump stations are not considered as high priority for seismic retrofit compared to other infrastructure. However, since the 1993 EQE study, we understand that 12 pump stations have been seismically upgraded or will be upgraded in the near future.

1.3 REASONS FOR UPDATING 1993 SEISMIC STUDY

An update to the 1993 EQE study is required due to following reasons:

1. The previous seismic assessment was based on the 1 in 475-year and 1 in 100-year return period events, which were determined under a different seismic model. These seismic demands are considerably smaller than those required by the Metro Vancouver Seismic Design Criteria (2015) where pipes are to be designed for an earthquake with a return period of 1 in 975, 2475 or 10,000, depending on the pipe classification (see Table 1-1 below).

	Metro Vancouver Seismic Design Criteria		**1993 EQE Study**	
Facility/Pipeline	**Performance Level or Importance Category**	**Annual Exceedance Probability**	**Performance Level or Importance Category**	**Annual Exceedance Probability**
Water treatment plants, Water disinfection facilities and pump stations	Post-disaster	1/2475	DBE	1/475
Pipelines crossing major water bodies	Post-disaster or Class IV (Essential)	1/10,000 or 1/2475	OBE	1/100
Pipelines- water	Class IV (Essential) or Class III (Critical)	1/2475 or 1/975	DBE	1/475

Table 1-1: Seismic performance requirements given in Metro Vancouver Seismic Design Criteria and 1993 EQE study

2. Beside the changes to the design return period, the post-earthquake performance requirements have changed as indicated in Table 1-1. For example, critical watermains and facilities shall meet the "post-disaster" performance requirements as outlined in the Metro Vancouver Seismic Design Criteria (2015) and other applicable design codes.
3. Metro Vancouver has conducted several seismic retrofits and upgrades to their system since 1993 and the seismic performance shall consider the impact from these upgrades.

4. Lessons learnt from various earthquakes throughout the world continues to improve our understanding of pipe behaviours and their interaction with soils during seismic events. New guidelines have been developed by various agencies such as American Society of Civil Engineers (ASCE), East Bay Municipal Utility District (EBMUD), Seattle Public Utilities (SPU), and the Los Angeles Department of Water and Power (LADWP). Our understanding of the behaviour of linear infrastructure, including fittings and joints under seismic conditions has somewhat improved due to earthquakes (e.g., 1994 Northridge, 1995 Kobe, 2010 Christchurch, 2016 Kumamoto). Some of the key fragility based relationships used for regional-scale studies were developed after the 1993 study (e.g., ALA 2001).
5. Even if the return period of the seismic event were to remain the same, the seismic hazard for the given return period has been updated several times in the last few decades. The most recent seismic hazard model (i.e., sixth-generation seismic hazard model of Canada) is expected to be released in 2022, with the 2020 edition of NBC.
6. Some of the design methods have been updated in recent years. For example, methods available to estimate the seismic liquefaction and ground displacements have been revised considerably since the early 1990's. This includes the 2002 and 2009 updates to the Bartlett and Youd (1992) method for estimating the lateral spreading. Also, the sloshing forces acting on the roof of the reservoir was not incorporated into design codes prior to 2000 (ALA, 2001).
7. Some of the existing pipelines and facilities were not included in the 1993 EQE study; and
8. Much of the system infrastructure has aged since the 1993 EQE study, potentially increasing the seismic vulnerability from aging and corrosion.

Besides Metro Vancouver's regional-scale study, FortisBC (formerly BC Gas and Terasen Gas) commissioned a seismic risk assessment in 1994 for its key assets within the Lower Mainland. The work was carried out by EQE International with assistance from Golder Associates. The assessment included key stations, transmission pipes and intermediate pressure pipes greater than 200 mm in diameter. This seismic assessment was updated in 2010 to account for the changes to the seismic hazard models. Consistent with the 1993 EQE study for Metro Vancouver, the most vulnerable areas of the pipeline were identified near major river crossings. The study identified approximately 49 pipe segments as high risk of rupture from a 1 in 2000-year return period event. This included 500 mm (20 inch) and 600 mm (24 inch) transmission pipes crossing the North Arm and South Arm of the Fraser River and the Pattullo site. Metro Vancouver's water supply network also includes major water crossings at these locations. Additional regional-scale seismic vulnerability assessments have been commissioned by the City of Surrey (Golder, 2000) and City of Coquitlam (WSP, 2020). The details of these seismic vulnerability assessments were reviewed as a part of this study.

1.4 LIMITATIONS

Seismic vulnerability assessments provide an indication of the "relative" vulnerability. As discussed in this report, the existing analytical methods alone may not be able to estimate the level of damage under seismic loading conditions due to the large number of variables involved. Any regional-scale assessment is limited by the quality of data that can be incorporated into the evaluation methodology. A statistically significant database is not available to adequately quantify the probability of failure of welded steel pipes due to lack of historical pipeline failure data for such pipes.

Subjectivity is found in all regional-scale vulnerability assessments and it is difficult to avoid even if a more systematic approach is followed. In general, an informal/qualitative approach to assessment does not hold up well to scrutiny, since the process is often poorly documented and not structured to ensure objectivity and consistency of decision making (Muhlbauer, 2003).

The vulnerability of a pipe or facility is not static and is expected to change with time. Natural and man-made activities, including seismic upgrades that may occur in the future, will alter the vulnerability ranking of a site making the current assessment outdated.

A regional-scale assessment comprised of large geographical areas; therefore, cannot be sensitive to local variations in soil and topographical conditions. The vulnerability assessment includes certain assumptions to quantify the seismic hazard. Results should be used with some caution as the analysis cannot fully incorporate site-specific factors that may impact the performance.

The study has identified seismically vulnerable areas within the Greater Vancouver region. This will have implications to current residents or other consequential impact to property value. In undertaking these studies to assist Metro Vancouver, we expect WSP to be indemnified against third party claims that might arise from loss of property value.

1.5 AVAILABLE INFORMATION

The details of pipelines and facilities were included in the following Operations and Maintenance (O&M) drawings prepared by Metro Vancouver:

- O&M - Water Book - October 2020: This document includes details of pipelines, including the alignment, pipe material, construction year, pipe diameters, pipe wall thicknesses, type of pipe connection and locations of various appurtenances. However, in most instances, the details are insufficient to determine the pipe burial depth and additional details related to pipe connection type (e.g., gas/arc or single or double lap welded joints). The drawings also include limited details for reservoirs, including their locations, capacity and pipe arrangements only.
- O&M - Water Book - Pump Stations - October 2020: This drawing set includes pump station locations, dimensions, indoor layout, connected pipe details, and key electrical and mechanical components (e.g., control chamber, valves, motors).
- O&M - Water Book - Seymour Capilano Filtration Plant - October 2020: This document includes details of the different components of the Seymour Capilano Filtration Plant such as the Capilano Energy Recovery Facility, Lynn Valley Water Storage Reservoir, ring main pump station, and sanitary sewer pipelines and connections.
- O&M - Water Book - Drinking Water Treatment Facilities - October 2020: This drawing set includes details of the different components of various secondary disinfection stations in the Greater Vancouver area including Central Park, Pitt River, Cape Horn, Kersland, Clayton, Boundary and Eton, and Newton sites.

A total of 123 reports have been provided by Metro Vancouver as source material for this seismic assessment. A breakdown of these reports is given below for each type of asset:

- Reservoirs (41 reports) – These includes geotechnical design and factual reports, structural evaluation reports, condition assessment reports and construction records. Most relevant details related to the status of seismic retrofitting are included in the structural assessment reports.
- Pump stations (21 reports) – Compared to reservoirs, only a limited number of reports are available for pump stations.
- Pipeline (61 reports) - These includes geotechnical design and factual reports, vulnerability assessments (seismic and non-seismic), and condition assessments.

Above included the following two documents, which provided additional information pertaining to water crossings and seismic performance of facilities:

- AECOM (2021) Condition Assessment Guidelines - Water Main River Crossings"
- Metro Vancouver (2018) "Seismic Records Study of GVWD Reservoirs, Tanks and Pump Stations”

In addition to the above information, the surficial geology maps published by Geological Survey of Canada (GSC) have been reviewed to identify geological units of concern based on their origin. The maps were superimposed onto Google Earth to aid in the interpretation of soil conditions and seismic ground response at a given facility or pipe segment. For evaluating landslide susceptibility, we have reviewed the available ground elevation contour /terrain attribute maps, digital elevation maps and LiDAR data.

In addition to the above, the seismic retrofitting conducted on reservoirs have been published in several conference papers, and are listed below:

- Atukorala, U., Puebla, H., Olivera, R., Honneger, D., Qian, M., Grant, M (2012) Challenges in Assessing the Seismic Vulnerability of two Water Main River Crossings in British Columbia, Canada, 15 WCEE Conference.
- Ayalp, M., Kemp, B., Sukumar, A.P., and Huber, F. (2006). Seismic upgrade of the Westburnco reservoir, Proceedings of the 8th U.S. National Conference on Earthquake Engineering., San Francisco, California, USA.
- Nikolic-Brzev, S. and Sherstobitoff, J. (1999). Seismic rehabilitation of a water reservoir using seismic dampers, 8th Canadian Conference on Earthquake Engineering, Vancouver, BC.
- Sherstobitoff, J. and Nikolic-Brzev, S. (1999) - Seismic upgrade techniques for concrete reservoirs, 8th Canadian Conference on Earthquake Engineering., Vancouver, BC.
- Sherstobitoff, J. Siu., D., Stewart., I., Chen, Q. (2004) – First Narrows and Port Mann Water Supply Crossings Seismic Vulnerability Assessment, 13th World Conference on Earthquake Engineering, Vancouver, BC.
- Sukumar, A.P., Sherstobitoff., J., Huber F (2004) Little Mountain reservoir reconstruction to meet MCE Performance Criteria, 13th World Conference on Earthquake Engineering, Vancouver, BC.

1.6 EXTERNAL SUBJECT MATTER EXPERTS

Doug Honegger – D.G. Honegger Consulting

WSP retained Mr. Doug Honegger of D.G. Honegger Consulting as a Specialist Advisor for this project. He has over 40 years of experience in a broad range of consulting activities related to understanding the response of structures, structural components and equipment from earthquake hazards, blast and impact. Mr. Honegger is an expert in assessing the impact to buried pipelines from permanent ground deformation. He is well-recognized for his contributions to advance the state-of-practice through active laboratory and field research activities, as he was in charge of developing new industry guidelines for the design of natural gas and liquid hydrocarbon transmission pipelines for hazards related to earthquakes, landslides, and subsidence. Mr. Honegger is also a co-author of several pipeline design guideline documents, including ALA (2001) and PRCI (2004). He was part of the Metro Vancouver's 1993 EQE study, and FortisBC's regional-scale seismic vulnerability assessments conducted in 1994 and 2010. Mr. Honegger provided high-level input for the pipe vulnerability assessment.

Dharma Wijewickreme, Ph.D., P.Eng. – University of British Columbia

Dr. Dharma Wijewickreme is a Professor of Civil Engineering and Director of the Pipeline Integrity Institute (PII) at the University of British Columbia (UBC), Canada. He joined UBC in 2001 after serving in the engineering consulting practice for 11 years, with particular reference to the geotechnical design of pipelines and bridges. Prof. Wijewickreme's research focus is on pipeline geotechnical engineering and earthquake liquefaction of soils, and this work has resulted in over 150 publications in these subject areas. Wijewickreme has contributed widely to the understanding of the seismic response of silts, and he pioneered the Pipeline Integrity Institute at UBC that was established in partnership with the pipeline sector to advance the pipeline engineering practices and innovation. Prof. Wijewickreme is a Fellow of the Canadian Academy of Engineering, Canadian Society for Civil Engineering, and Engineering Institute of Canada. He participated in the 1994 and 2010 FortisBC' seismic vulnerability studies and the seismic assessment conducted for the City of Surrey in 2000. Prof. Wijewickreme provided high-level input related to geotechnical aspects of the assessment.

Both Prof. Wijewickreme and Mr. Honegger participated in WSP’s internal team meetings at various stages of the project and provided input for the report.

2 SEISMIC HAZARD

2.1 LOCAL SEISMICITY

The project area has the potential to be impacted by three sources of earthquakes (Rogers, 1992, 1994):

Shallow crustal earthquakes: These earthquakes occur at shallow depths of about 10 to 20 km from fault ruptures. For a design earthquake return period of 1 in 2475 years, the earthquake magnitude in the study area is generally expected to range from about 6.5 to 7.5 with strong shaking lasting for about 10 to 15 seconds. An example of this earthquake is the 1946 M7.3 Central Vancouver Island earthquake, which is recognized as the most destructive in western Canada. Although active faults have not been identified within the Metro Vancouver area, earthquakes that occur in other parts of British Columbia or Washington State have the potential to cause damages to Metro Vancouver's infrastructure.

Deep intraplate earthquakes: These earthquakes occur between the southern Puget Sound and southern Gulf Islands where the Juan de Fuca Plate is subducted beneath the North American Plate. The estimated epicenter is at a depth of about 40 to 60 km from the ground surface. Historically, magnitudes of M6.5 to M7.5 earthquakes have occurred approximately every 30 years in the Puget Sound region with strong shaking lasting about 15 to 45 seconds. The 1949 M7.1 Olympia, 1965 M6.7 Seattle-Tacoma, and 2001 M6.8 Nisqually earthquakes are some examples of deep intraplate earthquakes.

Subduction earthquake: Mega earthquakes occur due to the subduction of the Juan de Fuca Plate underneath the North American Plate. An earthquake magnitude of 9.0 or greater could occur if the rupture extends from the coast from Northern California to southern British Columbia. Although the ground motions will attenuate somewhat by the time they reach the Metro Vancouver region, strong ground-shaking could still last for three or four minutes. Similar ground-shaking in Sendai, Japan during the 2011 Tohoku earthquake caused significant damage to buried utilities. The peak ground acceleration (PGA) in the study area could range between 0.2 and 0.3g which is somewhat smaller than that expected from a crustal or deep intraplate earthquake. According to the Pacific Northwest Seismic Network, at least seven mega subduction earthquakes of approximately M9.0 have occurred in the past 3,500 years with an average return interval of approximately 500 years. Since the last M9.0 subduction earthquake occurred over 300 years ago, there is a 14 percent probability that a M9.0 earthquake would occur within the next 50 years (Steele, 2013).

2.2 CHANGES TO DESIGN SEISMIC DEMAND

Over the years, several versions of seismic hazard models have been adopted in Canadian design codes as the knowledge and sophistication of seismic hazard modeling have improved. The first seismic hazard model was based on a qualitative assessment of historical earthquakes and their potential impact to a given region. In the second-generation models published in 1970, a fully probabilistic seismic hazard was introduced. The hazard was represented using PGA for a probability of exceedance of 40% in 50 years (i.e., 1 in 100-year return period). In the third-generation seismic hazard model incorporated into the 1985 NBC, the peak ground velocity (PGV) was also included, while the probability of exceedance was lowered to 10% in 50 years (i.e., 1 in 475-year return period). In the fourth-generation seismic hazard model included in the 2005 and 2010 NBC, the seismic hazard values were provided for several spectral accelerations while the probability of exceedance was further lowered to 2% in 50 years (i.e., 1 in 2475-years). In this model, the seismic hazard from Cascadia subduction events was considered in a deterministic manner. In the current fifth-generation seismic hazard model included in 2015 NBC, the influence from Cascadia subduction earthquakes was incorporated into the probabilistic framework with some additional updates to the ground motion and site amplification relationships. The sixth-generation seismic hazard model will be included in the 2020 NBC which will be released in early 2022. The draft seismic hazard values were published in February 2020 by Geological Survey of Canada for 624 locations in Canada, including 12 locations within the Greater Vancouver region. As per early discussions with Metro Vancouver, it was decided to use draft seismic demands from the 2020 NBC to determine the seismic deficiencies and vulnerabilities in the water distribution system. A summary of these seismic hazard models and design return periods are given in Table 2-1 below.

<table>
<tr><th>Year</th><th>Design Return Period</th><th>Ground Shaking Intensity Measure</th></tr>
<tr><td>1953</td><td>Qualitative</td><td></td></tr>
<tr><td>1970</td><td>100</td><td>PGA</td></tr>
<tr><td>1985</td><td>475</td><td>PGA, PGV</td></tr>
<tr><td>2005</td><td rowspan="4">2475</td><td rowspan="2">Sa(0.2), Sa(0.5), Sa(1.0), Sa(2.0), PGA (hazard from subduction events determined determinstically)</td></tr>
<tr><td>2010</td></tr>
<tr><td>2015</td><td rowspan="2">Sa(0.2), Sa(0.5), Sa(1.0), Sa(2.0), Sa(5.0), Sa(10.0), PGA, PGV</td></tr>
<tr><td>2020</td></tr>
</table>

PGA – Peak Ground Acceleration; **PGV** – Peak Ground Velocity; **Sa(T)** – Spectral Acceleration for a given period T

Table 2-1: Evolution of seismic return periods and ground shaking intensities

Some of the reservoirs and pump stations were designed or upgraded based on the Maximum Credible Earthquake (MCE) with a return period of 1 in 10,000 years. In the earlier seismic assessments, the PGA for the MCE event was 0.5g, which is approximately similar to those expected from the 2020, 2010 and 2005 NBC models for a 1 in 2475-year return period event. However, the MCE response spectrum is about 10 to 20% higher than the draft 2020 NBC in the 0.2 to 2.0 seconds period range. We understand that the response spectrum for MCE event was derived by Dr. D. L. Anderson using the BC Hydro model and was provided in Sandwell (2006) for Kennedy Park reservoir. Although we could not confirm if the same or similar spectrum was used for all other structures evaluated for the MCE event, based on the references to the PGA value, it is likely that above response spectrum was used in most of Metro Vancouver's seismic upgrades. Subsequent probabilistic seismic hazard assessments conducted for the MCE event have indicated much larger seismic hazards for the MCE event than those considered for seismic retrofitting of reservoirs. This updated response spectrum for the MCE is not shown Figure 2-1.

Figure 2-1 compares the firm ground seismic accelerations obtained from the fourth, fifth and sixth-generation seismic hazard models for a location on Burnaby Mountain. For comparison purposes, the MCE seismic demands adopted for the seismic retrofitting of the Kennedy Park reservoir are also shown. Similar comparisons for the remaining 11 locations are included in Appendix A of this report. Note that comparisons were made with respect to the "firm ground" seismic hazard values.

The key observations from this review are:

- In the short period range: With the release of the 2020 NBC, a significant increase in seismic demand is expected compared to the 2015 NBC. Generally, this increase will range from about 15 to 35 percent. In particular, a significant increase in PGA is expected, which could impact the liquefaction potential of a given location and increase seismic lateral earth pressures acting on buried structures. Compared to the 2010 or 2005 NBC, the difference is not significant in the short period range. The response spectrum of the MCE event is about 10 to 20 percent higher than the 2020 NBC values in the short period range.
- In the long period range: The seismic demand in the 2005 and 2010 NBC versions is significantly smaller than the 2020 NBC values. This may not provide a true reflection of the differences in seismic demand since the seismic demand from the Cascadia subduction sources, which has a greater impact on the long period seismic hazard, were accounted deterministically in the fourth-generation seismic models. In the long-period range, the difference between 2015 NBC and 2020 NBC is relatively minor (about 10%).

In general, the design seismic demand is expected to increase with the adaptation of 2020 NBC as discussed further in Section 5.4.3 of this report. For facilities, the increase will be influenced by the period of the structure.

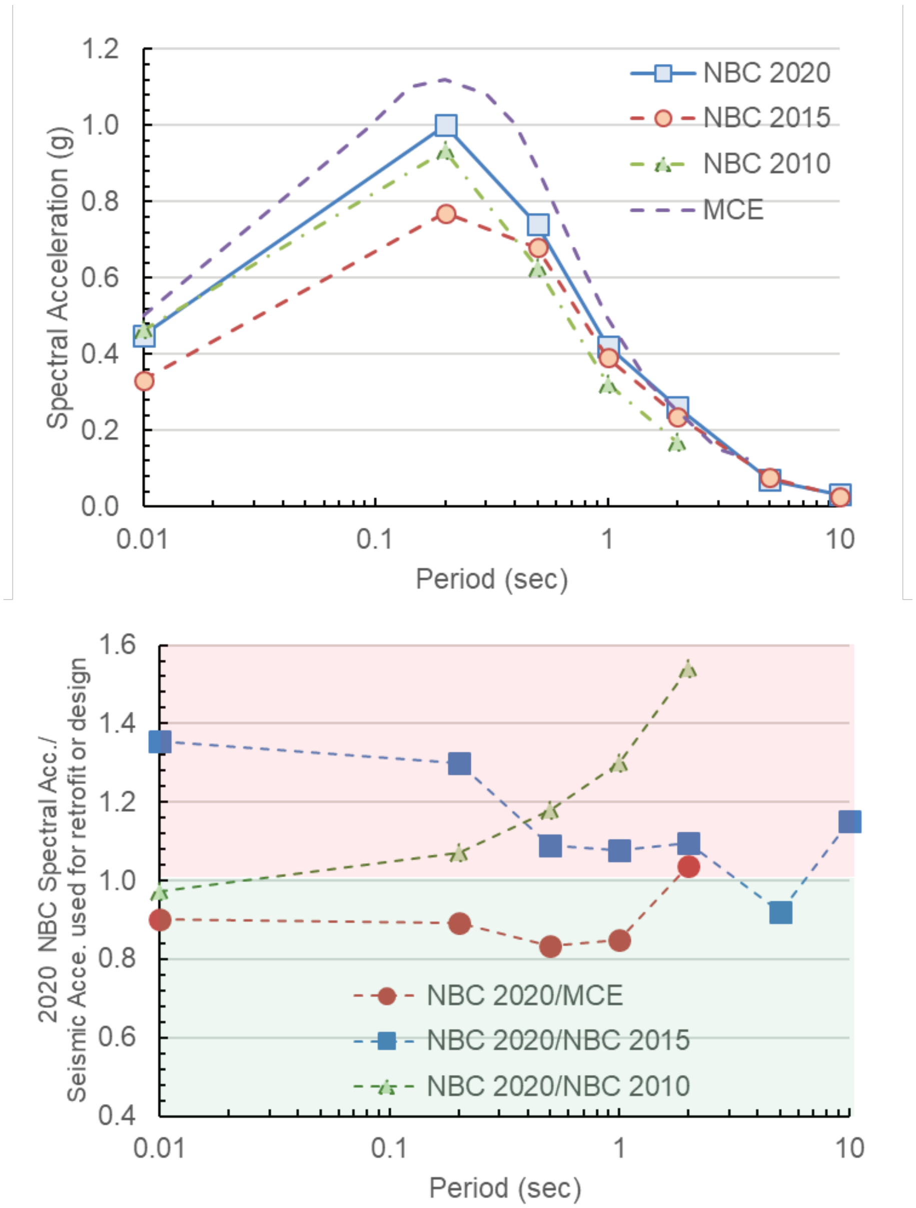

Figure 2-1: Comparison of firm-ground spectral accelerations from different design codes

2.3 DAMAGE FROM PERMANENT GROUND DISPLACEMENTS

Historical earthquakes indicate that a significant number of seismic damages to buried pipelines have been caused by permanent ground displacements (PGD) resulting from fault movements, landslides and liquefaction-induced lateral spreading, settlement and uplift. Liquefaction can occur in loose, saturated and relatively clean cohesionless soils where strong ground shaking causes an increase in porewater pressures within the soil deposit. The dissipation of excess porewater pressures will cause soils to settle and the amount of settlement depends on the ground motion intensity, density, and thickness of the liquefied deposit, among other factors. The combination of sloping topography and strength loss associated with seismic shaking can cause ground displacement in horizontal directions, which is commonly known as lateral spreading. The buried pipes are impacted by horizontal and vertical ground displacements as schematically shown in Figure 2-2.

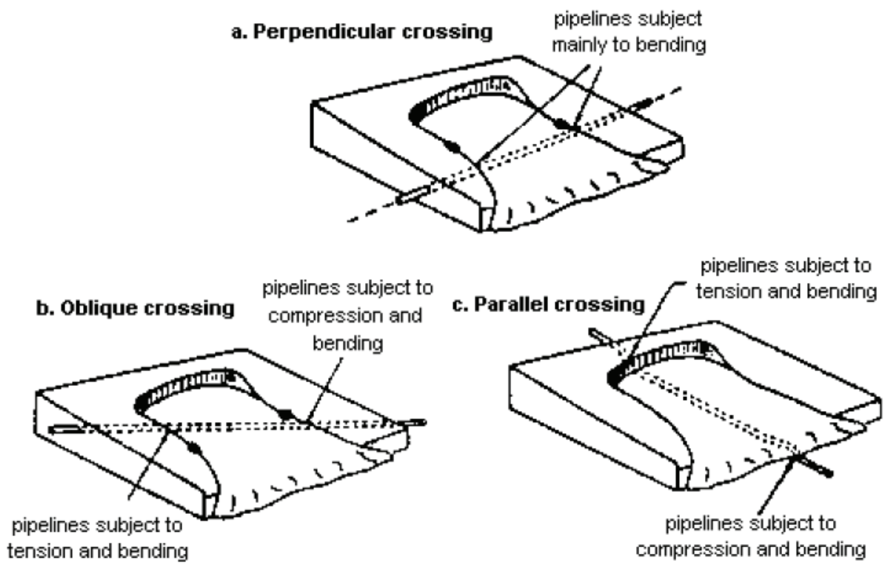

Figure 2-2: Impact of ground displacement on pipelines based on their orientation (after O'Rourke, 1998)

Even in the absence of soil liquefaction, slope instability (landslide) may occur in relatively steep slope areas as a result of strong seismic shaking. In general, seismically induced slope instability is a function of earthquake intensity, slope geometry (heights and inclination), soil composition and strength. As stated earlier, surface faulting is not recognized as a major concern for the transmission mains belonging to Metro Vancouver.

Compared to damages induced from seismic shaking that can spread over a large geographical area, the damages from PGD are concentrated in areas that experience soil liquefaction or landslide. For example, pipeline damages during the 1906 San Francisco earthquake were dominated by PGD due to widespread liquefaction, such that approximately 52 percent of pipeline breaks occurred within one city block in areas where liquefaction-induced lateral spreading was observed. This area was only 5 percent of the total area impacted by strong ground shaking (O'Rourke and Liu, 1999). Note that 1993 EQE study has identified 31 potential pipe failures in liquefiable areas compared to four in non-liquefiable areas.

2.3.1 LATERAL SPREADING AND FLOW SLIDES

Lateral spreading or flow slide occurs as a result of soil liquefaction on gently sloping ground or even on flat ground adjacent to a free face. Large ground displacements of several meters are referred to as flow slides, and in such situations the magnitude of ground displacement cannot be reliably estimated using existing methods. Lateral and vertical ground displacements are non-uniform resulting in large localized deformations of the ground. The non-uniformity of the ground deformation is further exacerbated by the spatial variability and severity of liquefaction. The damage potential is greatest where buried pipes transition from liquefiable soils to more competent material.

2.3.1.1 *MAGNITUDE OF LATERAL SPREADING*

Generally, the largest horizontal ground displacements are observed near water bodies and decrease away from the water body. For example, ground displacement profiles measured during the 1995 Kobe and 2010/2011 Christchurch earthquakes are shown in Figure 2-3.

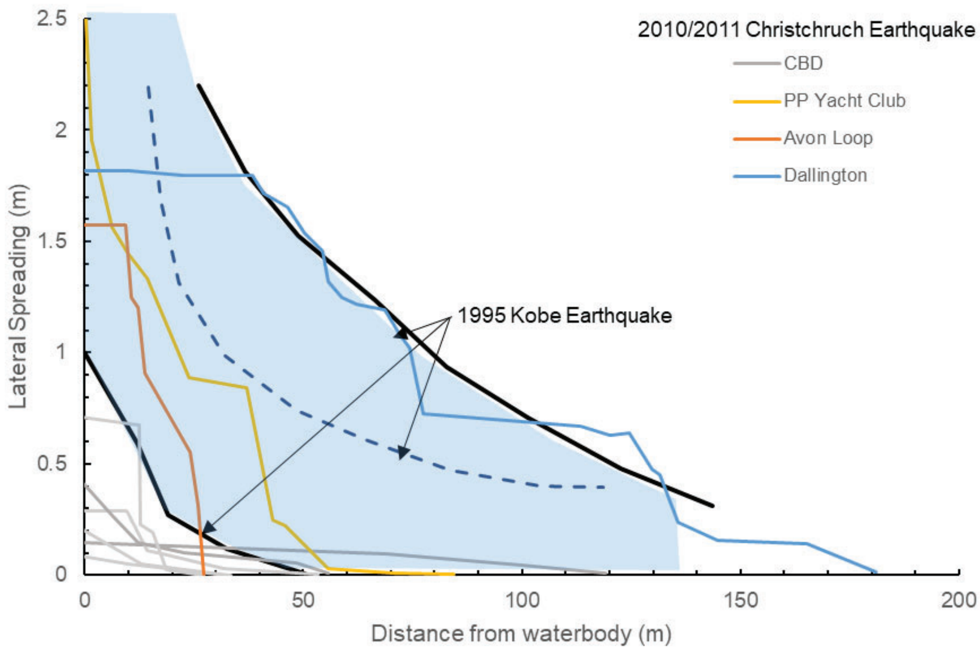

Figure 2-3: Distribution of lateral spreading estimated during the 1995 Kobe and 2010/2011 Christchurch earthquakes

The impacted zone generally extends 150 to 250 m from the free face. Beyond this point, the lateral spreading is largely governed by localized topographical features. Small slumps and soil failures could occur along drainage ditches/channels, although such failures are not a threat to pipes buried at a deeper depth than the invert of the drainage ditch. The length of pipeline exposed to such localized ground movements is very limited; therefore, the soil load is typically not sufficient to cause significant stresses/strains.

As highlighted in the following sections, numerical modeling (time-history analysis) has been conducted for most of the critical pipelines, while other pipes include ground movement predictions developed using simplified methods (e.g., Youd et al. 2002, 2009). For example, Figure 2-4, shows the range of lateral spreading displacements predicted using the Youd et al. (2002) method for Annacis Main No. 5 (after EBA, 2011). The actual lateral spread displacements measured during 1995 Kobe and 2010/2011 Christchurch events (Cubrinovski et al., 2012) are also shown for comparison.

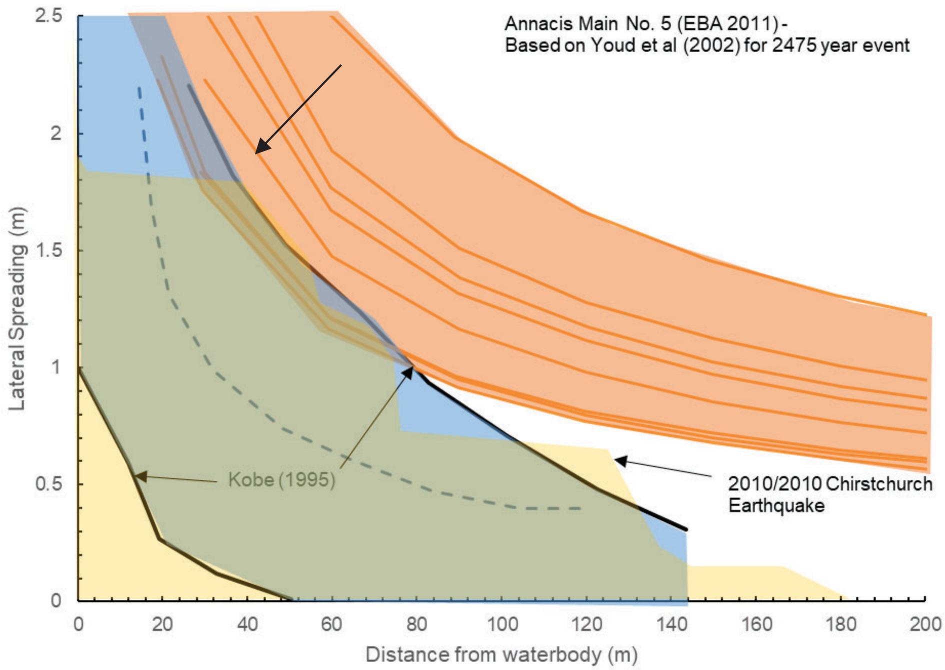

Figure 2-4: Estimated lateral spreading displacements for Annacis Main No. 5 (after EBA, 2011). Actual lateral spreading measured during the 1995 Kobe and 2010/2011 Christchurch earthquakes are also shown for comparison.

The magnitude of lateral spreading can also be estimated using advanced ground deformation analysis (e.g., FLAC or PLAXIS). Figure 2-5 shows some of the lateral spreading profiles estimated using the 2D FLAC for Haney Main No. 2 and Cambie-Richmond Main near Oak Street bridge. For comparison purposes, the displacement profiles estimated using the Youd et al. (2002, 2009) method are also shown. Somewhat similar trends can be observed despite the differences in locations, soil conditions, earthquake time histories and analysis approaches. We are aware that soil movements extending to deeper depths can sometimes be predicted using numerical modeling forcing the watermain to adopt a deeper profile to avoid soil layers that contribute to the movement. Such observations are not consistent with simplified empirical methods that do not suggest ground displacements in deep non-liquefiable soils or below a certain depth (e.g., Youd, 2018).

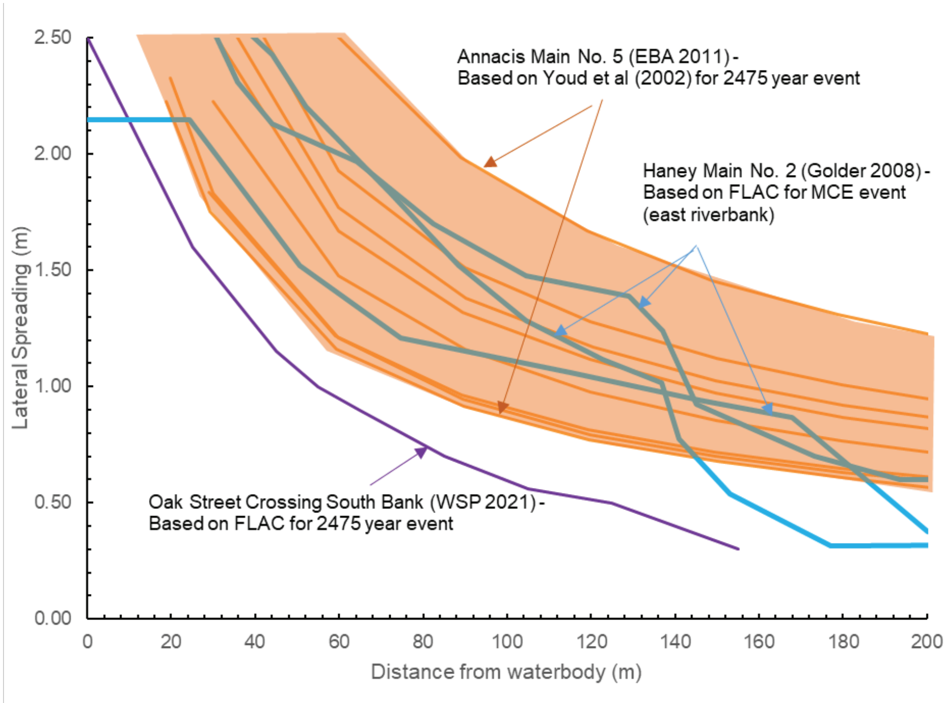

Figure 2-5: Estimated lateral spreading displacements using FLAC (after Golder 2008 and WSP 2021). Lateral spreading estimated using the Youd et al. (2002, 2009) are also shown for comparison.

Besides the smaller pipe sections or river crossings, there are only a couple of pipelines for which site-specific lateral spread estimations are not available. These include the following:

1. River Road Mains (West) - As indicated above, the influence zone of lateral spreading typically extends 150 to 250 m from the free face. For the River Road mains, the lateral spreading was estimated based on those estimated for the Annacis Main No. 5 considering the similarities in soil conditions (see Figure 2-6). There is some judgement involved in the selection of this displacement profile. A somewhat shaper drop in lateral displacement was considered as the distance from the river increases, which was consistent with actual case histories.
2. Marpole Crossover Main – The lateral spreading profile was assumed to be similar to the Oak Street bridge considering its proximity to the watermain (see Figure 2-5).
3. Angus Drive Main between Fraser Park to Southwest Marine Drive – Given the proximity to the Oak Street bridge, lateral spreading profile was extracted from 2D FLAC analysis conducted for this bridge (see Figure 2-5).
4. Annacis Main No. 3 - Assumed to be similar to the lateral spreading estimated the Annacis Main No. 5 considering the similarities in soil conditions (see Figure 2-6).
5. Sections of Tilbury Main near South Arm crossing of Fraser River – Adopted from 2D FLAC analysis conducted by Golder (2007) – not shown herein.

For smaller pipe segments, the lateral spreading was estimated based on previous project experiences in nearby areas. These do not have a significant impact on the overall outcome of the study because of their smaller pipe length impacted by PGD. The PGD values considered for estimating the damage rates are given in Appendix C. In some instances, PGD values for river crossings are not included in the overall numbers since damage rates at those locations are estimated separately from site-specific assessments already conducted.

Even if site-specific studies have been performed, there is considerable uncertainty related to the estimation of the magnitude of lateral spreading. Traditionally, the actual ground displacement is estimated to vary between 0.5 to 2 times the best-estimate value.

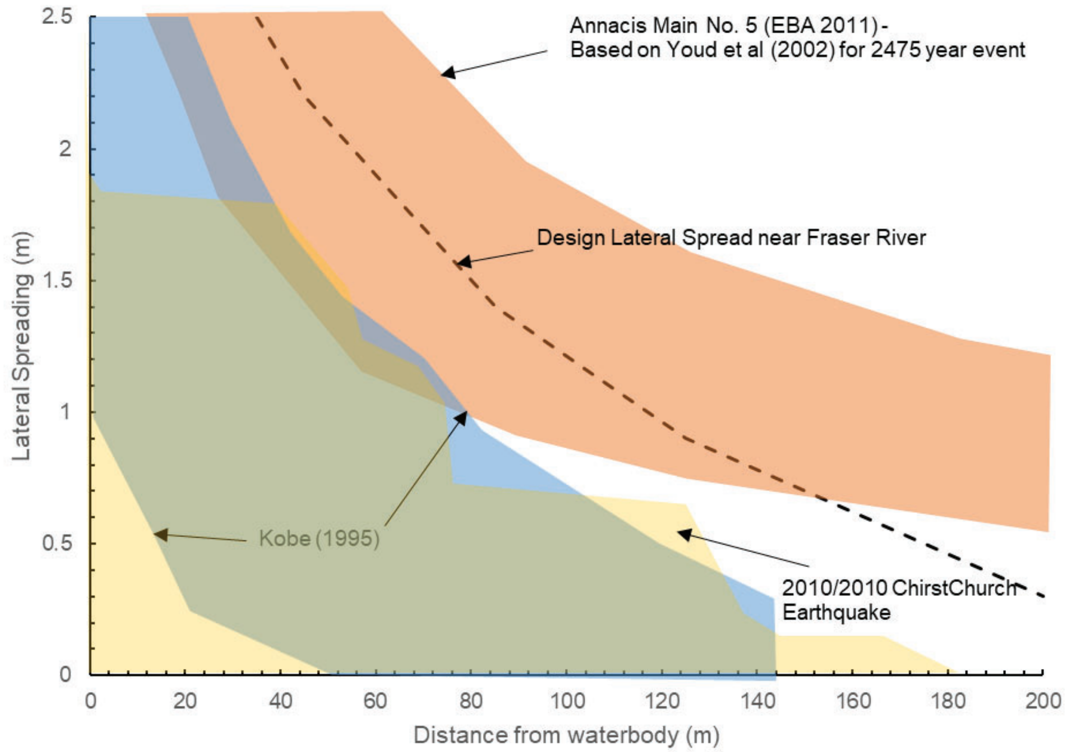

Figure 2-6: Lateral spreading displacement considered for River Road Main (East/West). Lateral spreading estimated using the Youd et al. (2002, 2009) method for Annacis Main No. 5 and past case histories are also shown for comparison.

2.3.1.2 *LATERAL SPREADING PROFILE*

The level of damage and soil load acting on the pipe will also depend on the shape of the ground deformation profile as shown in Section 4.3.3 of this report. This ground deformation profile depends on soil stratigraphy and is often difficult to estimate with confidence due to uncertainties in liquefaction manifestations. Typically, a rectangular (i.e., abrupt) ground deformation profile is considered if there is a sudden transition from liquefied to non-liquefied soil or between stable and unstable soil masses in some landslide areas. In contrast, if the ground conditions are expected to be more uniform, a cosine shaped ground deformation profile (i.e., y(x)) can be considered as recommended by M. O'Rourke (1989):

$$y(x) = \frac{\delta}{2}\left(cos\frac{2\pi x}{w}\right)$$

Where *x* is the distance along pipe axis, δ is the permanent ground displacement, and *w* is the estimated pipe length over which the ground displacement is occurring.

2.3.1.3 *WIDTH AND LENGTH OF LATERAL SPREADING ZONE*

Historical evidence indicates that the width of a lateral spread PGD zone generally varies from about 75 m (~250 feet) to 600 m (~2,000 feet). The length of a lateral spread PGD zone varies from a few meters to about 250 m (~800 feet). For this study, the width and length of the lateral spreading zone were selected from the statistical analysis completed by Honegger et al. (2014) using data from past case histories (O'Rourke and Hamada, 1992; Hamada and O'Rourke, 1992; Hamada et al. 1995). The results of this assessment were also relied upon for the regional-scale seismic vulnerability assessments conducted for FortisBC in 1994 and 2010.

The cumulative probability distribution of the width of the lateral spreading zone is shown in Figure 2-7 (Honegger, et al. 2014). A width of lateral spreading zone associated with a 50 percent cumulative probability is 100 m. This information is used in the pipe-soil interaction analysis discussed in Section 4.4.3.

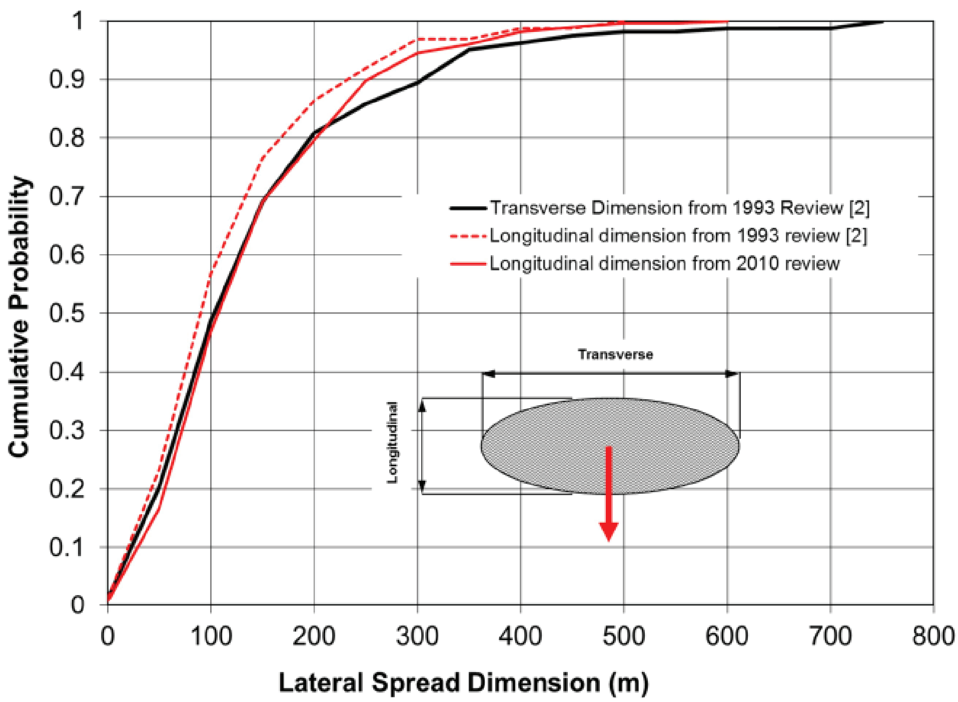

Figure 2-7: Width of lateral spreading distribution from past earthquakes (adopted from Honegger et al., 2014).

2.3.2 GROUND SETTLEMENT

PGD may also occur in the vertical direction as a result of the post-liquefaction settlement or floatation of the pipe or buried structure. In flat terrains, the post-seismic free-field settlement is expected to range from about 2 to 5% of the thickness of the liquefiable layer depending on the density of the soil and intensity of the seismic shaking. There is some uncertainty related to the liquefaction of deeper soil deposits. Even if liquefaction occurs at deeper depths, it is unlikely to result in a larger differential settlement near the ground surface. Therefore, in most instances, the ground settlements triggered from soil liquefaction are unlikely to exceed about 0.5 m. As a result, vertical displacement is considered as a lesser threat to pipe integrity compared to horizontal ground displacements. Notwithstanding that, much larger settlements have been reported in past earthquakes (i.e., in the range of 1 to 2 m), especially when the lateral movement of the soil mass contributes to the vertical displacement or heavy structures cause "shear-induced" deformations in combination with the free-field settlements described above.

Post-seismic settlements are expected to impact transmission mains situated in s.15(1)(l) [redacted] s.15(1)(l) [redacted] The PGD values selected for these pipes range from about 300 mm to 450 mm, and are summarized in Appendix C. These values were selected based on existing site-specific studies or previous project experience.

2.3.3 BUOYANCY

If the pipe is buried within liquefiable soils, uplift of the pipe may occur due to excess porewater pressures developed within the liquefiable soil and an associated loss of strength. According to PRCI (2009), the amount of displacement is generally limited to the distance between the bottom of the pipe and the water table or the depth of soil cover plus about one-third of the pipe diameter for cases where the water table is at the ground surface. This displacement is further limited by the drag forces created by the liquefied soil as the pipe moves through the soil. Pipes crossing waterbodies will float if the uplift resistance is less than the buoyant force exerted by the liquefied soil. O&M drawings have already identified Queensborough Mains No. 1 and 2 as susceptible to uplift if the pipe is empty. A somewhat larger PGD of 500 mm was considered for these two transmission mains to account for such uncertainties.

It is difficult to distinguish whether a structure was subject to uplift or if the apparent uplift failure was in fact due to the settlement of the surrounding ground (Rowland 2015). If uplift occurs, the flotation of the pipe typically results in moderate displacements which are distributed over a large pipe length. Modern pipes are typically able to accommodate such loading conditions.

Even if the pipe itself has adequate resistance to uplift, often the underground chambers are more vulnerable to this phenomenon. If the chamber is subject to uplift, the pipe entering or exiting the chamber will experience relatively large stresses near the transitions.

2.3.4 LANDSLIDES

Certain pipes are in areas of high landslide risk as a result of their proximity to steep terrains. To provide a perspective of the likelihood of seismically induced landslides, it is important to note that there were more than 300 recorded landslides over an area of about 20,000 km^2 during the M7.3, 1946 Vancouver Island earthquake. After the 1971 San Fernando earthquake, over 1,000 landslides were identified over a 250 km^2 of mountainous areas north of San Fernando valley. The most frequent were the surficial slides less than 1 m thick. Seismic-induced landslides could also manifest in the form of rockfalls, which are the most abundant type of earthquake-induced landslide.

Areas with historical landslides provide an excellent indicator of landslide susceptibility under seismic conditions. The likelihood of landslide increases if the area is already unstable under non-seismic conditions. Our assessment indicates that only a limited number of pipelines are predicted to be impacted by seismic induced landslides. This includes s.15(1)(l) [redacted]. Site-specific analysis has been already undertaken at some locations using more advanced methods s.15(1)(l) [redacted]. Therefore, these seismic assessments are likely to provide a more accurate estimate of the magnitude and extent of ground movement compared to those estimated using approximate methods pertinent to regional-scale studies. For s.15(1)(l) [redacted], slope displacements were used in subsequent pipe-soil interaction analysis to identify potential vulnerable locations. Therefore, this information was directly used to estimate the number of failures instead of carrying out separate analysis.

The corridor along the Pipeline Road has experienced several slope failures along the western hillside, where the headscarps of these slides are located up to 500 m above Pipeline Road. Large failures have been documented between 1963 and 1974, where the runout reached the Coquitlam River. The failures are likely to occur in the upper weathered colluvial layers. Aerial photographs have not revealed any deep-seated or widespread landslides. Furthermore, such failure surface may not intersect the watermains although there is a greater likelihood that failures would reach Pipeline Road and beyond. The runout may impact the stability of existing fill slopes and in turn the watermains along Pipeline Road. Seismic slope stability analysis conducted by EXP (2018) indicated a potential movement in the order of 150 mm for a slope encompassing the pipe. This information was used in the pipe damage estimations discussed in Section 4.3.3.

The results of the high-level seismic slope stability analysis completed by BGC (2009) for the District of North Vancouver was used in identifying the potential landslide areas and PGD. We are also aware that some remediation measures have been implemented at certain locations to mitigate the impact of landslides on the watermain s.15(1)(l) .

2.4 DAMAGE FROM TRANSIENT GROUND DISPLACEMENTS

Earthquake shaking will cause body waves and surface waves in the soil mass. Body waves are responsible for the radiation of seismic energy from the rupture zone to the surface. They are two types of body waves: P-waves (primary or compression waves) and S-waves (secondary or shear waves). Passage of P-waves cause disturbances parallel to the direction of wave propagation while S-waves cause disturbances perpendicular to the direction of travel (see Figure 2-8).

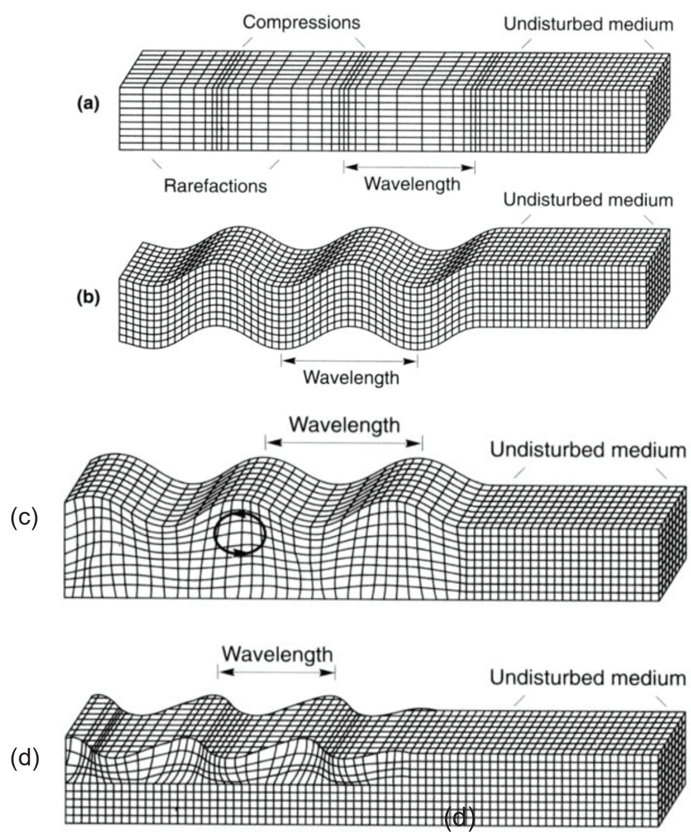

Figure 2-8: Deformations caused by (a) P-waves, (b) S-waves, (c) R-wave and (d) L-wave (after Bolt, 1993)

The ground displacements caused by the passage of these wave are transient in nature and are termed Transient Ground Displacement (TGD) in this report. Buried pipes may be damaged from these transient ground strains and curvatures due to the incoherence in the ground motion excitations along the pipe alignment. Also, buried pipes often pass-through different soil layers with varying soil properties resulting in differential displacements and stresses along the pipe. Observations from past earthquakes have shown that most pipeline damages occur from TGD when the shaking levels exceed the range 0.1 to 0.3g PGA. The effects of ground shaking can vary considerably, such that differential response between adjacent soil layers can cause additional axial pipe stresses. Hindy and Novak (1980) noted that the pipe stresses are highest near the boundary of two horizontally adjacent soil layers with different properties. In addition, appurtenances introduce discontinuities into the pipeline. They can take the form of junctions, valves, connections to structures and laterals. The pipe stresses near these discontinuities can increase considerably when they are subject to earthquake shaking. If a pipeline is attached to a structure, the structure may have a natural frequency that is different to the pipeline resulting in out of phase vibrations.

TGD is common to all earthquakes and affect wide geographical areas. Therefore, the pipe damages from TGD tend to spread over the whole pipe system unlike damages induced from PGD. While the damage rates are relatively low, damages from TGD cannot be ignored especially if the pipe system comprises brittle pipes and unrestrained joints (e.g., Concrete, Asbestos Cement and Cast Iron).

Past earthquakes have indicated that impacts from seismic shaking can vary considerably from one seismic event to another. For example, during the 1999 Chi Chi earthquake, approximately 48 percent of all buried water pipe damages were attributed to ground shaking (Shih et al, 2000), while the remaining damages were attributed to PGD. During the 1906 San Francisco earthquake 50 percent of damages were attributed to TGD. During the 1964 Puget Sound, 1969 Santa Rosa, 1983 Coalinga and 1985 Michoacán earthquakes, seismic wave propagation was identified as the main damage mechanism. Comparatively, during the 2010 Christchurch event only 19 percent of failures were attributed to TGD.

It should be recognized that most of these damages occurred in segmented pipes, particularly due to joint pullout (axial tension) or crushing or splitting of the belled portion of the bell-and-spigot joints and joint misalignment caused by lateral forces. For continuous pipes, there is considerable uncertainty related to the likelihood of damage from wave propagation. Damages from TGD is typically considered unlikely for these pipes, especially if they are constructed to modern construction standards.

The orientation of the pipe with respect to the wave propagation direction is a crucial factor that determines the damage potential. For example, pipes oriented parallel to a radial line extending away from the earthquake source are far more likely to suffer damage than a pipe that is perpendicular to the wave passage (O'Rourke et al., 1980; Takada et al. 2002). For example, during the 1923 Kanto, 1948 Fukui, and 1971 San Fernando earthquakes, pipes located parallel to the direction of the seismic wave propagation experienced severe damage than those with a perpendicular orientation. However, it is often difficult to predict the orientation of the wave propagation direction even if site-specific details are available. Therefore, fragility relations for TGD provide an “average” damage estimation with respect to pipe orientation considering the uncertainty in source direction and pipe orientation.

3 SEISMIC HAZARD MAPS

3.1 SURFICIAL GEOLOGY

The surficial geology of the region has been mapped in detail and published in various publications (e.g., Armstrong and Hicock, 1976; Armstrong, 1977; Armstrong, 1976; Mustard and Roddick, 1992); therefore, it is not discussed in detail in this report for each area. In terms of an overview, the referenced reports indicate that during Quaternary times, the area underwent two or possibly three major glaciations. Each glaciation was accompanied by two or three local ice advances (Hicock, 1976). During the last major glaciation (i.e., Fraser), two stades occurred (Vashon and Coquitlam), possibly separated by an interstade (Quadra). Consequently, the Quaternary geology of the area is one of the most complex due to the repeated glaciations, sea-level changes and modern deposits (Hicock,1976). As result of this complex history of glaciation and subsequent sediment deposition, bedrock is at or within 10 m of the surface in less than 5% of the Fraser Lowland, while the remaining areas are overlain by Quaternary sediments of 10 m to 300 m thick.

Most pipelines in Vancouver, Burnaby and North Vancouver municipalities are in relatively competent glacial till and pro-glacial deposits from Pleistocene era. The earliest of the Pleistocene deposits are the Pre-Vashon glacial, non-glacial and glaciomarine sediments deposited prior to, and heavily consolidated during the Fraser Glaciation. Vashon sediments primarily consist of very dense glacial tills and glaciofluvial outwash sands and gravels. During the recessional phase of the Fraser Glaciation, a complex succession of outwash and deltaic and marine deposits were deposited. These sediments are identified as Capilano Sediments and the Fort Langley formation.

Following the last glaciation, more recent deposits (Holocene or post-glacial) associated with the Fraser River and other watercourses have been deposited. In Richmond and Delta, the surficial geology is dominated by these Holocene-age deltaic deposits associated with the Fraser River (Claque et al., 1991). This includes thick layers of marine clays and silts overlain by deltaic and fluvial sands and silty sands, and floodplain silts and clays. The deltaic deposits vary from about 20 m in thickness at the North Arm to over 300 m further south (Dallimore et al, 1996). These recent sediments are typically characterised as loose or soft and are identified as potentially liquefiable even in a moderate earthquake. Similar deposits are encountered in the low-lying areas of Port Coquitlam, Pitt Meadows and Fort Langley. In addition to natural soil deposit, thick deposits of uncontrolled fills are also present in areas such as the False Creek area in Vancouver.

3.2 SEISMIC HAZARD MAPS

As stated above, the PGD and TGD are key factors that impact the seismic vulnerability. Structures exposed to such hazards are identified and evaluated by overlaying the pipe network and facilities on the hazard maps. The pipe sections are further divided into smaller segments based on changes in seismic hazard (liquefaction potential, ground motion amplification and magnitude and direction of ground displacement relative to the pipe) and pipe properties (e.g., diameter, wall thickness).

3.2.1 LIQUEFACTION AND SEISMIC-INDUCED LANDSLIDE HAZARD MAPS

Liquefaction "susceptibility" is the relative ease at which a soil deposit might liquefy if subjected to sufficient intensity of seismic shaking. It depends on the depositional mechanism, age, grain-size distribution, strength, density, depth, groundwater conditions, and cementation. Generally, the soil liquefaction is common in saturated geologically young (less than ~10,000 years old) deposits such as the late Holocene deposits, loose granular soils and low plastic fine-grained soils (Youd et al., 1975). Compared to liquefaction "susceptibility", liquefaction "potential" describes the likelihood that a soil deposit experience liquefaction. As a result, the liquefaction potential considers the geotechnical parameters and geological setting of the soil deposit as well as the seismic shaking intensity. Liquefaction potential is typically determined based on site-specific geotechnical and geological information.

Soils with a more porous matrix such as gravelly deposits or fine-grained deposits characterized as clay-like soils are typically not susceptible to liquefaction. However, low plastic sensitive clay-like soil may experience significant strength and stiffness degradation if its cyclic strength is exceeded from the earthquake shaking. Moderate and high plastic clay-like soils can also develop excess porewater pressures during undrained cyclic loading but generally do not reach zero effective stress. Hence, clay-like soils retain some level of stiffness and strength during cyclic loading and generally deform less than sand-like soils. Nonetheless, normally to lightly over-consolidated and sensitive clay-like soils can develop large excess porewater pressures that can trigger potentially large ground deformations depending on the ground geometry and external structural loads.

The region has been the subject of numerous site-specific geotechnical studies, and liquefaction and post-seismic ground displacements are well-known and often quantified by various consultants for different projects. Where site-specific data is limited, Youd and Perkins (1978) provides an approach for assessing the liquefaction susceptibility by considering the surficial geological data and the origin of the various deposits.

The areas with High, Medium and Low likelihood of liquefaction are shown in maps included in Appendix C. The maps show the liquefaction potential but not the magnitude or the severity of the ground movement that may impact the pipeline. For example, a pipe located within s.15(1)(l) has a high likelihood of liquefaction although the anticipated ground displacement may be small if it is located away from a waterbody or steeper slope. However, a pipe located in a steeper slope and with a low liquefaction potential may experience relatively large ground displacement due to seismic shaking. Furthermore, pipe may be intentionally located below the liquefiable layer. In other words, the ground displacement occurring at the pipe level is the key factor that impacts the pipe performance. Compared to mapping the likelihood of liquefaction potential, it is difficult to accurately predict the magnitude of ground displacement and its directions at a regional level (see Section 2.3).

Areas with a high likelihood of liquefaction were identified in municipalities such as s.15(1)(l) . The s.15(1)(l) is another area that is likely to be impacted by soil liquefaction and lateral spreading. This area includes key water crossings such as s.15(1)(l) The liquefaction assessments conducted by various consultants have identified thick deposits of potentially liquefiable layers at the north shore with the potential to impact these utilities.

Areas such as s.15(1)(l) area have been developed by placing mass fills over the native soil. The fill is highly variable and ranges from gravel, sand, silt and other construction materials and is often poorly compacted. Due to the variability in the type of fill material and compaction level, the liquefaction potential is expected to be highly variable.

The only area assigned a “Moderate” liquefaction potential occurs near the s.15(1)(l) area. The greatest uncertainty related to this classification is associated with the liquefaction potential of compact gravelly deposits encountered near the surface. Past earthquakes have demonstrated situations where gravelly soils have undergone liquefaction. For example, during the 1995 Kobe earthquake, widespread liquefaction was reported in well-graded reclaimed fills containing 30 to 60 percent gravels. In contrast, gravels with uniform particle-size have a lower liquefaction potential than sandy soils because of their high hydraulic conductivity and greater stiffness and strength (i.e., lower rate of excess pore pressure build-up, and smaller cyclic strains). Some of the loose to compact gravelly deposits are likely to be identified as potentially liquefiable based on the seismic hazard values included in the 2020 NBC. Even when liquefied, such gravels will experience a smaller shear strain and unlikely to cause ground displacements as large as those in sandy soils. The extent of liquefaction is expected to be relatively small compared to looser and more recent deposits. Furthermore, the area is relatively flat and located away from any major watercourses that may trigger large-scale lateral spreading displacements.

Regardless of the above discussions, the severity of liquefaction is not expected to be uniform and could vary substantially even within a geographical zone. As a result, the liquefaction hazard maps developed based on a regional scale review are not intended to identify the spatial variabilities at a smaller scale. These maps do not address any man-made alterations to ground conditions such as cuts and fills, which could alter the site response significantly. Also, the maps do not account for any areas of ground improvement that may reduce liquefaction potential and ground motion amplification.

3.3 GROUND MOTION AMPLIFICATION MAPS

Besides the maps presenting the liquefaction potential and seismic-induced landslide hazard, additional maps were developed to present the potential amplification of earthquake motions as they propagate through overlying soil layers. The most common measure of the earthquake shaking intensity is PGA. However, PGA itself is not a good indicator of damage to structures and buried utilities. For pipelines, areas of high PGA are associated with a higher pipe damage rate mainly due to the larger PGD (O'Rourke and Toprak, 1997). As the velocity is less sensitive to high frequency components of the ground motion, PGV is considered a better indicator of damage as it correlates well with transient strains induced in the ground. Furthermore, PGV is closely correlated to the energy of the earthquake. Compared to PGA, PGV values are not readily available; therefore, PGV was approximated from PGA using the relationship proposed by Seed et al., (1976).

3.3.1 EFFECTS OF SOIL TYPE

Ground motion amplification is strongly influenced by the nature of the near-surface geological materials. Areas with soft soils will experience higher amplifications than areas with stiff/hard soil or bedrock. Seismic hazard values provided by Geological Survey of Canada are for “firm ground” conditions, which represents very dense/dense or stiff/hard grounds where the measured (harmonic) average shear wave velocity in the upper 30 m is between 360 m/s and 760 m/s. As expected, firm ground conditions are not encountered throughout the region; therefore, the actual ground motion accelerations experienced by the pipe or facility will be altered as they propagate through overlying soil layers. For example, the earthquake ground motion is expected to amplify in areas with soft soils.

This ground motion amplification/de-amplification caused by overlying soils is partly reflected in the Seismic Site Classification for level ground conditions. The Site Class is typically determined from the average shear-wave velocity in the upper 30 m ($V_{s,30}$) or using other indices such as the Standard Penetration Test resistances or undrained shear strength in the upper 30 m. The intensity of ground shaking increases from Site Class A to E (e.g., Site Class A is associated with competent bedrock while Site Class E refer to sites with a soft soil overburden). The firm ground accelerations specified in NBC correspond to Site Class C conditions with an amplification factor of one. The maps representing the Site Class were developed by Taylor et al. (2006) using available borehole data and direct shear-wave velocity measurements. For areas poorly characterized by geotechnical and shear-wave velocity data, the Site Class was estimated by comparing the Site Class estimated for similar geological units. For each Site Class, the corresponding amplification of the ground motion can be determined using the empirical foundation factors included in NBC.

3.3.2 EFFECTS OF GROUND TOPOGRAPHY

Past earthquakes and theoretical models indicate that surface topography can alter the amplitude and frequency content of a ground motion (Geli et al. 1988). In general, ground motions are expected to amplify at the crest of a hill. The degree of amplification depends on the sharpness of the topography (e.g., steeper terrains have a greater amplification potential). The ground motion frequencies that are significantly altered by surface topography are those with wavelengths comparable to the horizontal dimension of the topographic feature. In general, amplification is more pronounced for the horizontal components of ground motion than for the vertical component.

The ground motion amplification effects were incorporated into the damage rate estimations by Isoyama et al. (2000). For example, correction factors ranged from 1.1 for “Disturbed hill” to 3.2 for “Narrow valley” have been considered. In this database, most of the damages were associated with Ductile Iron (DI), Cast Iron (CI) and Polyvinyl Chloride (PVC) pipes as there was limited amount data for steel pipes.

The ground motion amplification maps presented in Appendix C do not account for the variation in ground shaking intensity due to topography. Such amplifications are site-specific and difficult to quantify in a regional-level seismic study. This simplification is considered to have a minor impact to the overall results compared to other uncertainties associated with earthquake source, ground motion propagation and fragility relationships used in estimating the damage.

Electronic copies of the seismic hazard maps will be provided to Metro Vancouver in GIS format in the final submission of this report.

4 PIPE VULNERABILITY

The seismic performance expectations for watermains are outlined in the Metro Vancouver's Seismic Design Criteria (2015), in which the design return period of the seismic event depends on the watermain classification. Class III watermains need to be designed to a 1 in 975-year return period event, while Class IV watermains are designed to a 1 in 2475-year return period event. Available information is insufficient to distinguish the pipe classification, such that all pipes were evaluated for a return period of 1 in 2475-year. The findings are not expected to change significantly even if the 1 in 975-year return period is considered since the extend of liquefaction is approximately similar.

Metro Vancouver's water supply network has about 71 water crossings. As per the Metro Vancouver's Seismic Design Criteria (2015), the major water crossings need to be designed to a return period of 1 in 10,000 years. If designing to such high earthquake intensity is not practical, the crossing may be designed to a 1 in 2475-year return period subject to approval from Metro Vancouver. According to Metro Vancouver (2013), only five critical water crossings out of the major river crossings would meet the requirements of a 1 in 10,000 year earthquake. This includes the Port Mann Water Supply Tunnel constructed in 2017 and Second Narrows Water Supply Tunnel which is under construction. Annacis Water Supply Tunnel, Cambie Richmond Main No. 3 and Haney Main No. 4 are at various stages of design; therefore, not considered in our assessment. The seismic vulnerability assessment has not considered the 1 in 10,000-year return period event since the previous Metro Vancouver study has already identified the extent of the damage to water crossings. Therefore, the seismic vulnerability of water crossings was also assessed based on the 1 in 2475-year return period. In this manner, all transmission mains and facilities were evaluated for the same earthquake intensity.

The approach used for estimating the watermain vulnerability is explained in Section 4.3. For estimating the damages from PGD, previous site-specific studies and pipe-soil interaction analysis were considered. The approach that involves pipe-soil interaction analysis is a novel approach that was developed for this study. Sections 4.1 and 4.2 provide background details related to the selection of input parameters for the pipe-soil interaction analysis. The pipe-soil interaction analysis approach is presented in Section 4.3.3.

4.1 FACTORS IMPACTING SEISMIC VULNERABILITY OF PIPE

The following describes the factors impacting the seismic vulnerability of watermains and how they were incorporated into the vulnerability assessment. As indicated previously, approximately 90 percent of Metro Vancouver's transmission mains consist of large diameter steel pipes (see Figure 2-9). For this reason, the following discussion is primarily focused on factors that impact such pipes.

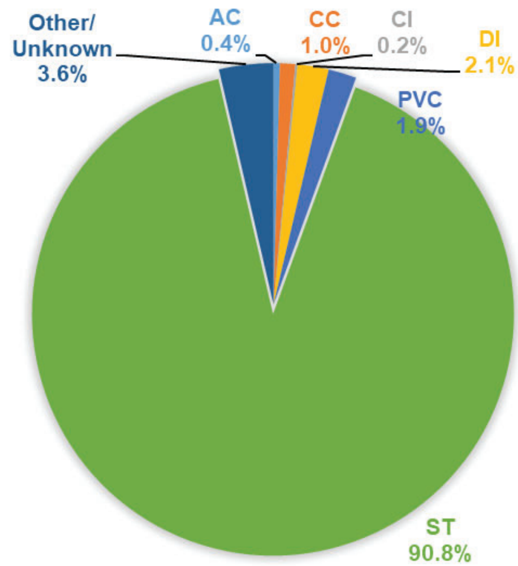

Figure 4-1: **Pipe material types in Metro Vancouver's water supply system**

4.1.1 PIPE MATERIAL AND CONNECTIONS

Pipe material and its connections are critical factors that impact the seismic performance. A brief description of historical damages observed in the drinking water supply systems is provided below to give an indication of the level of damage that should be expected from a major earthquake in the Greater Vancouver area.

4.1.1.1 *STEEL PIPE*

In historical earthquakes, welded steel pipes have shown both relatively high and low damage rates compared to other pipe material types. Although the reasons for vastly different performances are unknown, it is presumed that construction practices, corrosion and geotechnical conditions may have contributed to such performances. Also, the recorded damage rates in these historical events were often controlled by smaller diameter steel pipes (i.e., 150 mm to 200 mm) used in the distribution networks. The data for larger diameter steel watermains is sparse.

According to O'Rourke (1996), during the 1994 Northridge earthquake, nearly 100 repairs (out of 1400) were performed on large diameter transmission and trunk pipes. Out of which, 67 repairs were undertaken in steel trunk pipes, which experienced excessive deformation or rupture at approximately 34 welded slip joints. Post earthquake recovery modeling conducted by the Los Angeles Department of Water and Power (Davis 2015) indicated that it would have taken over five years to bring the water supply system to the same level of service that existed prior to the 1994 Northridge earthquake. A relatively high damage rate was also reported for welded steel pipes during the 1989 Loma Pieta earthquake. However, the post-earthquake inspections have noticed that steel pipes were intentionally used in areas with poor ground conditions to improve the network performance (Ballantyne, 1996). Thus, the higher damage rate is somewhat bias, and it may not be justifiable to compare with other pipe materials.

During the 1995 Kobe earthquake, the repair rate of welded steel pipes was 0.47 failures/km which was considerably lower than the damage rates reported for pipes such as CI, PVC and AC pipes, for which the damage rates ranged from 1.13 to 1.79 failures/km (Ballantyne 1997). The average PGA was in the range of 0.4 to 0.5g which caused widespread liquefaction and destruction. These conditions are somewhat similar to those expected during a major earthquake in the Greater Vancouver area (Kuraoka and Rainer, 1996).

During the 2010 Chile earthquake, Ballantyne (1997) reported that there were 72 breaks or leaks in the large diameter (greater than 500 mm) welded steel pipes. A large-diameter water transmission pipeline located in the epicentral area of the 2011 Tohoku earthquake was damaged at more than 50 locations, mostly due to pulled slip joints.

Between September 2010 and December 2011, Christchurch experienced a sequence of strong earthquakes with PGA predominantly ranging from 0.2 to 0.6g (Bradley and Cubrinovski, 2011). Widespread liquefaction occurred over large areas particularly along the Avon River where liquefaction was often associated by relatively large lateral spreading. The seismic demand and soil conditions are approximately similar to those expected in the Fraser River Delta during a major earthquake. During this event, 77.5 km out of 1511 km water pipes (5.1% of total pipe length) was damaged. Steel pipes suffered the largest damage with a percentage of 8.9%, followed by AC pipes and other pipe materials (6.1% and 6.8%, respectively), while PVC (1.8%) and PE (0.5%) pipes suffered the least damage. The sample size for steel pipes was insufficient for a robust statistical analysis (only 1.8% of total pipe length); therefore, the results should be treated with caution. Furthermore, most of these steel pipes are may have been smaller diameter distribution pipes.

During the 2016 Kumamoto earthquake, steel pipes showed the highest repair ratio among all pipe types with a value of 0.5 failures/km (Ishida et al., 2016), while other pipes such as DI pipes performed extremely well. By segregating into different pipe sizes, the highest repair ratio of 3.7 failures/km was observed in steel pipes with diameters greater than 300 mm.

Metro Vancouver's supply network includes a few older steel pipes with rivetted connections. For example, s.15(1)(l) [redacted] were constructed using rivetted steel pipes. Past earthquakes indicate a considerably higher damage rates for these connections compared to lap welded steel pipes. Single lap-welded joints are not considered as completely earthquake-resistant, but they do offer significantly more earthquake resistance than riveted steel pipes. Since the early 1960's, Metro Vancouver's steel pipes were routinely constructed using double-welded, bell-and-spigot joints. Pipe manufactured after 1970 would likely have improved the seam qualities due to advancement in welding techniques. However, AECOM (2021) reported that in some marine installations, the bell-and-spigot joints were ruptured during the installation or pressure testing. As a result, during 1990's, bell-and-spigots joints have been gradually replaced with butt-welded pipe joints (AECOM, 2021).

4.1.1.2 *DUCTILE IRON*

Available data indicates that DI pipes were used in 10.2 km of watermains, about 2 percent of Metro Vancouver's network. Typical DI pipe joints have limited capacity to accommodate bending and axial movement imposed by ground deformation (Wham and O'Rourke, 2015). However, in areas where ground displacements can occur, specialized "earthquake-resistant" joints can be utilized to minimize pull-out, compression and bending stresses near the joints. This in turn will provide added flexibility to accommodate ground movements up to a certain magnitude.

During the 2016 Kumamoto earthquake, except for a single repair required to address an improper assembly, there were no reported failures in 578 km of DI pipes that utilized earthquake resistant joints (Ishida et al., 2016). The performance of these pipes largely depends on the flexibility and robustness of the joint type rather than the strength of the pipe itself. Some other "restrained" type of joints such as bolt-in mechanical joints and push on joints with tie-rods do not provide the same level of flexibility as some earthquake-resistant joints. However, those restrained joints are likely sufficient to accommodate TGD from seismic shaking and moderate levels of PGD. A greater tolerance to ground displacement should be expected if earthquake resistant connections are utilized. The flexibility and axial tensile/compression capacities offered by these connections vary. At present, there is no unifying standard to decide if a given joint type will meet the minimum flexibility/deflection and load requirements to meet a certain performance requirement. O&M drawings do not indicate if expansion/flexible joints have been used, their locations or manufacturer.

4.1.1.3 *OTHER PIPES*

Besides welded steel, DI and "unknown" pipes in the Metro Vancouver's database, the lengths of each of other pipe types are less than 2 percent (PVC = 1.8%, Concrete = 1.0%, AC = 0.41% and CI = 0.15%). Therefore, their performance is not discussed extensively in this report. In past earthquakes, CI, concrete and AC pipes experienced extensive damage and they are typically considered as brittle with respect to earthquake loading. Similar to steel pipes, the performance of PVC pipes is variable in past earthquakes (e.g., 2010 Christchurch earthquake reported low damage rates while the 1995 Kobe earthquake showed somewhat higher damage rates).

4.1.2 PIPE DIAMETER AND PIPE WALL THICKNESS

For a given pipe material, the axial and bending stiffness of the pipe are influenced by the pipe diameter and pipe wall thickness. Therefore, the pipe diameter and pile wall thickness are recognized as two key parameters that impact a pipe's ability to accommodate ground displacement. Despite this, these parameters are not explicitly considered in empirical fragility relationships referenced in subsequent sections.

Most of the empirical fragility relationships developed prior to 1989 were based on the performance of small diameter (less than 300 mm) CI pipes. Those pipes were the most prevalent materials to have experienced some of the early earthquakes such as the 1906 San Francisco earthquake. Subsequently, earthquakes such as the 1989 Loma Prieta and 1994 Northridge have expanded the database for other pipe materials such as AC, DI and welded steel pipes. Nonetheless, the database mostly contains smaller sized pipes, typically less than 300 mm diameter. Therefore, in this study, an attempt was made to estimate the relative vulnerability of larger diameter pipes such as those encountered in Metro Vancouver's network, using pipe-soil interaction analysis (see Section 4.3.3).

Historical earthquakes have not always showed a consistent relationship between damage rates and pipe diameter. For this reason, pipe diameter is not considered as a variable in some fragility relationships. During the 1994 Northridge earthquake, there was evidence showing a reduction in damage rates with increasing diameter for CI, AC and DI pipes. The 1989 Loma Prieta earthquake also showed a reduction in damage rates for larger diameter welded steel pipe, although there was no clear indication for CI or AC pipes (Eidinger, 1998). Compared to the Northridge earthquake, the correlation was not as strong as for the Loma Prieta earthquake. One explanation given for the observations made during the 1994 Northridge earthquake was that smaller diameter pipes in this earthquake were in the worst soil areas and constructed with low-quality techniques. As a result, the diameter relationship observed in the 1994 Northridge earthquake may not be true for other water systems (ALA, 2001). According to Shirozu et al., (1996), no strong diameter dependency was observed during the 1995 Kobe earthquake where most of the pipe diameters ranged from 100 mm (4") to 300 mm (12"). However, a somewhat lower damage rate was observed when the pipe diameter was 400 mm (16") or larger. There was no distinction made between pipe diameter and soil conditions, so it cannot be ruled out that large diameter pipes were in areas with low shaking intensity or smaller PGD. Also, only a few data points were available for larger diameter pipes to conduct a statistical analysis.

According to Wham et al (2014), the damage rates of the drinking water supply system following the 2016 Kumamoto earthquake were largely independent of the pipe diameter. However, when the damage to pipe barrel is considered (i.e., ignoring the damages to air valves and other components), the smaller diameter service pipes showed a higher damage rate. Other studies such as those conducted by Sato and Myurata (1990) and O'Rourke and Jeon (1999) also reported lower damage rates for large diameter pipes.

Ni., et al. (2018) investigated the pipe diameter effect using numerical modeling. They argued that increasing the pipe diameter can result in higher flexural rigidity, which seems to be beneficial to the pipe, although the flexural strain is a function of the pipe diameter for a specific curvature. Therefore, the allowable curvature of a larger diameter pipe is smaller compared to that for a smaller diameter pipe. They also stated that a smaller diameter pipe tends to perform better in geotechnically problematic areas, as it may be able to tolerate more differential ground movement. Moreover, larger diameter pipes are associated with stiffer/stronger soil springs, such that for the same amount of ground displacement, a larger diameter pipe will experience a larger soil load. For these reasons, Ni et al. (2018) stated that an increase in pipe diameter could result in a higher failure probability regardless of whether a constant pipe wall thickness (t) or a constant diameter to thickness (D/t) ratio is maintained. Despite the conclusions by Ni et al. (2018), there are several possible reasons why larger diameter pipes have shown relatively lower damage rates in some earthquakes:

1. Damages from earthquakes showed evidence of poor weld quality and presence of corrosion. Smaller diameter pipes receive less attention in terms of the weld quality and construction quality control compared to larger diameter watermains.
2. Typically, relatively thin pipe walls are utilized for smaller diameter pipes compared to those used in larger diameter watermains. As a result, if the corrosion rate is assumed to be constant, the smaller diameter pipes are more vulnerable to damage in an earthquake.
3. Most of the failures occur at discontinuities due to stress concentrations. Generally, larger diameter pipes have a fewer number of discontinuities compared to distribution pipes, making them less vulnerable to damage.
4. Often larger diameter watermains are intentionally designed to avoid or mitigate the effects from areas with poor soil conditions.
5. Thicker pipe walls used in large diameter pipes for internal pressure provide greater flexural and axial capacities.

Considering these factors, ALA (2001) recommended empirical factors (K_1 and K_2) to decrease the damage rates by about one-half for lap welded steel pipes with diameters greater than 300 mm. As highlighted earlier, there is lack of data for larger diameter welded steel pipes, such that there is considerable uncertainty associated with these factors.

4.1.3 CORROSION AND AGE

Isenberg (1979), who investigated the role of corrosion in pipeline damage, reported that over one-half of the leaks attributed to the 1971 San Fernando earthquake were related to corrosion. According to Hakala (1981), about 60 percent of pipe breaks and leaks attributed to the 1965 Puget Sound earthquake occurred in steel pipes, galvanised steel mains and service pipes that were impacted by corrosion. Similar observations can also be observed in other earthquakes such as the 1969 Santa Rosa earthquake. Steel pipes have relatively thinner walls than CI or DI pipes; therefore, they are more vulnerable to failure from localized pitting corrosion. Nonetheless, age and corrosion will also accentuate damages in other pipe material such as CI pipes (e.g., Isenberg 1979). For example, in the 1983 Coalinga earthquake, O'Rourke and Ayala (1993) attributed the unusually high damage rate in CI pipes to corrosion.

Corrosion is not expected to be a significant factor if corrosion protection measures have been implemented. AECOM (2021) noted that 27 out of 63 watercrossings have active cathodic protection systems. While the pipes without corrosion protection are more susceptible to failure, the actual likelihood will depend on exposure conditions (e.g., pH, soil resistivity and soil aeration). However, it is not possible to incorporate such details in a regional-scale study. The EQE 1993 study identified pipes impacted by corrosion based on the maintenance records from 1989 through 1991. For these pipes, the damage rates were increased as explained in the following sections.

When collecting data to develop fragility-based relationships, corrosion status is often overlooked. Therefore, the damage rate is an overall rate that includes pipes with different levels of corrosion impact. ALA (2001) stated that damage rates for a small diameter steel pipe in corrosive soil is about 50 percent greater; while it is expected to be about 50 percent less if the pipe is not impacted by corrosion. This infers that the damage rate between a pipe impacted by corrosion is three times greater than a pipe not impacted by corrosion (1.5/0.50 = 3.0). There are some uncertainties related to the applicability of this factor for larger diameter pipes due to lack of damage data for such pipes. Corrosion of a larger diameter pipe may manifest as smaller pin hole leaks; therefore, it may not be as pervasive as it is for a smaller diameter pipe.

Often the age of the pipe strongly correlates to its state of corrosion since the impact of corrosion increases with time. Typically, relatively new steel pipes (i.e., 25 years old or less) will not be significantly impacted by corrosion even if they are in a corrosive environment compared to older steel pipes in the same environment. Eidinger (1998) observed that older pipes have a higher damage rate than newer steel pipes in earthquakes such as the 1987 Whittier Narrows and 1989 Loma Prieta earthquakes.

In the pipe-soil interaction analysis (PSI) discussed in the subsequent sections, the corrosion impact is accounted for using the K_c factor. The K_c selected for lap welded large diameter steel pipes are given in

Table 4-1 below. Note that K_1 factor given in Table 4-1 is consistent with those recommended in ALA (2001) for estimating the TGD induced damage rates. For other pipe types (e.g., welded pipes with rivetted connections, DI and CI pipes), K_1 and K_2 factors are consistent with those given in ALA (2001). Also note that PSI analysis was conducted only for lap welded steel pipes subject to PGD, and that the details are provided in Section 4.3.3.2.

4.1.4 TYPE OF WELD CONNECTION

Welded steel pipes installed prior to mid-1930s were often joined by a single-pass oxyacetylene gas weld. This type of weld has a large heat-affected zone and potentially higher carbon content compared to an arc welded or a multi-pass oxyacetylene weld (ALA, 2001). For example, the 1933 Long Beach earthquake caused more than 50 breaks in high-pressure gas pipelines. Each break was identified at welds that lacked the proper penetration or bond with the pipe body. This aspect is not explicitly considered in ALA (2001) fragility relationships. Nonetheless, ALA (2001) recommends that welded steel pipe with arc-welded joints to be considered as ductile while gas-welded joints to be considered as brittle. For a seismic vulnerability assessment conducted for Seattle, Ballantyne et al. (1990) estimated a repair rate of 0.5 failures/km for welded steel pipes with arc-welded joints in areas impacted by PGD. Comparatively, the damage rate was 2.4 failures/km for welded steel pipes with oxyacetylene welds based on historical damage rates. Based on these observations, it was decided to increase the damage rate by a factor of five for pipes constructed prior to 1935 to account for potential oxyacetylene gas weld connections. This impact of weld connection type on the damage rate is represented by the K_A factor as explained in Section 4.3.3.2. For these pipes, corrosion was not explicitly considered since the factor to account for the weld type would have already included any potential impact from aging/corrosion.

Analysis Approach	Type of Ground Displacement	Corrosion Impact (K_c)	Weld Type (K_A)	
			Before 1935	**After 1935**
Fragility curves by ALA (2001)	TGD	K_1 = 0.15 (default) K_1 = 0.45 (if known to be impact by corrosion)[(b)]	K_1 = 0.75[(a)]	K_1 = 0.15
Pipe-Soil Interaction	PGD	K_c = 1.0 (if catholically protected, regardless of the age)	K_A = 5	K_A = 1
		K_c = 1.0 (constructed before 1935 – see K_A factor).		
		K_c = 3 (constructed after 1935 and more than 50 years old)		
		K_c = 1.5 (between 25 to 50 years old)		
		K_c = 1 (less than 25 years old)		

(a) ALA (2001) does not explicitly provide K_1 factors for large diameter welded steel pipes with oxyacetylene gas welds. Based on observations by Ballantyne et al. (1990), the damage rate was increased by a factor of five.

(b) ALA (2001) does not explicitly provide K_1 factors for large diameter welded steel pipes impacted by corrosion. Based on discussion provided in Section 4.1.3, the damage rate was increased by a factor of three.

Table 4-1: Modification factors selected for large diameter lap welded steel pipes

4.1.5 DISCONTINUITIES IN PIPE NETWORK

Pipeline damages often tend to concentrate at discontinuities such as pipe elbows, tees, in-line valves, reaction blocks and service connections. Such components would create anchor points or rigid locations that promote force/stress concentrations. For example, the liquefaction-induced ground movements during the 1971 San Fernando earthquake caused severe damage to a 1245 mm (49 inch) diameter water pipeline at nine bends and welded joints (O'Rourke and Tawfik, 1983). Locally high stresses can also occur at pipeline connections to adjacent structures (e.g., tanks, buildings and bridges), especially if there is insufficient flexibility to accommodate relative displacements between the pipe and structure. After the 1994 Northridge earthquake, Valencia Water Company decided to install flexible joints for watermains supported by bridges as they experienced significant damage during the earthquake (Abercrombie, 2013).

During the 2016 Kumamoto earthquake, Ishida et al. (2016) reported the extent of damage to air valves. The damage rate at air valves was significant compared to the damages to the pipe itself. For example, the damage rate for air valves was 0.05 failures/km compared to the total damage rate of 0.14 failures/km. Similar damages to air valves have been observed during the 2011 Tohoku earthquake, although explicit mention of such mode of failure was not investigated in earlier earthquakes. These damages have occurred not only in areas near the earthquake epicenter but also in areas far from the epicenter, suggesting that the cause of damage is not related to the strong seismic shaking. Most of the damages were associated with the float valve body. Therefore, it was postulated that abrupt increase of water pressure from sudden closure of valves (triggered from the earthquake) led to this damage. Approximately 15 percent of the damages to air valves were at the flange joint and was assumed to be caused by a combination of aged deterioration and seismic shaking. This form of component vulnerabilities is difficult to model using PSI analysis. However, the fragility relationships are likely to incorporate such damages if these failures are incorporated into the database.

4.1.6 SOIL LOAD ACTING ON PIPE

The soil load acting on the pipe from PGD will depend on several factors as described below. Note that none of these factors are explicitly considered in fragility based relationships, except for the magnitude of PGD.

- **Magnitude and shape of ground displacement:** Inevitably, the magnitude of ground movement is one of the key factors that influences the pipe failure. It is not always possible to quantify the damage due to ground movement even with sophisticated analysis because of the uncertainties related the ground movement estimations and difficulties in accounting for the complex interactions that may occur at pipe bends and other discontinuities. Further details related to the magnitude, direction and shape of ground displacement are discussed in Section 2.4.
- **Pipe burial depth:** In areas susceptible to earthquake-induced ground movement, pipe burial depth (*H*) has a significant influence on the soil load acting on the pipe since the stiffness of the soil spring is a function of the *H/D* ratio. For example, for the same magnitude of ground displacement, a pipe with a shallower burial depth will experience a smaller soil load. O&M drawings do not indicate the burial depths for all watermains.
- **Backfill soil conditions:** The soil load acting on the pipe is impacted by the density and strength of surrounding soils. A denser soil will exert a larger soil load than a looser soil backfill. Details of backfill soil is not readily available and often ignored in the regional-scale seismic studies. As most of Metro Vancouver's transmission mains are located along existing roads and in relatively dense soil, PSI analysis conducted for this study has considered the backfill material to have a friction angle of 40 degrees and a soil density of 19 kN/m^3.

Above details were used to derive soil springs as per the recommendations given in PRCI (2017).

4.1.7 BUILT-IN PIPE STRESSES

Some watermains may be impacted by natural and man-made causes; in particular, the pipe may be impacted by long-term settlement if it is underlain by compressible soils such as peat/organic, silts and clays. For example, it was reported that sections of the s.15(1)(l) have experienced significant settlements resulting from nearby construction activities. This caused failures in the double welded bell and spigot joints. Another example is the s.15(1)(l) which was predicted to undergo a differential settlement of 1 in 300 under non-seismic conditions (AMEC, 2015). Several other pipes located in soft compressible soils are expected to be impacted by ground settlement. Additional loading from earthquakes will exacerbate the damage to an already stressed pipe. Golder (2007) reported that s.15(1)(l) [redacted] sustained a leak at a welded slip joint, near the north bank, where the ground settle by about 300 mm. Such effects are not explicitly considered in empirical fragility-based relationships. However, in the PSI analysis discussed in the following sections, the damage rate was increased by a factor of 1.5 to account for the built-in pipe stresses, if the pipe is known to be located in an area that is prone to long-term settlement s.15(1)(l) [redacted]

4.2 PIPE FAILURE MODES

4.2.1 JOINT CAPACITY

The majority of Metro Vancouver's steel watermains consists of lap welded (bell and spigot) joints. This type of joint is commonly used for watermains since it is a simple and efficient way of connecting large-diameter thin-wall pipe segments. The field welded double or single lap joints are considerably weaker compared to the pipe barrel; thus, the pipe performance is significantly impacted by the performance of joints under compression (local buckling or wrinkling) or tensile loads. The strain capacity of a lap welded joint is influenced by pipe wall thickness, bell geometry and eccentricity between spigot outside and bell inside diameters. The eccentricity of the lap welded joint introduces additional stresses at the joint and reduces its capacity to accommodate the ground movement (O'Rourke and Liu 1999). The associated failure mode for both tension and compression is expected to be the rupture resulting from circumferential cracking of the pipe material at the bell joint or at the bell-to-spigot weld.

4.2.1.1 *COMPRESSION CAPACITY*

Large compression forces could lead to the fracture or crushing of welded slip joints. The compressive strain limit of steel pipes is primarily governed by the D/t ratio. The joint efficiency of a single lap welded joint is approximately 40 to 45 percent (ASME Boiler and Pressure Vessel Code, 1998). The joint efficiency of a double lap welded (bell and spigot) joint is not signifcantly different to a single lap welded joint, and is estimated to be about 55 percent (Brockenbrough, 1990). In comparison, a full penetration butt weld will have a joint efficiency excceeding 95 percent. Most of the research on weld joint capacity have been focused on pure tensile or compression loading modes while the mechanical behavior of those joints under bending has not been investigated thoroughly.

For a double lap welded joint, Metro Vancouver's Seismic Design Criteria (2015) recommends the compression capacity be limited to the following, regardless of the pipe class:

$$\varepsilon_{cp} = 1170 \left(\frac{t}{D}\right)^2 \quad \text{...Eq. 1}$$

For a single lap welded joint, the compression capacity is limited to 40 percent of the yield stress.

Some older pipes are likely fabricated using methods that would not be permitted under current codes. The strain capacity of these pipe joints is highly variable and generally low compared to modern construction practices.

4.2.1.1 *TENSILE CAPACITY*

Typically, the tensile failure of a welded-slip joint occurs at strains greater than 2 percent, which indicates that those joints can sustain a considerable amount of inelastic deformation before failure. The allowable strain of a lap welded joint is smaller than the ultimate strain. For a single lap welded joint, Metro Vancouver's Seismic Design Criteria (2015) recommends the tensile strain of a Class IV pipe be limited to 40 percent of the yield stress while it can be increased to the yield stress for a double lap welded joint. Slightly different allowable strain limits have been considered in some of the previous site-specific assessments. For example, a tensile strain limit of 0.5 percent (i.e., nominal yield strain) was considered for the previous seismic assessments completed for the Cambie-Richmond Main and Haney Main No. 2

and 3. This strain limit is the macroscopic strain calculated from stress analysis, while the localized strain in the vicinity of the weld can be higher.

4.2.2 PIPE CAPACITY

4.2.2.1 *COMPRESSION CAPACITY*

When the compressive strain exceeds a certain threshold, local buckling or wrinkling of the pipe wall will occur. A water pipe may still be able to fulfill its basic functions even after local buckling or wrinkling (Gresnigt. 1986). Significantly high strains are required before wrinkling or buckling impede the passage of in-line inspection or cleaning devices. According to Metro Vancouver's Seismic Design Criteria (2015), the compression strain limit (ε_{cp}) is taken as:

$$\varepsilon_{cp} = 1.76\,\frac{t}{D} \qquad \text{Eq. 2}$$

4.2.2.2 *TENSILE CAPACITY*

For a displacement-controlled failure mode such as PGD, the pipe's tensile strain capacity can exceed the yield strain by a considerable amount. In such situations, the allowable tensile strain for a strain-controlled loading scenario can conservatively be assumed to be equal to about 6 to 8 percent under bi-axial loading conditions. This shows that tensile rupture of the pipe barrel is unlikely to occur because the tensile rupture of the weld joint will occur initially.

4.3 ESTIMATION OF DAMAGE RATES

Damage rates are often defined as the number of pipe repairs per unit length of pipe, e.g., the number of repairs per kilometer (or 1000 feet). The purpose of calculating the damage rate (or number of damages) is to identify the most vulnerable pipe sections in the network and to facilitate post-earthquake recovery planning. For example, a system-wide average of only 0.03 "breaks" per 1,000 feet of pipe (or approximately 0.1 breaks/km) is assigned a serviceability factor of 50 percent in the HAZUS loss estimation tool (FEMA 2020). In addition, the number of damages can be used to determine the resource requirements after an earthquake and for recovery planning. For example, Table 4-2 provides the approximate worker productivity rates given in FEMA (2020) for repairing watermains assuming a 16-hour day shift. The productivity and availability of staff may be different for Metro Vancouver's transmission mains and productivity rates in Table 4-2 are given only as a reference.

Pipe diameter (mm)	# Fixed Breaks per Day per Worker	# Fixed Leaks per Day per Worker
508 (20 inch) or larger	0.20	0.40
304 (12 inch) to 508 (20 inch)	0.50	1.0

Table 4-2: Worker productivity rates for watermains (adopted from FEMA (2020)).

The framework adopted for estimating the number of damages in watermains is shown in Figure 4-2, and details are discussed in the subsequent sections.

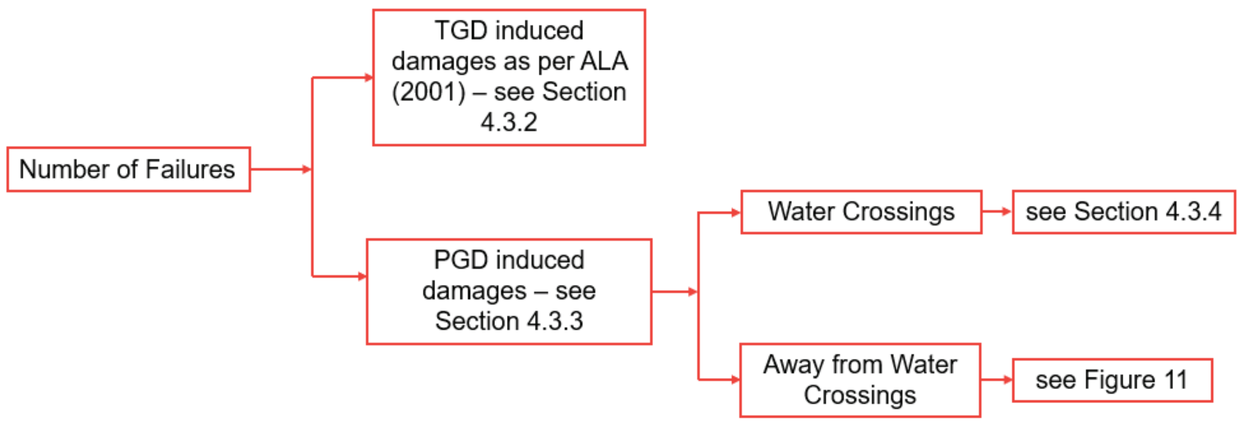

Figure 4-2: **Framework adopted to estimate the pipeline damage rates.**

4.3.1 PIPE FAILURE TYPES

Typically, a "leak" is defined "*as a pipeline failure where a pipeline is losing its product but might continue to operate until the leak is detected*", whereas a "break" is defined as a "*pipeline failure where a pipeline cannot continue to operate".* As expected, a break may lead to more severe consequence than a leak. Despite the differences, leaks and breaks have been collectively called "failures" or "damage" in this report.

Generally, when a pipe is damaged from PGD, the damage is more severe (i.e., more breaks). In contrast, the damages from TGD which are likely to be caused by joint pull-out or crushing at the bell which would lead to more leaks than breaks. The fragility relationships have been derived from pipe repair databases developed from historical earthquakes. Often the repair records include details such as type of repair, location and time, but do not always include sufficient information to determine the severity of the damage.

For damages predicted using TGD, FEMA (2020) recommended that 80 percent of damages be treated as leaks while the remaining 20 percent is considered as breaks. The repair data from the 1949 and 1969 Seattle, 1969 Santa Rosa, 1971 San Fernando Valley, 1983 Coalinga, and 1987 Whittier Narrows earthquakes shows approximately 15 percent of all repairs have been identified as breaks (ALA, 2001), which is somewhat consistent with the recommend split between leaks and breaks in FEMA (2020).

For PGD induced damages, FEMA (2020) suggests considering 80 percent of damages as breaks, while the remaining 20 percent is treated as leaks. There is significant uncertainty related to this breakdown. In the aforementioned earthquakes, approximately 50 percent of repairs were identified as breaks if the damage was caused by PGD (ALA, 2001). The 80/20 split recommended in FEMA (2020) is likely to be influenced by brittle segmented pipes; therefore, likely to overestimate the breaks for continuous welded steel pipes that requires a relatively large strain to cause a break/rupture in the pipe. For this study, 50/50 split was considered for steel pipes with welded slip joints and 80/20 split recommended in FEMA (2020) was adopted for other brittle and segmented pipes.

4.3.2 TRANSIENT GROUND DISPLACEMENT DAMAGE

The damage from seismic shaking can be estimated using a range of relationships proposed by different researchers. In the literature, these relationships range from Modified Mercalli intensity, PGA, PGV, PGD, Arias Intensity, Spectral Acceleration, Spectral Intensity, maximum ground strain, and the composite parameters that include PGV^2 /PGA. O'Rourke et al. (1998) identified PGV as the best indicator for estimating damage rates induced from TGD, and proposed a fragility relationship for steel, CI, DI, and AC pipelines. Similarly, Eidinger et al. (1995) proposed another set of fragility curves in terms of PGV but included empirical factors to account for the pipe material, joint type and soil corrosiveness. Another PGV based fragility relationship was proposed by O'Rourke and Ayala (1993) which was later adopted into the loss assessment methodology HAZUS-MH (FEMA, 2020). According to some studies (e.g., Tromans 2004; O'Rourke 1999) the fragility curves by O'Rourke and Ayala (1993) tend to overestimate the damage. It is important to highlight that these key developments in fragility relationships occurred after 1993 and were not available for EQE's seismic vulnerability assessment.

American Lifeline Alliance (ALA, 2001) conducted a detailed assessment of the existing data set and developed a set of relationships to compute the damage rates. The fragility relationship was based on PGV. The database included 81 data points mainly from brittle pipes such as AC, CI, concrete and prestressed concrete cylinder pipes. Only 16 percent of data points used for the development of ALA (2001) fragility curves were based on steel pipes. Furthermore, damage data was largely based on segmented pipelines and there is limited evidence of damage to continuous pipes from seismic wave propagation. Although some researchers have suggested continuous pipes are not impacted by seismic wave propagation (O'Rourke 2009), others have documented a few damage cases characterized by special circumstances. As highlighted earlier for the 2016 Kumamoto earthquake, failures such as those occurred at air valves are unavoidable even with the use of continuous and ductile pipes. These failures are captured using the empirical fragility relationships, although the pipe itself is unharmed.

The fragility relationship given in ALA (2001) for TGD is given below:

$$RR_{(TGD)} = \mathrm{K}_1 \times 0.00187 \times \mathrm{PGV} \quad \text{...Eq. 3}$$

Where $RR_{(TGD)}$ is the repair rate for pipe from TGD (repairs/1000 feet) and PGV is given in inches/second.

Similar to the fragility relationships developed by Eidinger et al. (1995) and Eidinger (1998), ALA (2001) includes factors to account for the pipe material, joint type, and corrosion. These were incorporated in the fragility relationship using a modification factor, K_1 (see Section 4.3.3 for additional details).

Typically, the fragility curves are associated with a relatively large uncertainty. For example, the median curve of ALA (2001) is associated with a lognormal standard deviation of 0.74. This implies that the median value should be factored by 1.65 and 0.60 to obtain the 25 and 75 percentiles. Despite the limitations, the fragility relationship proposed by ALA (2001) were adopted in this assessment to estimate the damages caused by TGD since it was derived from a larger database and contains some of the key parameters that would impact the pipe performance.

4.3.3 PERMANENT GROUND DISPLACEMENT DAMAGE

Pipelines subject to PGD have suffered severe damages during past earthquakes (Eckel 1967, O'Rourke and Tawfik 1983, and O'Rourke et al. 1989). An overlay of Metro Vancouver's water transmission network indicated that approximately 138 km out of 498 km of pipes are in areas likely to be impacted by PGD triggered from soil liquefaction or landslides. The damages from PGD can be more severe than those caused by TGD. Also considering the relatively large portion of pipes located in seismically vulnerable areas, an accurate estimation of pipe performance due to PGD is more critical than estimating damages caused by TGD. For comparison purposes, the 1993 EQE study estimated 30 failures due to PGD while only one (1.3 to be exact) failure was assigned to TGD.

At the time of the 1993 EQE study, the only approach available to estimate PGD induced damages was the generic fragility relationships that related ground displacement to the number of repairs. As explained earlier, these generic relationships do not account for pipe diameter, pipe wall thickness, material strength, burial depth, corrosion, soil parameters or the direction of ground displacement relative to the pipe axis.

4.3.3.1 *FRAMEWORK FOR ESTIMATING DAMAGE RATES FROM PGD*

As highlighted earlier, several site-specific studies are available for most of the key watermains. Therefore, it is paramount to maximize this information when estimating the extent of damage. With this intent, the following framework was adopted (Figure 4-3) for estimating the pipe damage rates due to PGD.

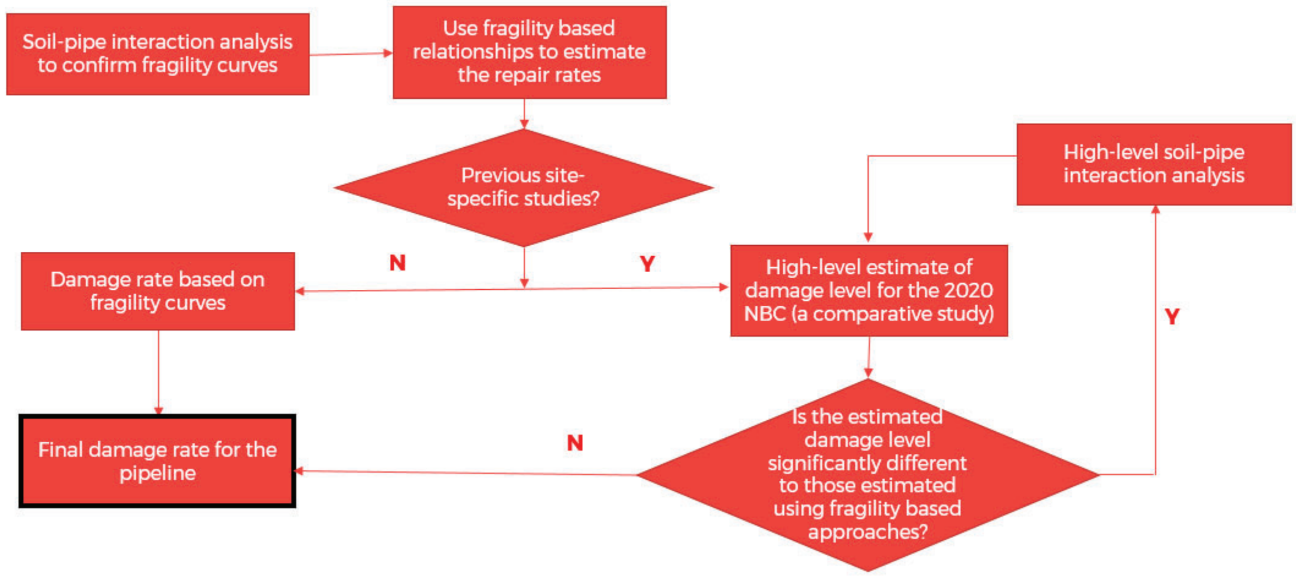

Figure 4-3: **Framework adopted to estimate the pipeline damage rates due to PGD.**

A key advantage of this approach is to utilize PSI analysis to estimate damage rates because the existing fragility curves were developed from a smaller database of large diameter welded steel pipes (see discussion in Section 4.3.2). Despite the benefits of PSI analysis, it is not always possible to rely entirely on a site-specific analysis to estimate the level of damage. For example, site-specific PSI analysis is not typically conducted for complex pipe configurations or at every discontinuity to assess the vulnerability at these locations. Although the damage occurring at these locations may be minor (i.e., a leak) compared to an area with a well-defined ground failure, such failures cannot be ignored. For this reason, if the damage rates predicted from site-specific analysis and fragility-based curves showed significant differences, further scrutiny was given to identify the root-causes and select the most appropriate damage rate. Some of these scenarios are discussed later in this report.

4.3.3.2 *DAMAGE RATES USING PIPE-SOIL INTERACTION (PSI)*

The method adopted to estimate the damage rates arising from PGD can be subdivided into two categories due to the differences in analytical approaches (see Figure 4-4):

- Horizontal and vertical ground displacement occurring perpendicular to the pipe axis
- Horizontal ground displacements occurring along the pipe axis

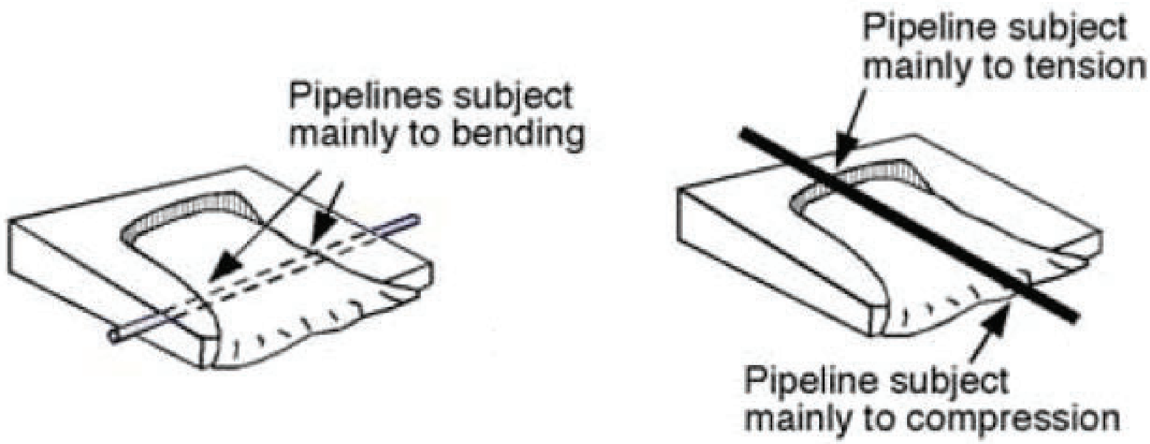

Figure 4-4: **Ground movements occurring perpendicular to the pipe axis (left) and those occurring along the pipe axis (right).**

LATERAL AND VERTICAL GROUND DISPLACEMENTS

A PSI analysis will provide an estimation of the pipe demands (bending moments, shear forces and axial forces) but not the damage or repair rates. Therefore, for pipes subject to PGD in vertical and horizontal directions, the following relationship was developed to convert the estimated pipe demands to a repair rate (RR_{PGD}), in repairs per kilometer.

$$RR_{PGD} = K_A \times K_s \times K_c \times \frac{1000}{L_1} \times \left[\sum_i P_{c,i}\left(\frac{d_i}{L_1}\right)\left(\frac{d_i}{L_2}\right) + \frac{d_i}{L_1} \times P_{p,i}\right] \qquad \text{……….……….Eq. 4}$$

Where:

L_1 = Pipe length in question (m). This was selected as 300 m if the pipe was located 300 m from a waterbody. As indicated in Section 2.3.1.2, the width of the lateral spreading zone is 100 m (median value). If so, approximately 33 percent of the pipe is assumed to be impacted by lateral spreading. This is considered reasonable and consistent with previous regional seismic studies conducted for FortisBC (Honegger et al., 2014). For example, in the 1994 FortisBC study, s.15(1)(l) [redacted] were assumed to have a 34 percent chance of experiencing lateral spread displacement. Away from the waterbody, this length was increased to 400 m, to account for the more uniform ground displacements that may occur at these locations. In other words, the ground settlement will impact 25 percent of the pipe length.

L_2 = Pipe length between two joints (m). The pipe segment length typically ranges from 6 m (20 feet) to 12 m (40 feet). The intent of including this term is to estimate the probability of encountering a weld within the zone of high bending strain (=d_i/L_2). Typically, this information is not provided in O&M drawings. Thus, the PSI analysis was based on a pipe segment length of 6 m, which is conservative. A length 12.2 m was considered for the Cambie-Richmond main since Golder (2003) stated that pipe installation involved field welding of 12.2 to 18.3 m long sections.

K_c = Corrosion factor. As discussed in Section 4.1.3, factors given in Table 4-1 were considered to account for corrosion.

K_s = Soil settlement factor to account for the built-in stresses in pipes impacted by long-term settlement of soil. As stated in Section 4.1.6, a K_s factor of 1.5 was considered for watermains located in areas that have undergone settlement.

K_A = Factor to account for the welding type. As discussed in Section 4.1.3, the damage rate was increased by applying a K_A factor of five to account for the potential weak welds constructed prior to 1935 using a single-pass oxyacetylene gas weld. This was a simplification in the analysis method instead of adjusting the failure probability of the weld connection. For these pipes, K_c factor was one since K_A is already assumed to contain the corrosion impact.

$P_{p,i}$= Probability of failure of the pipe itself for a given range of strain. The failure probabilities selected for the pipe are given in Table 4-3. Although there is some subjectivity involved in the selection of these failure probabilities, these values are consistent with those selected for the FortisBC study by D.G. Honegger Consulting (Honegger et al., 2014). For pipes involving bending, the strain limit was selected to represent the compression loading mode which is more severe than tension.

Bending Strain (Compression)	Probability of Failure
Less than 0.5%	0%
0.5% to 1.76 t/D	10%
1.76 t/D to 2 times 1.76 t/D	50%
Greater than 2 times 1.76 t/D	100%

Table 4-3: Failure probability of a pipe body under bending (compression)

$P_{c,i}$ = Probability of failure of a lap welded connection for a given range of strain. The failure probabilities of weld slip joints were selected after considering the relative vulnerabilities of the pipe and weld connection (see additional discussions in Section 4.2 for joint efficiencies). These failure probabilities are applicable to both single and double lap welded joints since their performance is not significantly different. For pipes experiencing bending, the capacity of the weld will be governed by compression; therefore, the failure probabilities are given only for this mode of failure.

Bending Strain (Compression) at welds	Probability of Failure
Less than min ($0.4\sigma_y$, 1170 $(t/D)^2$)	0%
Min ($0.4\sigma_y$, 1170 $(t/D)^2$) to σ_y	20%
σ_y to 2 times εy	50%
Greater than 2 times εy	100%

Table 4-4: Failure probability of a single or double slip weld joint under bending (compression)

d_i = Pipe length that falls within a certain range of pipe strain (m).

In this analysis, the non-linear stress-strain behaviour of the pipe material was considered as it provides a more realistic response of the pipe when the strain exceeds the yield strain of the pipe. As an example, Figure 4-5 shows the bending moment – curvature and bending moment- bending stiffness relationships derived for a 914 mm x 6.4 mm steel pipe with a yield strength of 248 MPa.

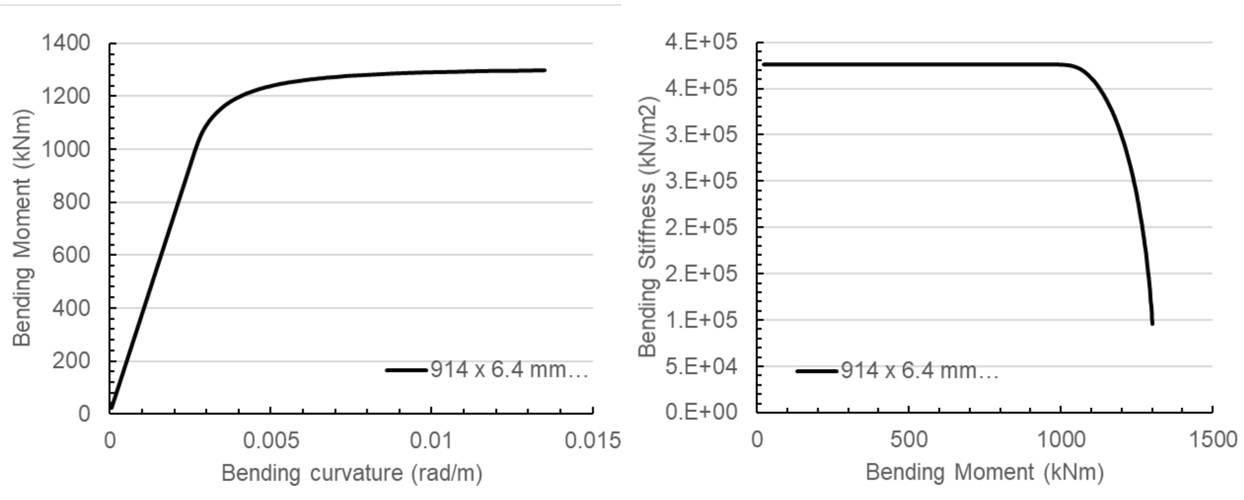

Figure 4-5: **Bending moment, curvature and bending stiffness considered for a 914 mm x 6.4 mm steel pipe.**

Both cosine and abrupt ground deformation shapes were considered in the PSI analysis. The largest damage rate out of the two ground movement profiles was selected for subsequent calculations. Except for certain larger diameter pipes subject to relatively smaller PGD, abrupt ground displacements were found to provide the highest damage rates.

As an example, Figure 4-6 shows the pipe deformations and pipe strains estimated for a 914 mm x 6.4 mm watermain buried at a depth of 2 m. The horizontal ground displacement is 0.5 m, and the width of PGD was considered as 100 m.

The red, orange and green areas represent the different strain thresholds considered in Table 4-4 for lap welded joints. In this case, the pipe strains induced from the abrupt ground deformation profile are more severe than the cosine ground deformation profile. The higher strains are experienced near the slide boundaries, while the cosine shaped PGD profile indicates strain less than 0.05 percent for the entire pipe length. For the abrupt ground deformation profile, 16.8 m of pipe length falls within 0.05 to 0.12 percent strain range, while 19.2 m of pipe length is within 0.12 to 0.24 percent strain range. As explained above, these lengths represent d_1 and d_2 in Equation 5 for estimating RR_{PGD}.

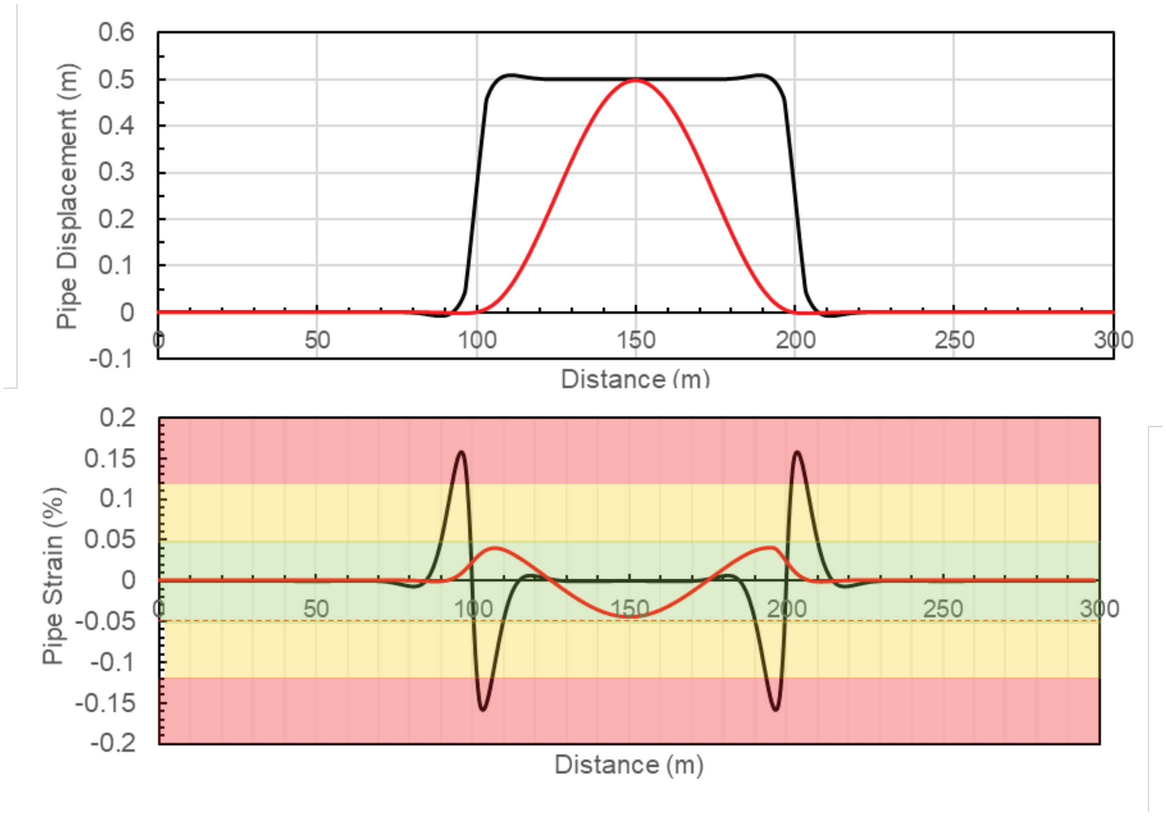

Figure 4-6: **Estimated pipe deformation shapes and bending strains for 0.5 m of lateral spreading (914 mm x 6.4 mm steel pipe)**

Figure 4-7 shows the results when 0.5 m of ground displacement occurs in the vertical direction.

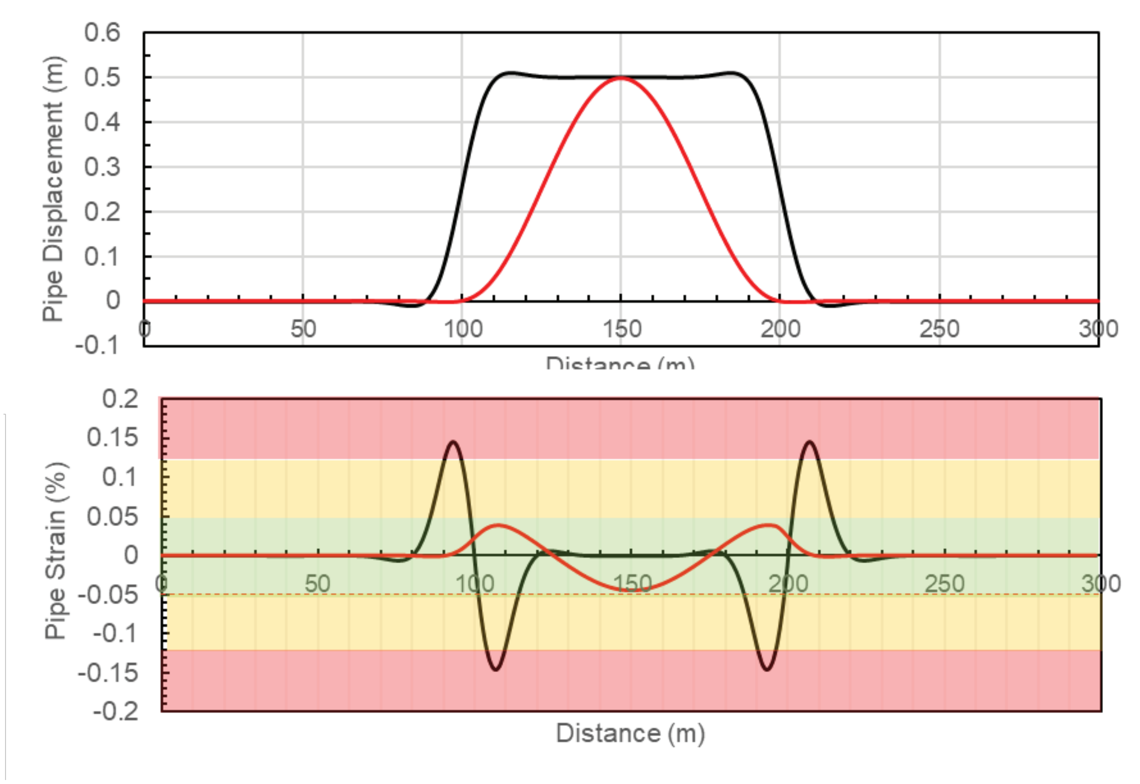

Figure 4-7: **Estimated pipe deformation shapes and bending strains for 0.5 m of vertical settlement (914 mm x 6.4 mm steel pipe)**

For comparative purposes, the damage rates can also be estimated using the following empirical relationships proposed in ALA (2001):

$$RR_{(PGD)} = \mathrm{K}_2 \times 1.06 \times \mathrm{PGD}^{0.319} \quad \text{......Eq. 5}$$

Where $DR_{(RGD)}$ is the repair rate for pipe from PGD (repairs/1000 feet) and PGD is in inches. Similar to the K_1 factor considered for TGD, K_2 is a factor that account for the pipe material type, connection and pipe diameter. The same factors given in ALA (2001) were adopted without any modifications.

COMPARISON OF DAMAGE RATES ESTIMATED USING FRAGILITY CURVES AND PSI ANALYSIS

For the example given in the previous section, the estimated damage rate is 0.50 failures/km if the ground displacement occurs horizontally and perpendicular to the pipe axis (i.e., lateral loading scenario). Comparatively, the fragility relationship given in ALA (2001) estimates a damage rate of 1.35 failures/km.

If the ground displacement occurs in the vertical direction, the estimated repair rate is 0.79 failures/km, which is greater than that considered in the horizontal direction (assuming a L_1 of 300 m). Fragility formulations, including ALA (2001) do not consider the direction of loading; therefore, the same damage rate is estimated for PGD occurring in the vertical direction.

The differences in damage rates estimated in the vertical and lateral directions can be explained by comparing the bending strain distributions (see Figure 4-8). The stiffer soil spring in the lateral direction increases will cause strain concentration within a narrow segment of the pipe although a slightly larger peak strain is induced in the pipe. In contrast, the relatively softer soil spring in the vertical direction will distribute the bending strain over a longer pipe length (i.e., more pipe length in the orange zone), although the maximum flexural strain is smaller. The results cannot be generalized for all displacement ranges and pipe sizes. Above observation is mostly true for relatively smaller ground displacements. As the ground magnitude of ground displacement increases, the lateral soil loading is expected to result in a higher failure rate than in the vertical direction.

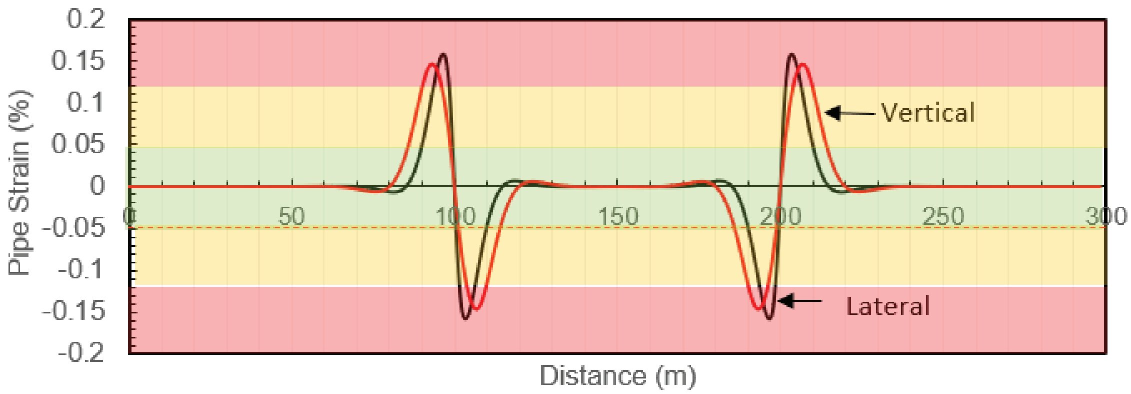

Figure 4-8: **Bending strains measured for soil loading occurring in vertical and lateral directions (914 mm x 6.4 mm diameter steel pipe.**

The analysis described above was repeated for several pipe diameters and different PGD values in the horizontal and vertical directions, and the results are presented in Figures 4-9 and 4-10, respectively. Note that D/t and H/D ratios are not similar in these scenarios, but they represent the most likely conditions observed in Metro Vancouver's transmission mains. The pipe diameters, pipe wall thicknesses and pipe burial depths considered in this assessment are summarized in Table 4-5. For comparison purposes, the repair rates estimated using ALA (2001) and FEMA (2020) are also shown. The comparison does not consider any corrosion, age and long-term settlement impacts.

Pipe Diameter, D (mm)	457	610	762	914
Pipe Wall Thickness, t (mm)	6.4	6.4	6.4	6.4
Pipe Burial Depth, H (m)	1.5	1.5	1.5	2
D/t	71	95	120	143
H/D	3.3	2.5	2.0	2.2

Table 4-5: Pipe diameters, pipe wall thicknesses and burial depths considered for pipe-soil interaction analysis

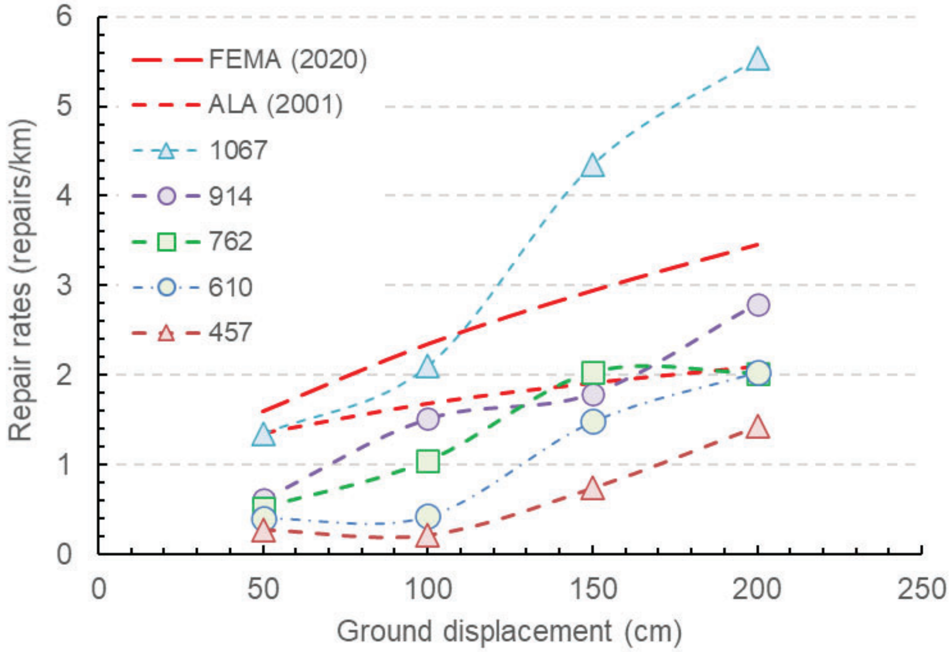

Figure 4-9: **Damage rates estimated from the proposed pipe-soil interaction analysis and fragility curves provided in ALA (2001) and FEMA (2020) for different pipe diameters and PGD values in the horizontal direction**

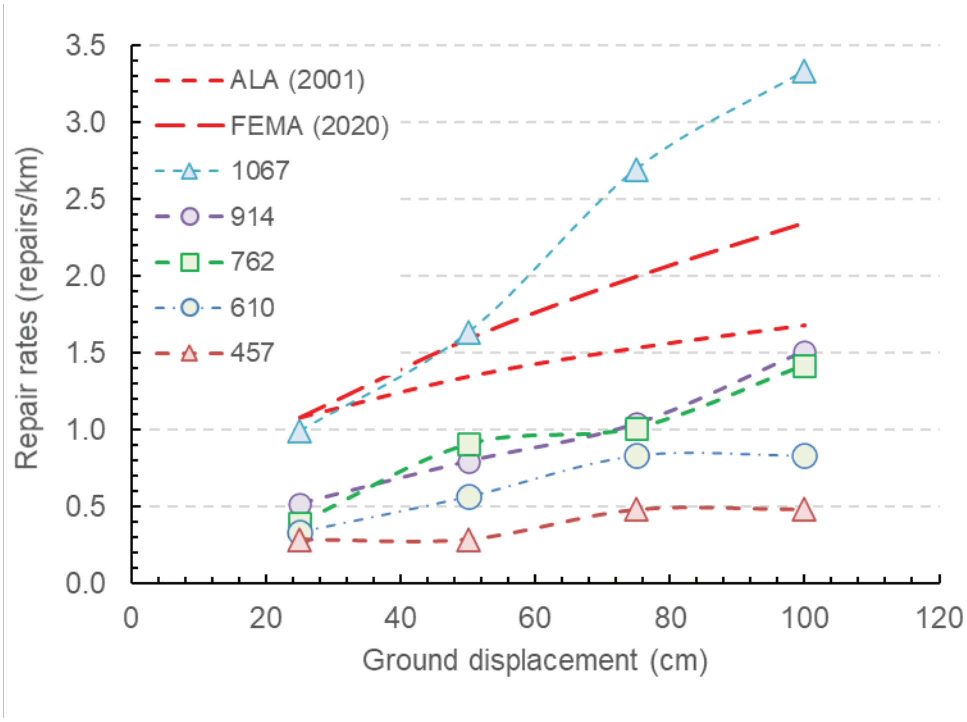

Figure 4-10: **Damage rates estimated from the proposed pipe-soil interaction analysis and fragility curves provided in ALA (2001) and FEMA (2020) for different pipe diameters and PGD values in the vertical direction**

A summary of the key observations is given below:

- In general, the damage rates estimated using the ALA (2001) fragility curves are higher than those estimated using PSI analysis up to about 1.5 m of lateral spreading. Beyond this displacement, the damage rates estimated for larger diameter pipes using ALA (2001) are somewhat less than those estimated using PSI, although the difference is not significant considering other uncertainties associated with this assessment. The damage rates estimated using FEMA (2020) are considerably higher than those estimated from PSI and ALA (2001), especially at larger PGD values.
- For diameters exceeding 1m, a larger damage rate is predicted using PSI than those estimated using empirical fragility relationships. The difference increases as the ground displacement increases.
- ALA (2001) damage rates are somewhat insensitive to the PGD value. For example, an order of magnitude increase in PGD only produces a factor of roughly two to three times the increase in failures. This may be attributed to the fact that once yielding occurs at a certain pipe location, the remaining sections are protected from damage since most of the deformation is localized to this location. PSI analyses indicate a somewhat higher sensitivity to the magnitude of PGD but it greatly depends on the pipe diameter and other factors.
- Besides some exceptions, PSI analysis indicates that damage rate increases with the pipe diameter, consistent with the explanation given by Ni et al. (2018). However, it should be noted that D/t and H/D ratios are not the same for each case; therefore, PSI results may not provide a true reflection of the pipe diameter impact if D/t and H/D ratios remains the same. However, it is considered more pragmatic to consider the actual conditions observed in Metro Vancouver's transmission mains instead of carrying out such analysis for hypothetical scenarios with academic interest. As stated earlier, better construction and quality control measures adopted for larger diameter pipes are not reflected in the PSI analysis, although they can have a significant impact on the relative vulnerability.
- A key information extracted from the PSI analysis is the relative vulnerability of the pipe barrel and weld connection. According to the above formulation, the number of damages expected in the pipe and weld can be calculated separately. Such comparisons cannot be made using the fragility relationships. PSI analysis indicates that about 90 to 95 percent of failures would occur at weld connections. This was in the order of 60 percent during the 1994 Northridge earthquake and 80 percent during the 1995 Kobe earthquake. During the 2016 Kumamoto earthquake, in pipes larger than 600 mm, only about 10 percent of the damages occurred in the pipe barrel. This indicates that PSI predictions are somewhat consistent with observations from actual earthquakes, implying that relative failure probabilities selected for pipe and weld connections are reasonable.
- Some pipes may already be under stress from other natural and man-made causes. For example, it was reported that sections of the s.15(1)(l) [redacted] have experienced significant settlements resulting from nearby construction activities. The magnitude of settlement was sufficient to cause failures in the double lap welded bell and spigot joints. Another example is the s.15(1)(l) [redacted] which was predicted undergo differential settlement of 1 in 300 under non-seismic conditions (AMEC, 2015). Several other pipes located in soft compressible soils are expected to have been impacted by ground settlement; therefore, more prone to damage from additional seismically induce ground displacements. Such effects are not explicitly considered in fragility based curves. However, in PSI analysis, damage rates for these pipes were increased to account for assumed built-in stresses of the pipe due to ongoing settlement (e.g., s.15(1)(l) [redacted] were increased by a factor of 1.5).
- The direction of loading was not considered in empirical fragility formulations and may overestimate the damage rate in the vertical direction. There are about 80 km of pipes located in s.15(1)(l) [redacted] areas that are likely to be impacted by post-seismic settlement. Therefore, a more detailed investigation of this aspect is warranted considering the impact from these pipes to the total failure count. For these pipes, the damages rates were estimated using the PSI approach described in this section. In general, the ground displacement is estimated to range from about 300 mm to 500 mm although some site-specific studies have indicated somewhat larger settlements. The most significant factor that impacted the damage rate is the areal distribution that is represented by $L_{1.}$ As stated previously $L_{1.}$ was increased to 400 m. K_c factors selected for each of

these pipes based on the criteria given in Table 4-1. Figure 4-11 shows the different damage rates estimated for some of the key watermains in these areas.

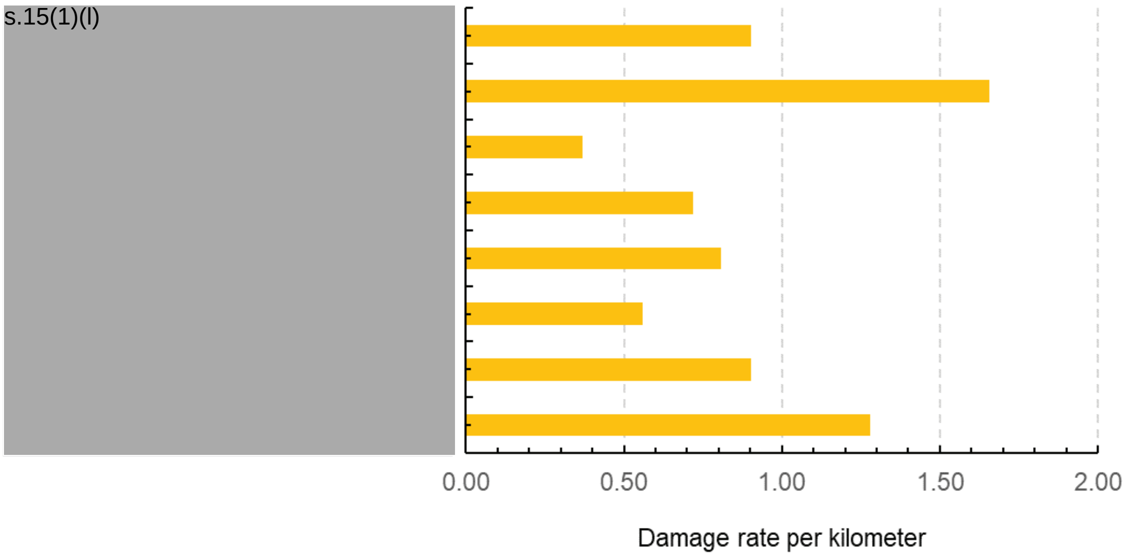

Figure 4-11: **Damage rates estimated from PSI and those selected for estimating the damage rates for different watermains subject to post-seismic settlement..**

- The lowest damage rate was estimated for the s.15(1)(l) [redacted] constructed in 2017. Large damage rates were estimated for the older mains including [redacted] s.15(1) [redacted] The damage rate of s.15(1)(l) [redacted] is exacerbated by concerns related to ongoing settlements in the area.

	Diameter (mm)	Pipe Wall Thickness (mm)	Assumed Pipe Burial Depth (m)	Year of Construction
s.15(1)(l) [redacted]	450	6.4	2.5	1949
[redacted]	1372	9.5	4	1979
[redacted]	660	6.4	3.5	1966
[redacted]	1219	9.5	3	1990
[redacted]	1067	9.5	3	1980
[redacted]	914	6.4	3	2017
[redacted]	838	7.9	2	1926
[redacted]	1372	9.5	4	1990

Table 4-6: Pipe diameters, pipe wall thicknesses and burial depths considered for seven watermain located in liquefiable soils

- Compared to post-seismic settlement, there are only a couple of pipes that are impacted by ground displacements occurring horizontally and perpendicular to the pipe axis s.15(1)(l) [redacted] For those pipes, the damage rates were estimate using PSI analysis, as described in the preceding sections. Further details are not discussed herein for brevity and it has a relatively small impact to the overall failure count. Ground displacements do not occur entirely in the lateral direction and some pipe sections are likely to experience additional movements along the pipe (axial movement) and in the vertical direction. For this regional scale study, loading from one predominant direction was considered.

- Most of the pipes in the North Shore traverse through a number of gully infills. At these locations, the seismic vulnerability of the pipe depends on the span of the infill and magnitude of ground displacement. For example, BGC (2003) concluded that a pipe could separate by rotation at their slip joints if the ground movement was more than about 0.25 m for the short span or more than about 1 m for a longer span. The damage would be sufficient to cause significant leakage. Similar site-specific studies are available only for sections of s.15(1)(l) to identify these gully infills and their vulnerabilities. In the damage rate calculation, a leak was assigned to gully infills explicitly identified in the site-specific analysis, while others were assumed to be captured in the generic fragility relationships for TGD.

AXIAL GROUND MOVEMENT

A different approach is required to estimate the vulnerability of pipes under axial loading compared to the approach considered in the horizontal and vertical direction. The loading mechanism of a pipe subject to lateral spreading (or flow slide) along their pipe axis is schematically shown in Figure 4-12.

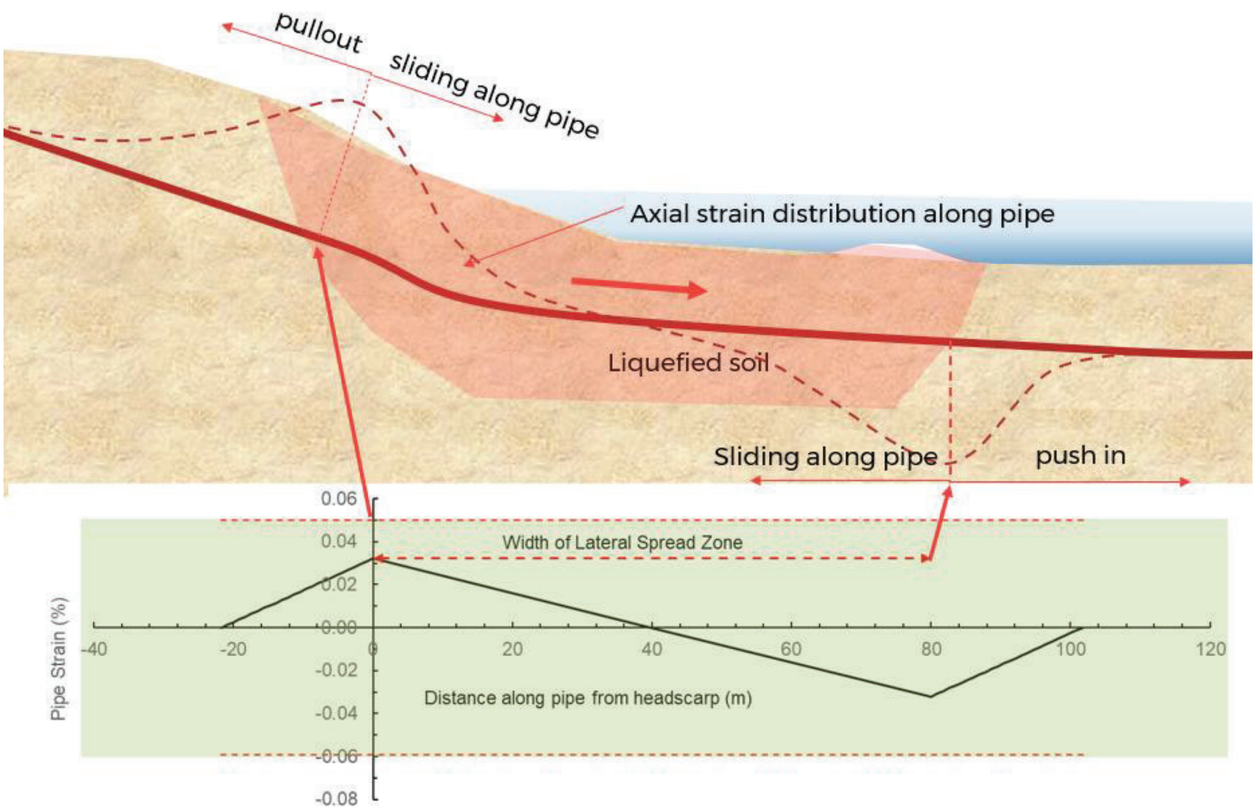

Figure 4-12: **A schematic representation of a pipe subject to lateral spreading along the pipe axis and estimated axial strains along the pipe.**

In this form of loading, the pipe will experience frictional forces as a result of ground movement. Within the slide area, soil will slide over the pipe. Note that only a small relative displacement between the pipe and soil is required to develop the full frictional force (i.e., about 2 to 5 mm). Therefore, the axial strain (or force) in the pipe is largely independent of the magnitude of the movement and depends mainly on the length of pipe exposed to the ground movement. The friction forces arising from ground movement is estimated using the following equation as recommended in ALA (2001) and PRCI (2017).

$$F = 0.5\pi DHL\bar{\gamma}(1 + K_0)\tan\delta \qquad \text{..........................Eq. 6.}$$

Where D is the pipe diameter, H is the burial depth of the pipe, $\bar{\gamma}$ is the density of the surrounding soil, K_o is the lateral earth pressure coefficient at rest, δ is the interface friction angle between pipe and soil, and L is the pipe length impacted by lateral spreading.

The largest axial strains/forces are expected near the headscarp or toe of the slide (see Figure 4-12). While the pipe sections near the headscarp will experience the maximum tensile strains/forces, those at the toe of the slide will experience the peak compressive strains/forces. Outside the slide zone, the axial force in the pipe does not follow the mechanism represented by Equation 6. In those regions, the mechanism will be similar to a pullout (push in) of a pipe from (or into) a stationary soil mass. For these loading scenarios, a closed form solution was developed by Weerasekara and Rahman (2019) for steel pipes experiencing such loading conditions, and details are not given herein for brevity. Once the total axial (friction) force from Equation 6 is determined, the distribution of axial force/strain outside the headscarp and toe of the slide can be determined using the equations proposed by Weerasekara and Rahman (2019). Figure 4-12 shows the axial strain distribution for an 80 m long pipe segment impacted by lateral spreading. The example considers a 914 mm x 6.5 mm diameter steel pipe buried 3 m from the ground surface.

Typically, axial loading is not a concern for a continuous pipe unless a relatively long pipe section is exposed to lateral spreading. Generally, it is unlikely for a pipe to experience ground displacements entirely along the pipe axis for such long distances without any displacements occurring in the lateral or vertical directions. This is evident in some of the 2D numerical modeling conducted for river crossings (e.g., Pitt River crossing of Haney Main No. 2 and 3). The vertical and horizontal ground displacement components tend to induce larger bending strains than the axial strains arising from the axial loading component, and sometimes dictate the likely damage location.

Although the axial loading scenario is discussed above for completeness, there is limited applicability of this formulation for this study. As discussed in the following section, site-specific assessments completed by others for river crossings were relied upon to estimate the vulnerability of these pipe sections.

4.3.4 WATER CROSSINGS

There are at least 71 waterbody crossings in Metro Vancouver's transmission network. Considering their seismic vulnerabilities and consequences, they can be subdivided into the following four categories:

CATEGORY 1: MAJOR RIVER CROSSINGS – BURIED PIPES

This category includes major river crossings such as Fraser River (North and South Arms), Pitt River, False Creek and Burrard Inlet. We have also included Deas Slough, Annieville and Annacis Channels into this category considering the conditions of these crossings. Widespread liquefaction is expected at these crossings with lateral spreading (or flow slides) extending to about 300 m from the riverbanks. Any damages to the transmission main are difficult repair due to the location of the pipe, water depth, width of the crossing and water flow condition. Several site-specific assessments have been already conducted and some of those reports have been provided to WSP (e.g., Annacis Main No. 2 and 3, Second Narrows Water Tunnel, Port Mann Crossing, Tilbury Main, Cambie-Richmond Main and Haney Main No. 2 and 3). Except for Port Mann and Second Narrows Water Tunnels designed to withstand a 1 in 10,000-year earthquake, these analyses have confirmed the vulnerability of other crossings even for a 1 in 2475-year event. According to these analyses, at least two to three breaks in the transmission main can be expected. For example, the analysis completed for the s.15(1)(l) has found that the pipe is highly susceptible to compression buckling beneath the river besides those failures near the riverbanks. Additional breaks may occur if bends or discontinuities exist within 300 m from the riverbank, where relatively large ground displacements would occur. Interestingly, approximately similar number of failures are predicted using empirical fragility curves given in ALA (2001). However, for this seismic assessment, the number of failures were adopted from existing site-specific studies or inferred from similar studies if such information is not available for a particular crossing.

CATEGORY 2: SHALLOW RIVER, STREAM AND CREEK CROSSINGS – BURIED PIPES

This category includes somewhat shallow rivers such as Coquitlam, Seymour and Brunette Rivers, streams and creeks. At these locations, the ground displacement may occur at the riverbank and will extend to the pipe level. However, the magnitude of displacement is much smaller compared to major river crossings discussed under Category 1. For example, the 2D FLAC analysis conducted by Golder (2009) estimated PGD to range from 100 to 200 mm for s.15(1)(l) [redacted]. This was estimated for the 1 in 2475-year seismic event as per the 2005 NBC. A slightly higher ground displacement may be estimated from the 2020 NBC; however, the magnitude expected to be considerably smaller than those estimated for Category 1 crossings. The estimated ground displacement for the 1 in 10,000-year event was about 400 to 500 mm. Golder's analysis indicated that both mains are deficient under the 1 in 10,000-year event. The most likely locations of pipeline failure are the mitre bends and overbends of the pipeline (Golder, 2009). The pipes were also found to be deficient under the 1 in 2475-year event, although Haney Main No. 3 was considered marginally unacceptable with a maximum compressive strain closer to the 0.5 percent limit. Although site-specific analyses are not available for other similar crossings, we consider the results of s.15(1)(l) [redacted] are somewhat indicative of these crossings. Accordingly, we have considered at least two leaks would occur near these crossings during a 1 in 2475-year event. Compared to Category 1 crossings, it is unlikely that ground displacements would cause a pipe failure at the middle of the river due to compression buckling.

CATEGORY 3: SHALLOW CREEKS AND SLOUGH – BURIED PIPES

There are several shallow creek/slough crossings in the Metro Vancouver's supply network where the impact to the pipe is less severe than those expected for Category 1 and 2 crossings. At these locations, the pipe is located below the bottom of the creek/slough and it is unlikely to be impacted by the shallow ground failures that may occur at the banks. However, the watermain may still experience post-seismic settlement if the soil below the pipe invert is liquefiable. In this study, no breaks or leaks have been considered for this type of crossings. At present, there are no site-specific studies conducted to confirm the pipe performance in similar crossings.

CATEGORY 4: BRIDGE CROSSINGS

If the pipe is supported on a bridge, seismic vulnerability of the pipe is directly linked to the performance of the bridge structure. Recent seismic vulnerability assessments conducted by WSP for the s.15(1)(l) [redacted] (2021) and s.15(1)(l) [redacted] (2019) have indicated that these bridges are seismically deficient even under a moderate earthquake. Similar seismic concerns have been raised at other bridges such as the [redacted] s.15(1)(l) [redacted]

A watermain can be damaged even if the bridge structure is not damaged. For example, during the 1995 Kobe earthquake, two 600 mm diameter welded steel watermains supplying water to the Port Island were damaged where they were supported on the Kobe Bridge. Both pipes were completely severed about 50m from the bridge abutment on the island side. This was caused by the differential settlement of bridge approaches while the bridge structure remained intact. During the 2016 Kumamoto earthquake, Wham (2017) reported a failure of an 800 mm diameter welded steel watermain. A leak occurred at the slip joint close to the south abutment due to compression of the joint. The compressive force was attributed to the lateral spreading that occurred in the riverbanks. It is important to highlight that seven repairs were required in the vicinity of the pipe bridge, including four repairs to valves and joints of the large diameter steel pipeline.

Seismic evaluation conducted by BGC (2003) for the pipe bridge supporting s.15(1)(l) [redacted] which could impact the north abutment. If occurs, watermain will fail at welded joints near the centre of the span. BGC (2003) proposed to install post-tensioned rock anchors to stabilize the rock slopes. We have considered this proposed remediation work has been undertaken; therefore, no failures have been considered at this crossing.

For bridges located in liquefiable areas, two breaks in the watermain were assumed due to the damaged bridge structure or/and ground movements at the bridge approaches. However, if liquefaction is not anticipated, two leaks were considered to account for any damages from differential movement occurring during seismic shaking.

4.4 RESULTS

The damage rates estimated for each transmission main are given in Appendix C of this report.

- Table C1 - summarizes the estimated damages for each watermain. Certain watermains are subdivided into smaller segments to account for different magnitudes and directions (with respect to the pipe axis) of ground movement and pipe types.
- Table C2 –summarizes the estimated leaks and breaks for each watermain. Leaks and breaks estimated at water crossings are shown separately and not included in the respective watermain. The reason for separating water crossings from watermains is to clearly illustrate the performance of watermains away from water crossings for the purpose of calculating the breaks per kilometer which in turn is linked to the serviceability of the pipe (see Section 4.4.1). For example, a water crossing may increase the number of breaks assigned to that watermain; thus, implying a poor performance of the entire watermain. Instead, the rating of the watermain is mainly controlled by the water crossing while remaining pipe sections are expected to perform satisfactorily.
- Table C3 - summarizes the estimated leaks and breaks for each watermain. Compared to Table C2, water crossings are also included in the respective watermain.
- Table C4 - provides the key input parameters used in estimating the number of failures and assumptions. Certain watermains subdivided into sections to account for variations in pipe material and seismic hazards.
- Figures F1 through F8 (Appendix F): Vulnerable pipe sections with a low Serviceability Index (marked as red). It should be reminded that plots shown in Appendices A and B show the hazards related to soil liquefaction and ground motion amplification but not necessarily the performance of the pipe. For example, if it was designed to accommodate the seismic loading, the vulnerability is considered low while the hazard remains high. Similarly if the pipe is located below the liquefiable soil, the watermain is expected to have a low vulnerability rating. For this reason, we consider the results included in Appendix F are more relevant for identifying the vulnerable pipe sections while maps included in Appendix B provide background information used in deriving the pipe vulnerability ratings. All maps are provided to Metro Vancouver in GIS format.

The key results are summarized in Table 9.

	Total	Failure Rate
Total Number of Failures	267	0.54 failures/km
TGD Induced failures	18	0.04 failures/km
PGD induced failures (all)	249	1.98 failures/km (within PGD areas)
PGD induced failures (water crossings)	97	
Total number of breaks	129	
Total number of leaks	138	

Table 4-7: Summary of pipe failures predicted from the vulnerability assessment

- For a 1 in 2475-year return period, the regional-scale assessment has estimated a total of 129 leaks and 138 breaks in the Metro Vancouver's water transmission network. The estimated damage rate is 0.54 failures/km when the total pipe length is considered. As expected, the highest concentration of failures was estimated to occur in areas susceptible to liquefaction and PGD. For example, 249 failures would occur in areas prone to liquefaction or landslides (i.e., 1.98 failures/km), which includes the water crossings. When water crossings are excluded, the damage rate becomes 1.10 failures/km in areas prone to PGD.
- As shown in Figure 4-13, approximately 93 percent of the failures can be attributed PGD induced failures. Also, approximately one-third of failures can be attributed to failures occurring near water crossings.

- A total of 129 breaks have been estimated, which is almost entirely (approximately 96 percent) attributed to PGD. Out of which, approximately 60 percent would occur near/at water crossings (see Figure 4-14). It is possible that number of breaks have been overestimated for steel watermains due to the assumed split between breaks and leaks. There are no reliable statistical data to improve this value and it was considered conservative to overestimate the breaks instead of considering them as leaks (see discussion on Serviceability Index in Section 4.4.1).

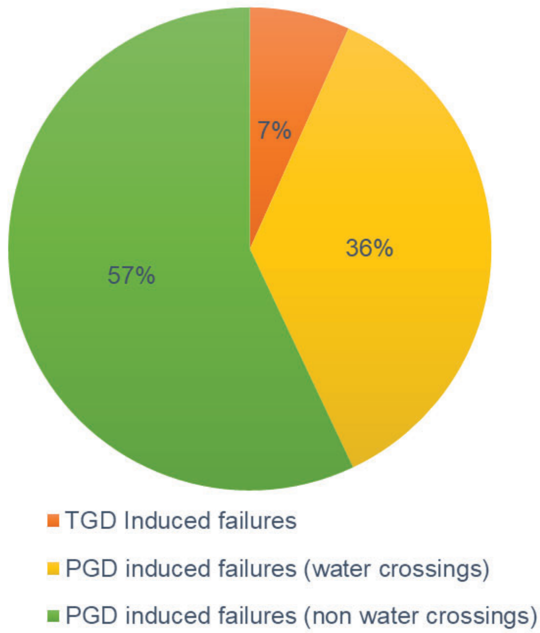

Figure 4-13: **Breakdown of failures into different modes.**

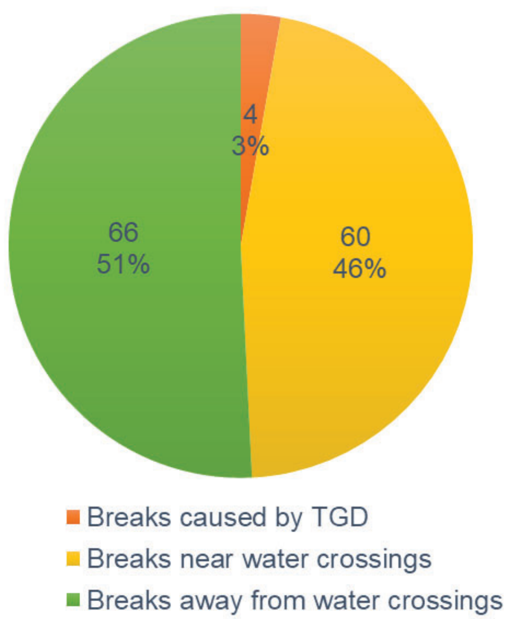

Figure 4-14: **Breakdown of breaks occurring in transmission mains**

- A total of 138 leaks have been estimated, and approximately 14 percent is attributed to TGD, while the remaining is attributed to PGD (see Figure 4-15).

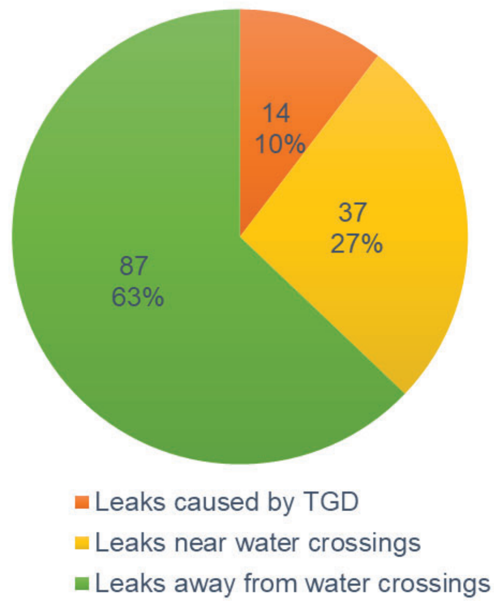

Figure 4-15: **Breakdown of leaks occurring in transmission mains**

- The average damage rate associated with TGD is 0.04 failures/km when all pipe material types and ground conditions are considered. Although this value is consistent with those observed in past earthquakes for a similar intensity of shaking, there is considerable uncertainty involved in fragility relationships used to estimate the damage rates for larger diameter pipes. The estimated damage rate may be conservative for continuous large-diameter pipes and further updates may be necessary if more refined fragility relationships are developed in the future for those pipes.
- It is critical to note that all leaks may not be detected immediate after the earthquake. For example, during the 1965 Nisqually earthquake, the official number of damages was only 38 leaks which were detected immediately after the earthquake. However, Ballantyne et al. (1991), who re-examined the repair rates of the next five years after the earthquake, noticed considerably higher repair rates than the average repair rate prior to the earthquake. Based on this information, they concluded that about 293 repairs can be attributed to this earthquake event. These leaks may not have a significant impact on the system performance immediately after the earthquake. A major break in the transmission main could also hide some failures in the downstream side.

4.4.1 HIGHLY VULNERABLE WATERMAINS

Typically, the breaks per kilometer of pipe is considered as a good indicator to determine the serviceability of a watermain after an earthquake. The serviceability levels estimated by different agencies and scholars are shown in Figure 4-16. The serviceability level relationship for Metro Vancouver's watermains may be different and it should be estimated from a separate study after considering the local conditions. However, for the purpose of identifying transmission mains with a high vulnerability rating, the serviceability relationship proposed by EBMUD for larger diameter pipes was considered. Note that the overall break rate for the entire pipe may not provide a true indication of the highly vulnerable sections if the impacted pipe length is only a fraction of the total pipe length. For this reason, pipe sections are divided into sections to depict the sections with high seismic vulnerability (see Table C4 in Appendix C).

Excluding the water crossings and watermains less than 2 km long, Table 4-8 lists the transmission mains with a Serviceability Index of 20 percent or less (or an average break rate greater than 0.12 breaks/km). Eidinger (2005) stated that in order to supply water to about 90 percent of the customers within about three days, the break rate needs to be less than about 0.1 to 0.2 breaks/km. This was considered in selecting the above serviceability index. For more detailed breakdown of pipe segments with respect to breaks per kilometer, please refer to Table C4 in Appendix C.

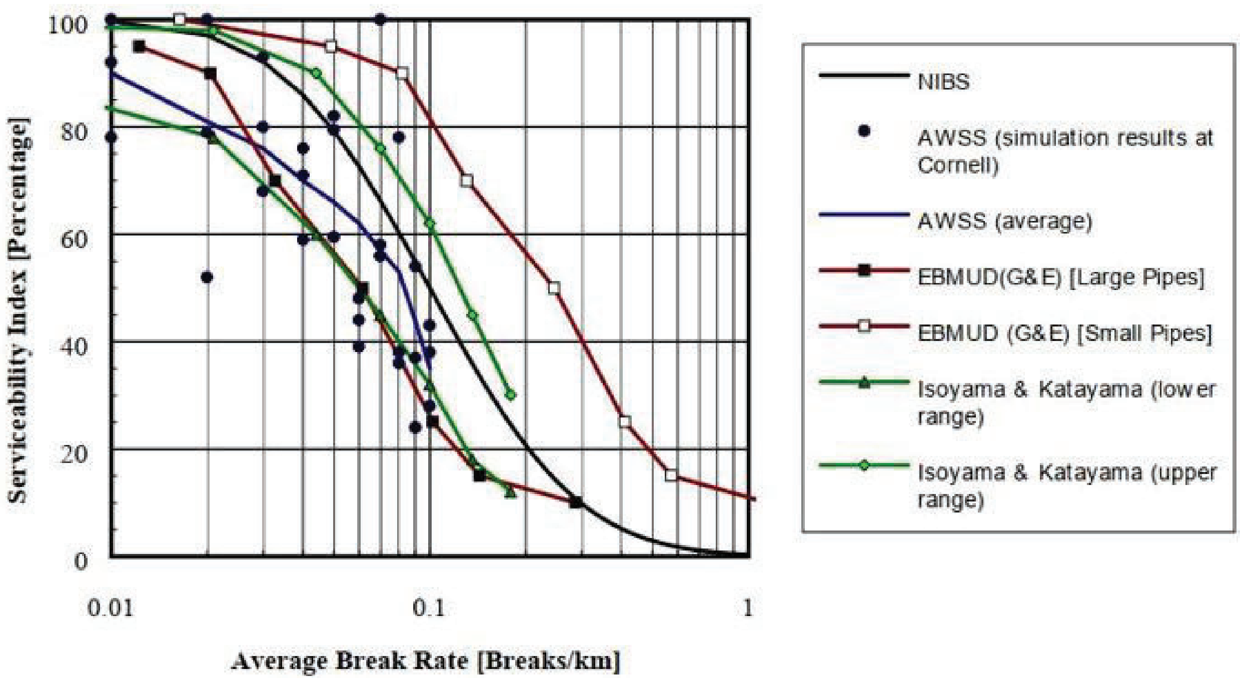

Figure 4-16: **Serviceability and average break rate relationships**

Watermain	Year of Construction	Pipe Material Type (l)	Total Number of Breaks	Total Number of Leaks	Breaks per Kilometer	Notes
Annacis Island Main, No. 2	1961	ST/SP	s.15(1)(l) [redacted]			
Annacis Island Main, No. 3	1973	ST/SP				
Barnston Island Main	1990/1998	ST/SP				
Capilano Main No. 4	1936	ST/SP and C				
Capilano Main No. 5 (Marine Drive to North Shaft and Beach Ave)	1969/1975	ST/SP				
Capilano Main No. 7	1957	ST/SP				
Coquitlam Main No. 2	1956/1959	ST/SP				
Coquitlam Main No. 3	1987	C, ST/SP				
Haney Main No. 2	1966	ST/SP				
Haney Main No. 3	1990/1994	ST/SP				
Lulu Island Delta Main	1962	ST/SP				
Port Moody Main No. 2	1976	ST/SP				
Queensborough Main No. 1	1949-1952	ST/SP				
Queensborough Main No. 2	2018	DI				
River Road – East Main	1980	ST/SP				
River Road Main – West Main	1978	ST/SP				

Watermain	Year of Construction	Pipe Material Type (I)	Total Number of Breaks	Total Number of Leaks	Breaks per Kilometer	Notes
Seymour Main No. 2	1948	ST/SP	s.15(1)(l) [redacted]			
Seymour Main No. 3	1925/1948	ST/R				
South Delta Main No. 1 (existing)	1964	ST/SP, AC, PVC				
South Delta Main No. 1 (Replaced)	2017	ST/SP				
South Delta Main No. 2	1992/1996	ST/SP				
Tilbury Main	1979	ST/SP				

s.15(1)(l) [redacted]

Table 4-8: Transmission mains with high vulnerability rating (excl. water crossings)

Transmission mains in areas such as s.15(1)(l) [redacted] are impacted by PGD resulting from high liquefaction likelihood. The s.15(1)(l) [redacted] is identified as another vulnerable area that could have a significant impact on the overall seismic performance of the system. Some of the key transmission mains are located in this area include s.15(1)(l) [redacted]. We expect the impact to be lessened after the construction of the Second Narrows Water Tunnel. However, s.15(1)(l) [redacted] by soil liquefaction. We understand that the tunnel section through Burrard Inlet will include two 2100 mm diameter watermains (Seymour Mains No. 2 and 5) with one 1500 mm diameter watermain (Capilano Main No. 7). We have assumed that the tunnel will be designed to protect these pipes from ground movements. s.15(1)(l) [redacted] is in potentially liquefiable soils where liquefaction induced lateral spreading may impact the tunnel/shaft connection. We have assumed that shaft/tunnel connections will be designed to prevent any damages.

Except for the sections of s.15(1)(l) [redacted] replaced in 2017, the other pipes are more than 30 years old. We could not confirm if special measures have been taken to improve the seismic vulnerability of recently replaced sections of s.15(1)(l) [redacted].

Besides above watermains, there are some other vintage pipes such as the [redacted] s.15(1)(l) [redacted] that may experience a significant number of leaks due to TGD. This relatively high damage rate was estimated considering their age, brittle nature of the pipe and pipe connections. These watermains are not included in Table 4-8 since their serviceability indices are marginally greater than 20 percent or length is less than 2 km. Note that the criteria for selecting serviceability is dependant on the “breaks/km” instead of “leaks/km” or total “failures/km”. Although the damage rate is low compared to those encountered in liquefiable regions, the failure rates of these watermains are about three times higher than those predicted for newer lap welded steel pipes situated in similar ground conditions.

4.4.2 WATER CROSSINGS

There are several pipe sections that show high damage rates in localized areas and near water crossings. The major river crossings (Category 1) were identified as highly vulnerable except for the Second Narrows and Port Mann Water Tunnels assuming they have been designed to withstand a major earthquake. We expect the seismic performance to improve with the construction of the remaining three water tunnels in the future. Even if the watermains are not impacted by soil liquefaction, tunnel shafts may be damaged from liquefaction induced ground deformations. Note that in some instances, relatively deep movement in non-liquefiable soils have been predicted using numerical (i.e., finite element or finite difference) methods. In such situations, a relatively small movement of stiff soil can exert a significantly large soil load on the shaft. These loads acting on the shaft may be sufficient to damage the pipe-shaft interface or the shaft itself. Some of the shafts have been designed to support earthquake levels somewhat similar to those expected under the 2020 NBC (e.g., Port Mann Water Tunnel, Second Narrows Water Tunnel). Therefore, it was assumed that these shafts have sufficient capacity to withstand loads from ground displacements.

The s.15(1)(l) [redacted] is also located in potentially liquefiable soils. A failure was reported in 1982, which was attributed to corrosion that occurred at the joint between the welded steel pipe and Bonna Pipe at a depth of 37 m below the ground surface. Once the leakage was fixed, the outside area was injected with grout to fill voids created by the leak. Another failure was reported in 1986, which occurred at a joint in the Bonna Pipe at a depth of approximately 43 m below the ground surface. This failure resulted in a complete failure of the shaft intake pipe and minor horizontal movement and tilt of the shaft s.15(1)(l) [redacted]. Following this failure, grouting was undertaken to fill the large voids created by the failure followed by ground densification around the shaft. Considering the age, performance history and soil conditions, the s.15(1)(l) [redacted] was assumed to be seismically vulnerable. Previous assessment by EBA (2000) for the MCE event have indicated potential vulnerabilities at (a) Intake s.15(1)(l) [redacted]

We assumed shafts surrounded by stable non-liquefiable soil were designed to withstand loads from transient ground shaking.

Category 2 crossings are also considered as deficient although the ranking and number of failures are not clear as in Category 1 crossings. At certain water crossings, there is some uncertainty related to the classification of the water crossings relative to Categories 2 and 3. Note that water crossing classification was mainly based on information included in O&M drawings and review of site photographs to determine the site conditions, topography and consequences of a failure; therefore, some subjectivity was involved.

The North Burnaby Main crossing the Burnaby Lake was not considered vulnerable due to expansion and flexible joints used at either ends of the water crossing. It was assumed the transmission main was designed adequately to accommodate the ground displacements. Our review of O&M drawings did not identify any similar joints in other water crossings, although it is possible that such details may have been excluded from O&M drawings.

We also recognize that some of the older pipes may have been significantly impacted by corrosion. According to AECOM (2021), cathodic protection was used in 27 out of 63 water crossings. However, this was not explicitly considered since the loading from large lateral spreading (or flow slides) may likely to overshadow the impact from corrosion.

4.4.3 COMPARISON WITH 1993 EQE STUDY

As stated earlier, the EQE (1993) study identified a total of 171 km (out of 432 km) of pipeline installed in liquefiable soil (i.e., approximately 40 percent). The current study has identified approximately 140 km out of 498 km pipes in potentially liquefiable or landslide areas (i.e., approximately 30 percent). This is understandable since the increase in seismic hazard has not significantly altered the areal extent of potentially liquefiable areas. In some watermains, liquefiable soils may encounter above the pipe. In this study, these pipe sections were not considered as PGD areas. Furthermore, some of watermains located in highly liquefiable areas have been either relocated, replaced or abandoned.

Despite the similarities in the areal extent, there are considerable differences in the number of damages estimated in the 1993 EQE study and in the current study. For example, EQE (1993) estimated approximately 31 watermain failures compared to 267 failures predicted by WSP. This is partly attributed to the increased seismic demand (e.g., 475-year event under 1990 NBC versus 2475-year event under 2020 NBC), where the PGA has approximately doubled. EQE (1993) estimated only 30 failures in the 171 km long pipe sections impacted by PGD, which resulted in an overall damage rate of 0.18 failures/km. Where steel pipes are involved, EQE estimated 18.7 damages over a length of 148 km of pipe in liquefiable soils (i.e., a damage rate of 0.125 failures/km). This is a considerably smaller ratio compared to observations from past earthquakes where the shaking intensity is consistent of a 2475 year return period event (see further details in Section 4.4.5).

As highlighted earlier, most of water crossings have been identified as vulnerable. EQE (1993) also classified these water crossings as high risk, except for the Capilano Main No. 7 and Seymour No. 3 which were classified as moderate risk. Our assessment has identified 40 water crossings in Categories 1, 2 and 4 as high risk, and at least one to three failures have been predicted at each of these crossings. These failures alone would exceed the total number of failures estimated by EQE.

It appears that EQE (1993) estimated the damage rate based on Ariman, et. al. (1990). The low damage rate may have been partly resulted from the “areal distribution” factor assumed in the analysis. A factor of 0.35 was considered for the PGD induced damages. In a later publication, Donald Ballantyne of EQE highlighted the significant uncertainty associated with this value which was estimated by project’s geotechnical team. He reported that estimates for areal extent of liquefaction varied by a multiple of seven.

The number of damages estimated by EQE for TGD is also low for steel pipes with 0.7 failures predicted for 217 km of steel pipe (i.e., 0.003 failures/km) in non-liquefiable areas. Comparatively, the failure rate calculated from ALA (2001) for the current study is 0.04 failures/km, which includes older pipes and those with highly vulnerable connections such as rivetted connections. As highlighted earlier, the 2016 Kumamoto earthquake resulted in a failure rate of 0.05 failures/km when air valves alone are considered. This reflects the significant uncertainty associated with the estimation of TGD induced damage rates for large diameter continuous steel pipes.

As highlighted earlier, the 1993 EQE study was conducted prior to any major developments in fragility relationships and based on a smaller database. Despite the disparities in damage rates, EQE concluded that the pipe network would be inoperable after a 475-year return period event. The WSP study has also identified several critical pipes that are likely to experience severe damage after a major earthquake rendering large sections of the network inoperable (see Section 4.4.1).

4.4.4 COMPARISON WITH PAST EARTHQUAKES

4.4.4.1 *TGD INDUCED DAMAGES*

According to the current assessment, TGD induced failures account for approximately 7 percent of the total failures. Although higher damage rates have been estimated in other earthquakes such as the 1906 San Francisco, 1964 Puget Sound, 1969 Santa Rosa, 1983 Coalinga and 1985 Michoacán (see Section 2.4.1), Ballantyne (1995) highlighted concerns related to the division of pipe damages between TGD and PGD after an earthquake. Often, damages are attributed to PGD if there are clear surface manifestations to confirm soil liquefaction. However, in some instances ground settlement may occur without visible surface manifestations; therefore, the damages from TGD may be overestimated. As an example, during the 1994

Northridge earthquake, Ballantyne (1995) noted that approximately 900 water pipe damages occurred in areas with almost no evidence of liquefaction or fault movements; therefore, those were attributed to TGD damage category. Compared to some historical earthquakes, the areas that experienced liquefaction and PGD during the 2010 Christchurch event were well documented using borehole investigations and LiDAR maps (Cubrinovski et al., 2012), which may have led to a relatively higher PGD damage percentage than in some other earthquakes.

The overall TGD induced damage rate from the current study is 0.04 failures/km (0.012 failures/1000 feet). This is shown in Figure 4-17 along with the range of PGV values estimated for a 2475-year seismic event occurring in the Greater Vancouver region. The figure background includes data from the 1994 Northridge earthquake for all pipe material types.

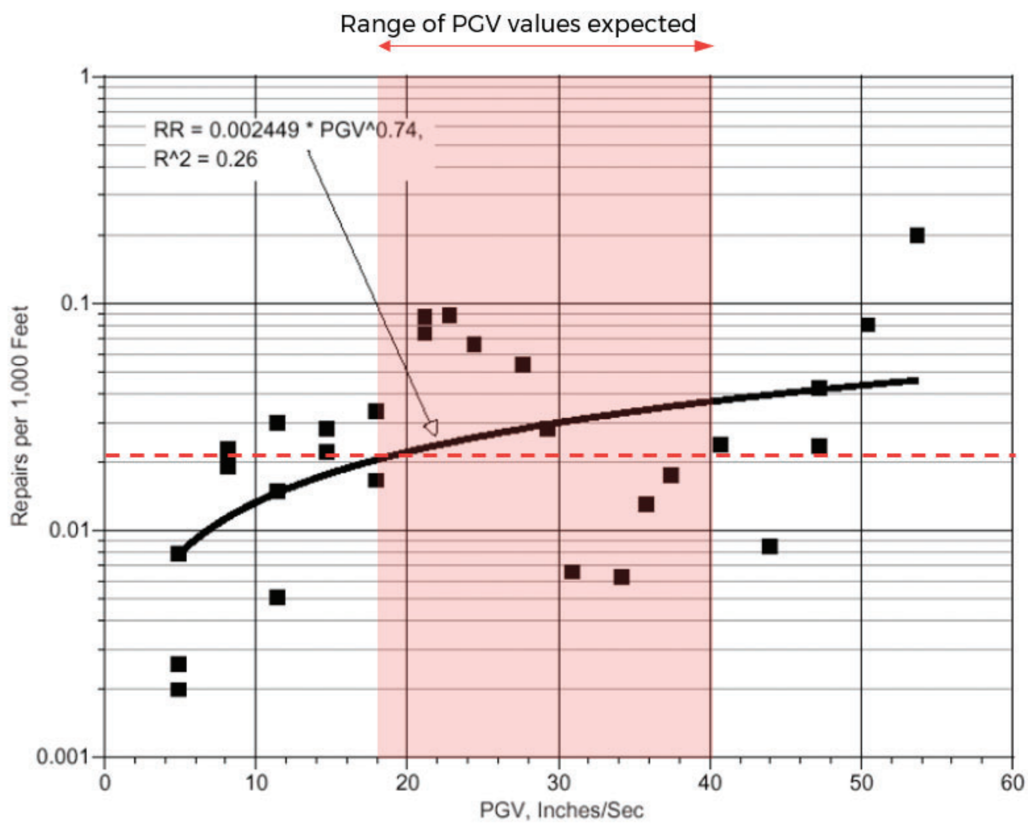

Figure 4-17: **TGD induced damage rate from the current study compared against those measured during the 1994 Northridge earthquake (figure background adopted from ALA 2001)**

4.4.4.2 *PGD INDUCED DAMAGES*

Typically, the damage rate from PGD is one (Ballantyne, 1995) to two orders (ALA 2001) of magnitude greater than that caused by TGD. In this study, the estimated damage rates for PGD and TGD were 1.99 and 0.04 failures/km, respectively, which is somewhat consistent with this observation (i.e., a ratio of approximately 1:50).

The weighted average PGD for this study is about 38 cm (15.3 inches) and the corresponding damage rate is 1.1 failures/km (or 0.34 failures/1000 feet). Figure 4-18 shows several empirical fragility based relationships recommended for estimating the PGD induced damages. Those relationships are applicable for brittle pipes, and for ductile pipes, the damage rate is expected to be about 0.3 times of those recommended for brittle pipes (see Section 4.4.3). Majority of pipes included in Metro Vancouver's transmission network are considered as ductile. Therefore, for comparison purposes, the damage rates estimated from this study was divided by 0.3 and plotted in Figure 4-18. Despite the approximate nature of this comparison, it is of interest to note that the damage rates estimated using the PSI based method (used in this study) fall within the range of damages estimated using different fragility based relationships.

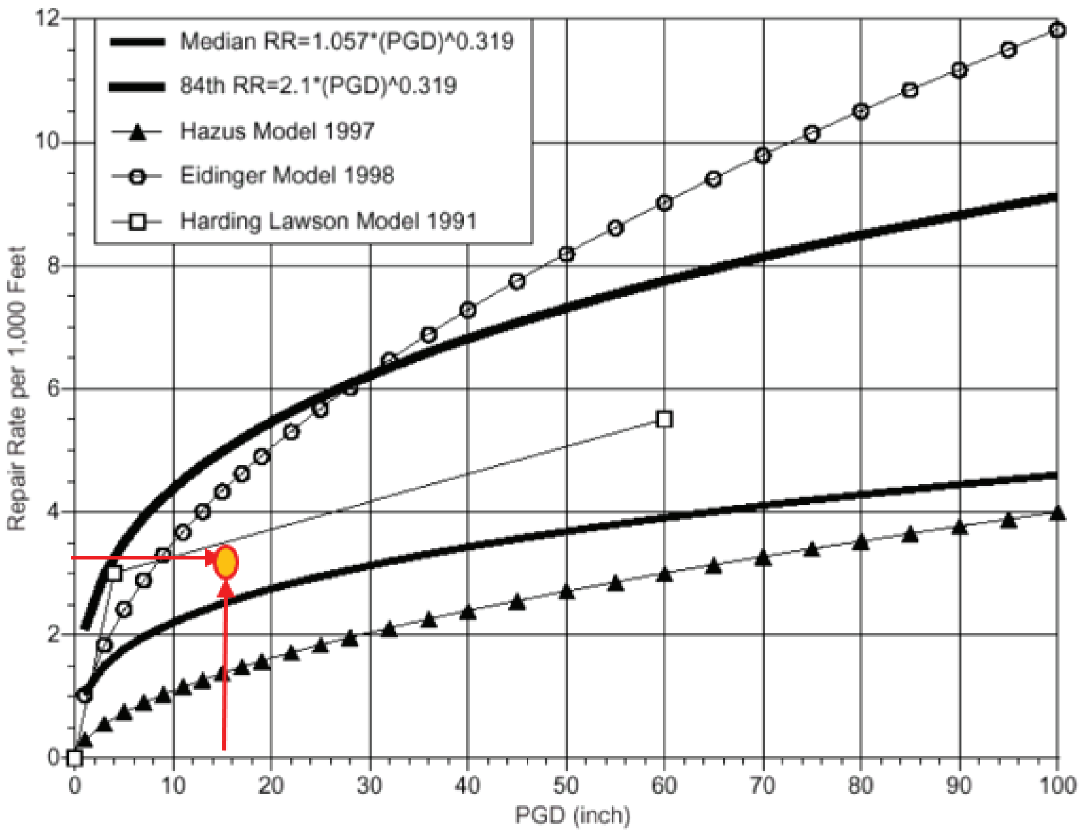

Figure 4-18: **PGD induced damage rate estimated from the current study (divided by 0.3) compared against empirical fragility based relationships (figure background adopted from ALA 2001).**

4.4.5 TOTAL DAMAGE RATE

As discussed earlier, there is considerable uncertainty related to the separation of damages caused by TGD and PGD in past earthquakes. For this reason, Ballantyne (1995) argued for the use of generalized pipe damage calculation without separating the damages into TGD and PGD categories. An example of this approach proposed by Katayama., et al (1975) where the damage rate is related to PGA (see Figure 4-19). It should be noted that the use of Katayama's relationship for this study will overestimate the damage rates because their database mainly consisted of brittle pipes.

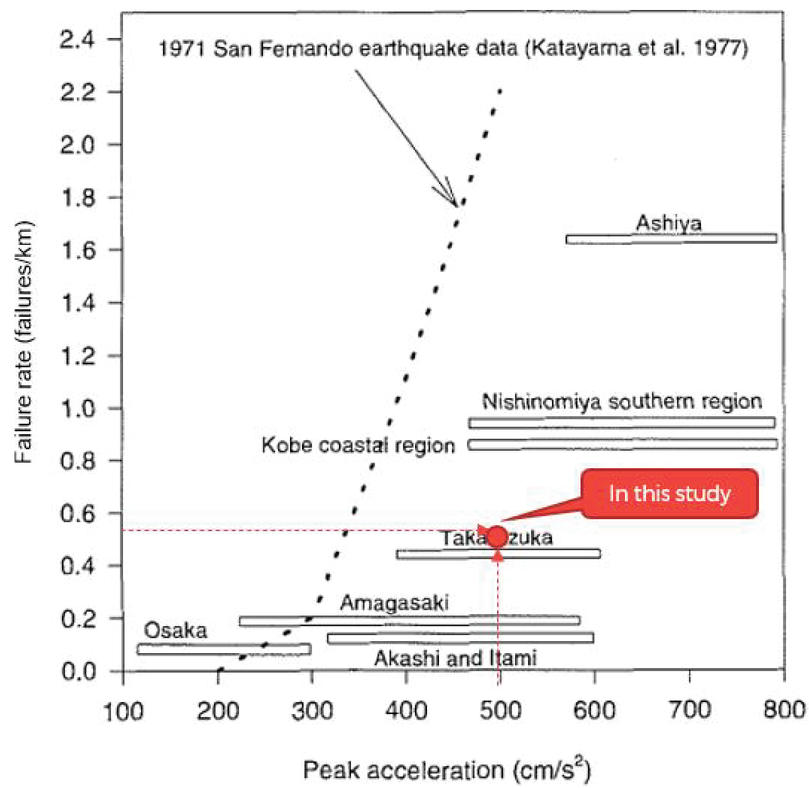

Figure 4-19: **Pipe damage rates from Katayama et al. (1977) and 1995 Kobe earthquake (adopted from Kuraoka and Rainer, 1996).**

Figure 4-20 shows the damage rates observed during the 1995 Kobe earthquake where the PGA and extent of liquefaction are expected to be somewhat similar to a major earthquake occurring in the Metro Vancouver region. The overall damage rate estimated for Metro Vancouver's transmission mains is 0.54 failures/km, which is comparable to the damage rates observed during the 1995 Kobe earthquake.

In the current seismic assessment, the damage rate for welded steel pipes is 0.51/km (including the water crossings), which is slightly smaller than the overall damage rate estimated for all pipe types. Comparatively, the reported failure rate for welded steel pipes in the 1995 Kobe earthquake was 0.47 failures/km (Ballantyne, 1997). This damage rate was considerably lower than the damage rates reported for brittle pipes such as CI, PVC and AC pipes. It was also reported that the damage rate for larger diameter pipes (greater than 300 mm in diameter) was 0.69 failures/km during the 1995 Kobe earthquake, although it is unclear if this included both brittle and ductile pipes. Somewhat lower damage rates have been reported in the Northridge and Loma Prieta earthquakes probably due to their smaller shaking intensity and relatively smaller extent of liquefaction (see Table 4-9). During the 2016 Kumamoto earthquake, steel pipes resulted in a repair rate of 0.5 failures/km (Ishida et al., 2016), which is approximately similar to the damage rate predicted from this study. Comparatively, the shaking intensity and perhaps also the extent of liquefaction for this event are also similar to a 2475 year event occurring in the Metro Vancouver region.

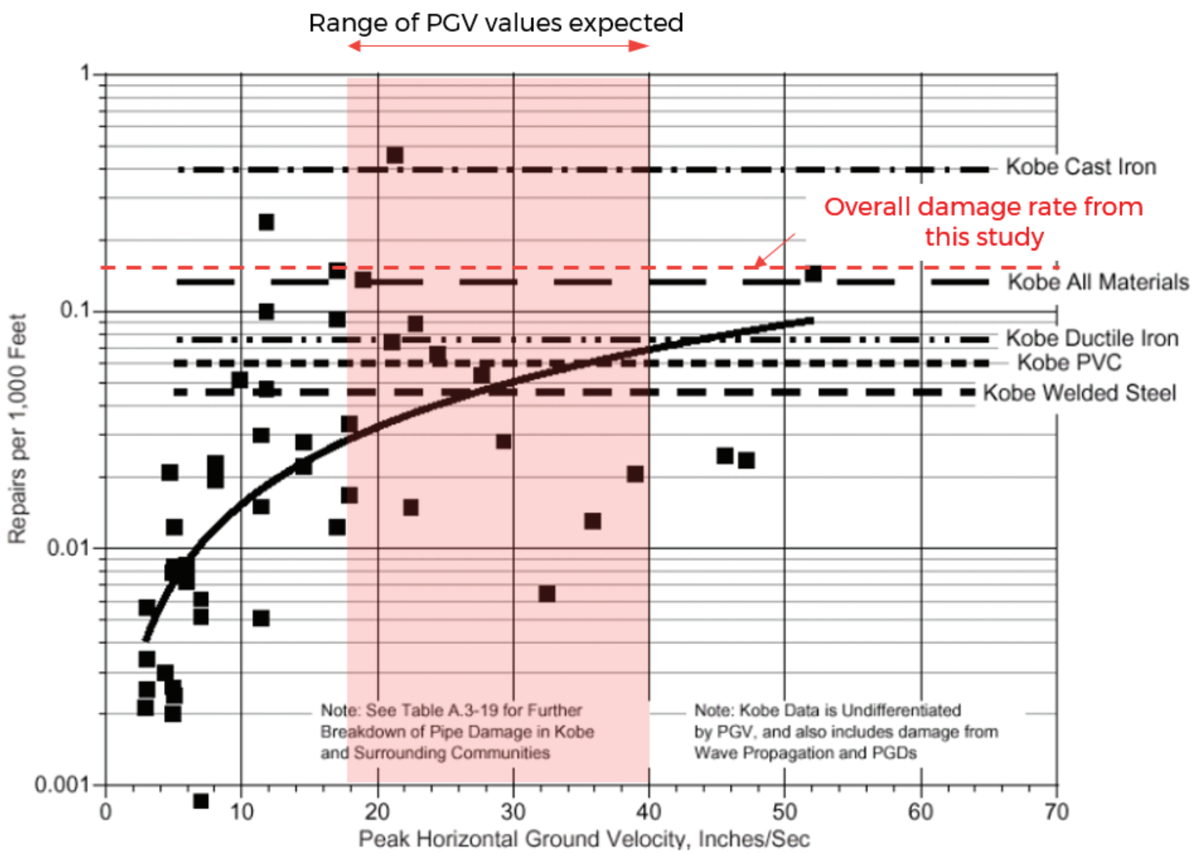

Figure 4-20: Damage rates estimated from this study compared against those occurred during the 1995 Kobe earthquake (background figure adopted from ALA (2001)).

Observations from the 2010 Christchurch event indicate that steel pipes suffered the greatest damage with an average “affected length” of 8.9 percent in all areas (Cubrinovski et al., 2012). For steel pipes located in areas that are subjected to severe liquefaction, the affected pipe length was 20 percent. There is no simple method to convert the “affected” length to repair rates; therefore, the data from 2010 Christchurch earthquake was not included in Table 4-9.

In the process of this assessment, we have observed some inconsistencies in the reported damage rates in various publications for the same event. The reasons for these discrepancies were not investigated but we expect they were caused by various qualitative factors. For example, detection of minor leaks would take months to observe after the earthquake; therefore, such repairs may not be included in some reconnaissance conducted immediately after the earthquake. Also, identifying the number of repairs is not straight forward when several pipe sections are replaced at once. More importantly, there is subjectivity involved in the definition of impacted zone and pipe length.

Earthquake	Overall Damage Rate (failures/km)	PGA and Extent of Liquefaction	Source
1995 Kobe earthquake	0.47	0.4 to 0.8g with significant liquefaction	Ballantyne (1995)
1994 Northridge (LAPWP – distribution pipes)	0.17	0.5 to 0.9g with minimal liquefaction	
1989 Loma Prieta (EBMUD)	0.04	0.05 to 0.25g with high liquefaction	
2016 Kumamoto (all pipe sizes)	0.50	0.27 to 1.18g with widespread liquefaction	Ishida et al. (2016)
Metro Vancouver Watermains during a 1 in 2475-year return period	**0.54**	**~0.5g with severe liquefaction expected impact more than 30 percent of watermains**	**Current Study**

Table 4-9: Damage rates reported for steel pipes in past earthquakes

In general, the estimations from the current seismic assessment are generally consistent with the assessment of damages from historical earthquakes, with respect to the overall damage rate and those estimated for PGD and TGD.

Table 4-9 and Figure 4-19 also highlight the differences in damages that can be expected if a smaller intensity earthquake occurs in the Metro Vancouver region. For example, if the PGA is half of that considered for the 1 in 2475-year event, the damage rates can be several times smaller than that estimated for this study. Estimating the damage rates for different earthquake levels is beyond the scope of this study.

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5 FACILITIES

5.1 INTRODUCTION

This regional-scale seismic vulnerability assessment has considered 26 reservoirs (at 21 sites), 19 pump stations at (17 sites). Details of these structures were provided by Metro Vancouver in several design reports and O&M drawings.

Most of the reservoirs and pump stations are below-ground reinforced concrete structures. Past earthquakes have indicated that these structures are generally resilient to earthquake loading unless they are situated in liquefiable soils. For example, during the 2010 Christchurch earthquake, out of 54 reservoirs, only four reservoirs required minor repairs and two reservoirs required substantial repairs or replacements (Hunt and Hutchison, 2014). Three of these reservoirs were in liquefiable alluvial soils, while others were founded on bedrock. Two tanks experienced serious damage and were located on the alluvial plain which was subjected to liquefaction.

5.2 FAILURE MODES

5.2.1 ROOF DAMAGE

The most common structural failure associated with reservoirs is the failure of the connection between roof and perimeter walls due to the seismic inertial force of the roof slab. In addition, in reservoirs either fully or nearly filled, sloshing of fluid during earthquake shaking can cause an upward pressure on the roof if sufficient freeboard is not provided. In these situations, unconnected roofs or those with weak connections have sustained extensive damage from sloshing forces. Design codes (e.g., API, AWWA) prior to 2000 did not include specific design guidance for roof systems against sloshing forces. As stated in Section 5.1, two reservoirs experienced severe damage during the 2010 Christchurch earthquake which included damage to the roofs. In one tank, the damage was at a concrete nib at the top of the wall, which was caused by the inertial load of the roof. In the other tank, the roof partially collapsed due to the sloshing forces, which caused the slab to be lifted off the perimeter wall and subsequently fractured upon impact with the perimeter wall.

Roof damage was identified as a likely failure mode in previous seismic assessments conducted on Metro Vancouver's older open cut reservoirs. The 1993 EQE study indicated that several reservoirs, most of which were built between 1910 and 1980, have insufficient structural capacity in the roof slab to withstand a moderate earthquake.

Although roof damage may be expensive to repair, it generally does not lead to a significant loss of water. In some past events, the falling debris have clogged the inlet/outlet pipes or impact the water quality.

Compared to large reservoirs, the pump station is more compact, and the roof slabs of these pump stations may be able to sustain a greater seismic loading without damage.

5.2.2 FLEXURAL FAILURES OF WALLS

Previous seismic assessments have identified flexural deficiencies along perimeter walls of reservoirs. This is caused by hydrodynamic forces from water inside the reservoir and/or dynamic lateral earth pressures. During an earthquake, these horizontal forces will amplify the bending moments and shear forces in the perimeter walls. This mode of failure is common to underground reservoirs and pump stations and is recognized as one of the most likely failure modes in the past seismic assessments.

In reservoirs, the lateral forces could cause a large drift of the structure, which in turn may impact the roof-column or/and base slab-column connection.

Compared to a reservoir that is more susceptible to lose its content, flexural cracking of perimeter walls of a pump station is unlikely to cause a significantly impact to the post-disaster functionality. However, some of the above-ground pump stations include unreinforced masonry structures s.15(1)(l) [redacted] These masonry blocks can support the vertical loads, but they are weak against lateral loads from earthquakes. Therefore, unreinforced masonry/brick structures are highly vulnerable, and likely to experience severe damage or collapse from seismic shaking.

5.2.3 DAMAGE TO OUTLET AND INLET PIPES

Failures of the inlet-outlet pipe is considered as one of the most common failure modes as they represent the weakest link in the system (Hunt and Hutchinson, 2014). Damage to pipe connections is usually caused by the differential displacement of the pipe and structure arising from permanent ground displacements or differential response of the pipe and structure during seismic shaking. The most critical case is the liquefaction induced vertical or horizontal displacements since the magnitude of differential displacement is likely sufficient to cause failure of the outlet/inlet pipes (e.g., 1985 Chilean earthquake). For example, during the 2011 Canterbury earthquake, the water stored in the Huntsbury reservoir was discharged through cracks in the floor and a damaged outlet valve (Macbeth et al. 2014). Subsequent investigations identified a shear zone in the underlying basalt layer directly below the reservoir, which contributed to these damages.

In the absence of soil liquefaction, Kennedy and Kassawara (1989) suggested that any type of flexible loop provided at the reservoir interface would result in a low probability of failure, provided that the PGA is less than about 0.5g. Note that a PGA of 0.5g is expected for a 1 in 2475-year return period event as per the 2020 NBC. Damage may occur at a smaller PGA level if there is no or only slight flexibility. It is difficult to completely eliminate damages even if retrofitting measures have been implemented considering the intensity of ground shaking expected from a 1 in 2475-year return period event. Even if ground displacements do not occur, the vertical displacement caused by wall buckling (in steel tanks) and uplift could lead failures in the inlet/outlet pipes. However, such failures are more likely to occur in unanchored steel tanks with relatively high H/D ratio (i.e., greater than about 0.7); therefore, it is not recognized as a likely failure mode for Metro Vancouver's reservoirs.

Almost all reservoirs belonging to Metro Vancouver's supply network are founded on competent ground with a low likelihood of liquefaction. However, liquefaction may occur above the foundation level where pipe connections are located. This liquefaction may lead to horizontal and vertical displacements that may exceed the capacity of the connection. If an inlet or outlet pipe breaks, the reservoir is likely inoperable, and it will be out of service even though the tank itself is structurally sound. A piping failure could also result in an extensive scour near the failure location, which could further exacerbate the damage to the structure and repair times. The loss of water from a minor leak may not significantly impact the functionality and can be repaired within a short period if the damaged pipe section is easily accessible. Available O&M drawings do not include sufficient details to examine the type of inlet/outlet pipe connections. The degree of damage to inlet/outlet pipes and repair times cannot be estimated without a site-specific study.

5.2.4 FOUNDATION/GROUND FAILURES

Typically, soil liquefaction below the foundation will pose a significant threat to storage tanks and pump stations. Even a few centimeters of ground settlement may be sufficient to cause damage to the base slab of a concrete tank. According to FEMA (2020), there is a 50 percent chance of substantial damage to concrete water storage tanks if the differential movement is about 600 mm, which could also lead to the loss of the entire stored content. Similarly, a permanent ground displacement of about 200 mm is sufficient to cause widespread damage to the roof structure of an open cut reservoir. For example, a large underground reinforced concrete reservoir at the Balboa water treatment plant suffered severe damages during in the 1971 San Fernando earthquake. This was caused by a foundation failure, where the vertical displacement of loose/compact fill supporting the tank was about 450 mm.

The liquefaction of soil below the tanks combined with the additional shear-induced settlement caused by rocking of the tank can settle and tilt the structure. This shear-induced settlement component cannot be ignored and is sometimes comparable to the free-field settlement caused by liquefied soils. Simplified methods to evaluate shear-induced settlement component were developed only recently (e.g., Bray and Dashti, 2014).

Except for a couple of structures, all reservoirs and pump stations are likely to be founded on competent ground where soil liquefaction is unlikely. Therefore, the likelihood of damage from soil liquefaction is considered low despite the anticipated increase in seismic demand in the 2020 NBC. Additional geotechnical analysis may be required for the s.15(1)(l) [redacted] to confirm the cyclic softening potential of clay-like soils encountered at these locations (further details given in Section 5.2.4).

In addition to the impact from soil liquefaction, any above-ground reservoirs or pump stations located near landslide areas or below areas of rock source could be impacted by debris flows or rockfalls. In our regional-scale assessment, we have not identified any facilities that are susceptible to damage from rockfalls or debris flows.

Sliding is a less common failure mode for large reservoirs and pump stations. There are no known cases of sliding failures of anchored tanks with diameters greater than 10 m. Even if sliding occurs, it is unlikely to cause structural damage, aside from potential damage to inlet/outlet pipe connections.

5.2.5 BUOYANCY

Similar to pipes in liquefied soils, buried structures could experience large buoyancy forces as a result of the liquefaction of surrounding soils. For example, during the 2010 Darfield earthquake in Christchurch, at least two pump stations in the wastewater collection system were uplifted and failed near the Avon River. We have not identified any pump stations or reservoirs in the Metro Vancouver network with a high likelihood of uplift as they are mostly founded in competent soil. In some instance, loose saturated soils surrounding the buried structure may experience liquefaction. However, our review indicates that these layers are mostly discontinuous and sporadic, thus insufficient to cause widespread liquefaction that could cause uplift of the structure.

5.2.6 OVERTURNING OF ABOVE-GROUND TANKS

There are only a couple of above-ground tanks in Metro Vancouver’s water supply network. For these tanks, critical factors that influence the likelihood of overturning are the height to diameter ratio (H/D) and the bearing capacity of the foundation soil. Given the relatively low H/D ratio of these tanks and relatively competent soil conditions, the likelihood of overturning is deemed low. No overturning risks have been identified for any of the pump stations.

5.2.7 SECONDARY AND NON-STRUCTURAL DAMAGES

Secondary damage occurs when a failure of a structure or ancillary component damages another. For example, during the 1971 San Fernando earthquake, the collapse of the east outlet structure in the Lower Van Norman Reservoir caused sand gravel and rocks to enter the water distribution system. As a result, pump packing and seals of all pumps were damaged by sand (FEMA, 2020). It is not possible to accurately estimate the likelihood of a secondary damage without conducting a site-specific assessment of the facility.

The post-earthquake performance of these facilities will also be impacted by non-structural damages, which is not within the scope of this study. For example, movements from ground shaking may damage guides, ladders and other appurtenances attached between the roof and base of the structure. Also, treatment plants will include a number of pipes supported on hangars or blocks. These pipes may experience damage from differential movements between two supports or resonance of the pipe itself. If the amplitude of vibration is large, failures could occur at discontinuities such as elbows, valves, attachments to equipment, wall penetrations and dissimilar points of restraint.

5.3 DAMAGE LEVEL

The five-level damage rating system proposed by FEMA (2020) was adopted for facilities. We believe that this system will allow Metro Vancouver to estimate the recovery period based on the guidelines provided in FEMA (2020). For example, Table 5-1 shows the anticipate restoration periods for different Damage Levels. These values were adopted from ATC-13 (1985) into FEMA (2020) and are based on past earthquake experiences. The recovery periods of Metro Vancouver's facilities may differ and will depend on the resource availability, accessibility, damages to other infrastructure including hydro, etc.

Facility	Damage State	Mean (Days)	Stand. Deviation (Days)
Water Treatment Plants	Slight/minor	0.9	0.3
	Moderate	1.9	1.2
	Extensive	32	31
	Complete	95	65
Pump Stations	Slight/minor	0.9	0.3
	Moderate	3.1	2.7
	Extensive	13.5	10
	Complete	35	18
Reservoirs	Slight/minor	1.2	0.4
	Moderate	3.1	2.7
	Extensive	93	85
	Complete	155	120

Table 5-1: Typical restoration times for water treatment plants, pump stations and reservoirs based on the damage level (adopted from ATC-13, 1985)

As stated earlier in the report, recovery modeling is beyond the scope of this paper. Examples of recovery modeling are included in different publications e.g., the Santa Clara Valley Water distribution system by Ballantyne et al., (2006). In addition to modelling, the actual recovery periods observed in different earthquakes have been reported in different publications. For example, after the 2011 Christchurch earthquake, several reservoirs were significantly damaged. Apart from the Huntsbury No. 1 Reservoir, most of these reservoirs returned to service within a few weeks after carrying out temporary repairs. The Huntsbury Reservoir No. 1 required 5 and 17 months to reach 6ML and 7 ML capacities, respectively. Further details related to damages to these reservoirs and recovery periods are given in Hunt and Hutchison (2014).

5.3.1 DAMAGE RATING

The five-level damage rating system adopted for facilities is described below (slightly modified from FEMA, 2020):

NONE

- No damage to the structure. It is unlikely that such conditions would prevail considering the intensity of seismic shaking expected during a 1 in 2475-year return period event. Although there may not be any structural damages, non-structural damages are almost unavoidable.

SLIGHT/MINOR DAMAGE

- For reservoirs, the structure has suffered minor damage without loss of its contents or functionality. This may include minor damage to the reservoir roof due to water sloshing and minor cracks in concrete perimeter walls or/and columns.

- For pump stations, this includes a plant malfunction for a short time (less than three days) due to the minor damages to the structure. FEMA (2020) also considers the loss of electric power and backup power during this period.

MODERATE DAMAGE

- For reservoirs, this involves considerable damage to the structure with minor loss of water. For example, moderate cracking in concrete perimeter walls or base.
- For pump stations, this involves moderate damage to buildings, loss of electric power for about a week, and considerable damage to mechanical and electrical equipment.

EXTENSIVE DAMAGE

- For reservoirs, this involves severe damaged and service outages to the structure.
- For pump stations, this involves extensive damage to buildings, or the pumps being severely damaged beyond repair.

COMPLETE DAMAGE

- For reservoirs, this involves complete collapse of the structure and loss of all of its content.
- For pump stations, this involves collapse of the structure.

Based on the descriptions provided above, it is apparent that there is subjectivity involved in selecting the appropriate Damage Level. This subjectivity is difficult to avoid in a regional-scale study, and the rating provides an indication of the “relative” vulnerability of different facilities.

5.4 RESULTS

For each facility, a Seismic Evaluation Sheet has been prepared and included in Appendix D. Each sheet includes a brief description of the structure/location, previous seismic analysis, targeted performance, retrofitting measures (if any), soil and groundwater conditions, liquefaction potential, anticipated ground displacements, likelihood of a given failure mode, Damage Level, uncertainties associated with the evaluation and recommended actions to address the uncertainties.

5.4.1 RESERVOIRS

The following summarizes the Damage Levels selected by WSP for each reservoir. To further confirm the validity of these assessments, WSP’ structural engineering team conducted a somewhat in-depth assessment for three selected reservoirs s.15(1)(l). A memo prepared to summarize the findings of the structural analysis is included in Appendix D of this report.

Reservoirs	Year of Construction	Year of Seismic Retrofit	Seismic Event for Retrofit or Design	Importance Factor	Damage Level
Prospect	1962	2003	MCE	1.0	s.15(1)(l) [redacted]
Vancouver Heights(a)	1928/1968	1996	MCE	1.0	
Little Mountain(b)	2003	2003	MCE	1.0	
Kersland(c)	1954 and 1958/59	1997	MCE	1.0	
Sasamat	1964	2021	2015 NBC	1.5	
Burnaby Mountain (d)	1971	2017	2010 NBC	1.5	
Westburnco (e)	1967/68	2004	MCE	1.0	
Whalley (f)	1966	2006	MCE	1.0	
Kennedy Park (g)	early 1980's	2012	2005 NBC	1.0	
Hellings Tank	1973	2015	2010 NBC	1.5	
Newton	1976/1986	2010	2005 NBC	1.5	
Sunnyside	1971 & 91	2021	2015 NBC	1.5	
Central Park(h)	1974/1975	1998	MCE	1.0	
Pebble Hill	1971/1977/1990	2019/2020	2015 NBC	1.5	
Clayton	2018	-	2010 NBC	1.5	
Glenmore tank	1989	-	1980 NBC	1.0	
Cape Horn (i)	1980	1980	MCE	1.0	
Grandview	1999	-	1995 NBC	1.0	
Greenwood	1984	-	1980 NBC	1.0	
Maple Ridge	1983	-	1980 NBC?	1.0	
Jericho	2023?	-	2010 NBC	1.5	

s.15(1)(l) [redacted]

Table 5-2: Damage Levels selected for reservoirs

5.4.2 PUMP STATIONS

The following summarizes the estimated Damage Levels selected by WSP for each pump station.

Pump Station	Year of Construction	Year of Seismic Retrofit	Seismic Event for Retrofit or Design	Importance Factor	Damage Level
Barnston/Maple Ridge	2018	-	2005 NBC	1.5	s.15(1)(l) [redacted]
Burnaby Mountain	1967	2011?	2005 NBC?	1.5	
Cape Horn	1980/94	2018	2015 NBC	1.5	
Capilano	2006	-	1995 NBC?	1.0	
Central Park	1974	2004	MCE	1.0	
Cleveland Dam	1973	2017	2015 NBC	1.5	
Grandview	1991	2014	2010 NBC	1.5	
Kersland	1963	-	-	-	
Little Mountain	1969	-	-	-	
Mahon	1984	-	-	-	
Newton No. 1 & 2	1966	2013	2010 NBC	1.5	
Pebble Hill	1972	2019	2015 NBC	1.5	
Sasamat	1964	2014	2010 NBC	1.5	
Vancouver Heights	1958	2004	1995 NBC	1.0	
Westburnco No. 1	1971	2003	MCE	1.0	
Westburnco No. 2	1999	2012	2010 NBC	1.5	
North Delta/ Hellings	1971	-	Pre 1970 NBC	1.0	
Greenwood	2003	-	1995 NBC?	1.0	

Table 5-3: Damage Levels selected for pump stations

We understand that Kersland, Little Mountain and Mahon pump stations have been seismically evaluated, although they were not upgraded.

5.4.3 DISCUSSION

To illustrate the changes to the design seismic demand, the evolution of the Metro Vancouver's Seismic Design Standards and changes to the National Building Codes should be considered. The initial version of Metro Vancouver's Seismic Design Standard (revision A) was issued in 2003. In this edition, structures were classified as 'Level 1' or 'Level 2' based on their importance:

- Level 1 Facility: This is critical to the system performance and has little or no redundancy. If damaged, service will be substantially reduced for an extended period. This type of structure should be designed to MCE with a 1 in 10,000 year return period. A considerable number of reservoirs have been upgraded based on this requirement.
- Level 2 Facility: There is some level of redundancy and it will have a marginal impact on the post-earthquake service for a limited period (days to weeks) if damaged. This should be designed to a 1 in 475 year return period event as per the 1995 NBC. We have identified only the Vancouver Heights pump station retrofitted as per this category. With the release of revision B of the Seismic Design Criteria in 2007, the design seismic event was upgraded to 1 in 2475-year return period, which is consistent with the 2005 NBC requirements. Only Kennedy Park reservoir was upgraded according to this requirement.

The release of the 2005 NBC resulted in a major change to seismic design approach, which required facilities to be designed to a 1 in 2475-year return period event compared to the 1 in 475-year return period specified in the previous codes. This caused the seismic design force to increase approximately by two times, depending on the soil type and fundamental period of the structure. No significant changes to the seismic demand occurred with the introduction of the 2010 NBC. Four of Metro Vancouver's reservoirs and five pump stations have been upgraded or designed based on the 2005 and 2010 NBC codes.

A significant drop in the seismic hazard values in the short period range occurred with the release of the 2015 NBC. In particular, the PGA was about 35 percent less than those expected in 2020 NBC. This resulted in significantly smaller seismic lateral earth pressures compared to the 2005, 2010 and 2020 NBC codes. Three reservoirs and three pump stations have been retrofitted or designed based on this code.

5.4.3.1 *CHANGES TO SEISMIC INERTIAL FORCE*

For pump station and reservoirs, the seismic event/design code and Importance Factor selected for design or seismic retrofitting are given in Tables 5-2 and 5-3. This allows a comparison to be made between the seismic "inertial" demands (mass times acceleration) considered for each structure against those expected in the 2020 NBC. As discussed in Section 2.2, the seismic demand is expected to increase with the adoption of the 2020 NBC as compared to previous seismic design codes. In general, this increase will range between approximately 10 to 25 percent for structures designed/retrofitted using the 2010 or 2005 NBC codes, with a larger increase expected as the period of the structure increases. For the 2015 NBC, the difference is approximately 10 to 30 percent with greater difference expected for shorter period structures. In both instances, the comparison has considered an Importance Factor of 1.5. The percentage increase in seismic demand, compared to 2020 NBC, is shown in Figure 5-1.

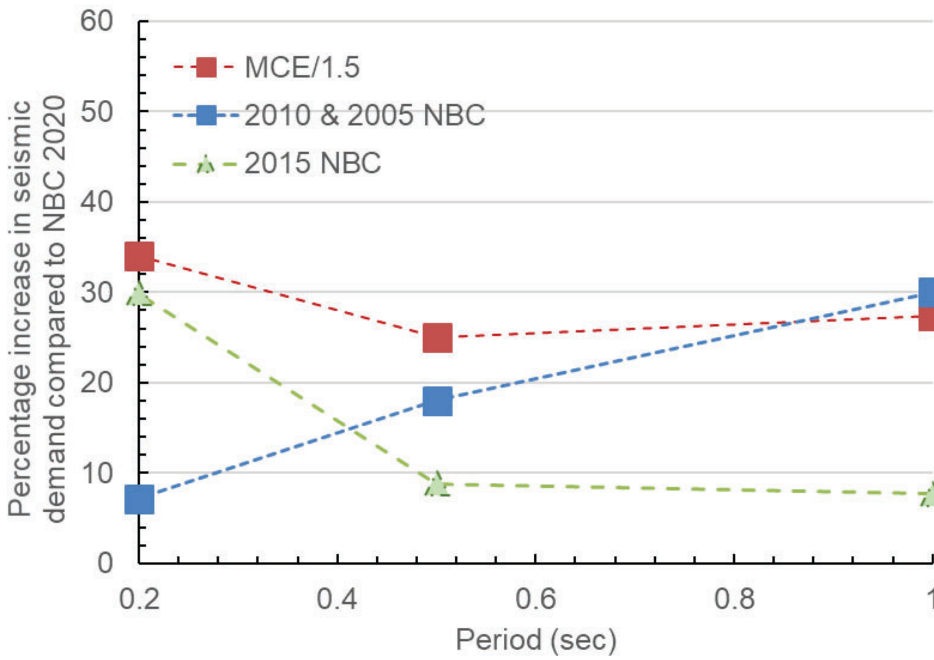

Figure 5-1: **Percentage increase in seismic demand in 2020 NBC compared to MCE/1.5, 2010 & 2005 NBC and 2015 NBC**

Although the seismic demand considered for the MCE event exceeds the seismic hazard given in 2020 NBC, all structures were retrofitted targeting an Importance Factor of 1.0. Therefore, to illustrate the expected increase in seismic demand with an importance Factor of 1.5, as required by newer codes for post-disaster structures, the demand from the MCE event was divided by 1.5 in Figure 5-1. This shows that structures retrofitted for the MCE event, generally prior to 2003, are under designed by approximately 25 to 30 percent as compared to the seismic demands of the 2020 NBC. The difference in seismic demand depends on the fundamental period of the structure. The period for each structure is not known but is likely to be in the short period range for buried concrete structures. A longer fundamental period is expected for above ground tanks. Also, the period of the structure is likely to be impacted by the retrofitting measures. For example, the analysis undertaken by Sandwell (2006) for the Kennedy Park reservoir indicated a period of 0.35 seconds before retrofitting, which was estimated to decrease to about 0.2 seconds after constructing an external shear wall.

Note that Figure 5-1 shows the increase in seismic demand for the Burnaby Mountain area. Appendix A shows similar plots for several other locations within the Greater Vancouver area.

5.4.3.2 *CHANGE TO SEISMIC LATERAL EARTH PRESSURES*

According to current design approaches, the seismic lateral earth pressure depends on the PGA. Except for the structures designed or retrofitted as per the 2015 NBC, the PGA is approximately similar to that from 2020 NBC; therefore, significant changes in seismic lateral earth design pressure is not expected. Compared to the 2015 NBC, PGA is expected to increase by approximately by 35 percent in the 2020 NBC. Using the Mononobe-Okabe method (1926), the corresponding increase in seismic lateral earth pressure is about 35 percent. Note that the increase in seismic lateral earth pressure is not necessarily proportional to the increase in PGA. The lateral earth pressure will depend on many factors including the ability of the structure to deform or/and rock. An increased seismic lateral earth pressure can also increase the deformation of the structure; thereby, the net increase in seismic demand may not be significant. Estimation of seismic lateral earth pressures is a controversial topic within the geotechnical community and there is no consensus on the appropriate methodology. However, it is important to point out that there are not many reported buried concrete reservoir or pump stations that have been damaged from seismic lateral earth pressures even when some of them have not been designed to support such magnitudes of loading.

In addition to the lateral pressures, the hydrodynamic forces are also likely to increase with the increased seismic accelerations in the 2020 NBC, although its impact is considered minor.

The potential increase in seismic demands with the adaptation of 2020 NBC are listed in Table 5-4 for reservoirs seismically retrofitted as per different codes and Metro Vancouver's Seismic Design Criteria.

Seismic Code Revision	Seismic Return Period and Expected Performance	Importance Factor	Facilities Considered	Approx. increase in seismic Inertial demand	Approx. increase in lateral earth pressure
Metro Vancouver SDS (Rev A), 1995 NBC	Level 1: 1/10,000 (MCE)	1.0	Kersland, Vancouver Heights, Prospect, Little Mountain, Kersland, Central Park, Cape Horn	+25 to 30%	No or minor change
Metro Vancouver SDS (Rev B)	Level 1 Facilities: 1/10,000 (MCE)	1.0	Westburnco, Whalley	+25 to 30%	No or minor change
	Level 2 Facilities: 1/2,475 (2005 NBC)	1.0	Kennedy Park	**+60 to 70%**	No or minor change
2005 NBC	Post-disaster: 1/2,475	1.5	Newton	+10 to 20%	No or minor change
2010 NBC	Post-disaster: 1/2,475	1.5	Clayton, Burnaby Mountain, Hellings Tank	+10 to 20%	No or minor change
2015 NBC	Post-disaster: 1/2,475	1.5	Sunnyside, Pebble Hill, Sasamat	+10 to 30%.	+ 35%

Table 5-4: A summary of design codes and approximate increase in seismic demand compared to 2020 NBC (only for seismically retrofitted reservoirs)

s.15(1)(l) [redacted]

5.4.3.3 *TARGETED SEISMIC PERFORMANCE*

The seismic demand alone will not provide a true indication of the damage if the targeted seismic performance is different (see Figure 5-2). For example, both s.15(1)(l) [redacted] were seismically retrofitted to the MCE event with an Importance Factor of 1.0, although they were assigned Moderate and Extensive Damage Levels, respectively. This is because of the differences in seismic performances targeted for these two structures. Another example is the Westburnco and Kersland reservoirs. According to Engineering & Construction (2004), Westburnco reservoir was upgraded "*to withstand a 1:10,000-year MCE event with "little or no repairable damage*". In comparison, the targeted seismic performance for Kersland reservoir was stated as "*possible localized failure of the south wall, or local roof collapse in an MCE event would not be catastrophic from a life safety perspective and the remaining in-ground portion of the "reservoir" would still retain approximately 50% of its original capacity*" (Sherstobitoff and Nikolic-Brzev, 1999).

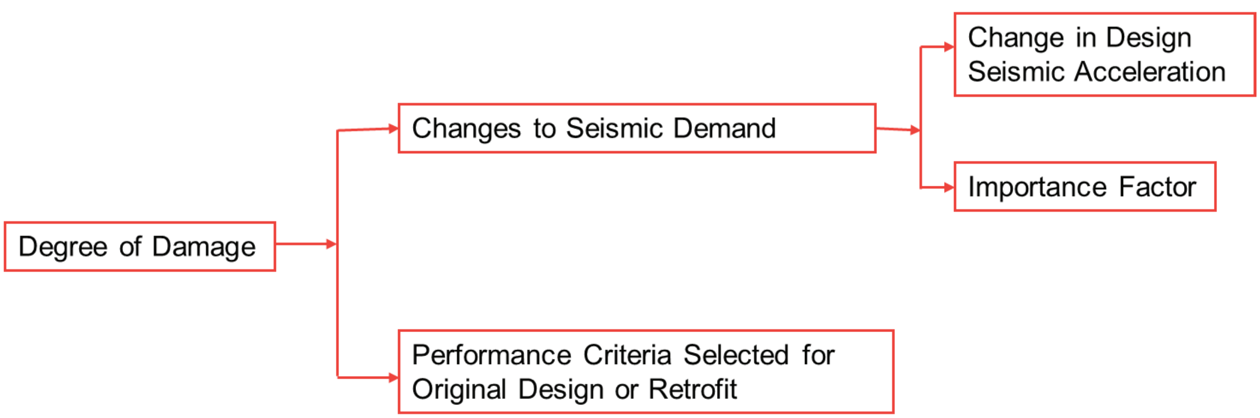

Figure 5-2: **Factors impacting the Degree of Damage**

In this regional-scale study, we were not able to identify the seismic performance objectives of all structures. However, Table 5-5 summarizes the targeted seismic performances and relevant references. In modern design codes, the damage and serviceability levels such as "post-disaster", "repairable damage" and "life-safety" are more elaborately defined (and quantifiable to a certain degree) as compared to earlier references to similar terms. There is greater subjectivity involve in the definition of certain damage/serviceability levels.

In some instances, it appears that the actual seismic performance achieved from a certain seismic upgrade is more robust than the minimum seismic performance level mandated in the Metro Vancouver's Seismic Design Criteria or applicable design code. In such situations, the degree of damage may be overestimated. To estimate the true seismic vulnerability, a more detailed assessment is required where the Capacity/Demand ratio is estimated based on the actual structural details.

Also, it is possible that actual retrofitting measures undertaken for a given structure may be different to that recommended in the design report depending on funding availability. Our seismic assessment dependant on the information contained in the design reports.

Reservoir	Seismic performance achieved or targeted for the MCE event	Reference	Damage Level assigned in this study
Westburnco	s.15(1)(l) [redacted]	Ayalp et al (2006)	s.15(1)(l) [redacted]
Kersland	[redacted]	Sherstobitoff and Nikolic-Brzev (1999)	[redacted]
Vancouver Heights	[redacted]	Sherstobitoff and Nikolic-Brzev (1999).	[redacted]
Little Mountain	[redacted]	Sukumar et al. (2004)	[redacted]
Whalley	[redacted]	Engineering & Construction (2004)	[redacted]
Cape Horn	[redacted]	Sandwell (2001)	[redacted]
Kennedy Park	[redacted]	Sandwell (2006)	[redacted]
Central Park	[redacted]	Sandwell (1999)	[redacted]

Table 5-5: Descriptions of seismic performance levels achieved or targeted during seismic retrofitting

5.4.3.4 *POSSIBILITY OF GROUND FAILURE*

The methods available to assess the likelihood of liquefaction has evolved over the last few decades. Most notably, new methods have been developed to assess the likelihood of cyclic softening/failure of "clay-like" soils. For example, Boulanger and Idriss (2006) proposed a method to quantify the cyclic resistance of a clay-like soil which allowed to compare it against the cyclic demand. Prior to this approach, the "susceptibility" for cyclic softening was evaluated using Index tests such as water content, Liquid Limit and Plasticity Index (Bray and Sancio, 2007; Seed et al., 2003). However, in the absence of cyclic demand, the "potential" to cyclic softening could not be evaluated.

Our review has indicated that cyclic softening potential at several facilities was evaluated based on simplified methods. For example, Golder (2005) considered the Chinese Criteria (Wang, 1979) for the evaluation of cyclic softening potential of the soft to firm clayey silt to silty clay deposit below Grandview pump station, which was consistent with the state of practice at that time. However, under high seismic shaking intensities expected in the 2020 NBC, cyclic shear stresses could exceed the shear strength of the soft to firm clayey silt to silty layer.

Similarly, for the Barnston/Maple Ridge pump station, Golder (2011) considered clayey silts/silty clays with a Plasticity Index greater than 7 as not susceptible to liquefaction. However, sensitive clay-like soils can also experience drastic reduction in strength and stiffness if the seismic demand exceeds the cyclic resistance. Our approximate calculations indicated a factor of safety of about 1.0 to 1.1 against cyclic softening in certain layers based on the seismic demands expected in the 2020 NBC and using the approach recommended by Boulanger and Idriss (2006) for clay-like soils. A detailed site-specific evaluation would be required at these sites to evaluate the cyclic softening potential of clay-like soils and its impact on the foundations.

More recent seismic assessments conducted by Metro Vancouver have identified bearing capacity issues s.15(1)(l) [redacted] capacity issues have been identified only in these two reservoirs while such issues were not encountered for other similar structures. It is possible that the factored bearing capacity may have been underestimated. Further analysis would be required to confirm such deficiencies. If such deficiencies exist, it will be difficult and expensive to retrofit the structure.

5.5 SUMMARY

- For all reservoirs and pump stations, the seismic demand from the 2020 NBC (for a post-disaster structure) is likely to exceed the demand considered for the design or seismic retrofitting. If the structure was retrofitted or designed based on the 2005 or 2010 NBC, this increase is marginal to moderate (about 10 to 20%). Considering the built-in factor of safety against post-disaster structures (i.e., Importance Factor of 1.5), the expected damage is likely to range from minor to moderate. For structures constructed or upgraded as per the 2015 NBC, a considerable increase in seismic lateral earth pressures and seismic inertial forces are expected, which may cause extensive damage to the structure.
- Some of the key reservoirs (i.e., Level 1) upgraded prior to 2003 may experience extensive damage despite being designed to the MCE earthquake with a return period of 1 in 10,000. This is partly due to the targeted seismic performance considered for seismic retrofits, which is less stringent than that of a “post-disaster” structure. There is some uncertainty related to the seismic accelerations considered for the MCE event since later studies have indicated much greater values for the 1/10,000-year return period event.
- s.15(1)(l) [redacted]
- Despite the seismic deficiencies, the reservoirs and pumps stations are likely to perform better compared to the findings of the 1993 EQE study, even though the design seismic demand has increased considerably. Better performance is attributed to the series of seismic retrofitting undertaken by Metro Vancouver since the 1993 EQE study. As expected, structures that are not retrofitted will experience severe damage.

- Based on observations from previous earthquakes, failure of inlet and outlet pipes is identified as the most likely failure mode that can impact the post-earthquake serviceability of a reservoir. This may occur even if the structure remains undamaged. It is unclear if special measures have been undertaken to improve the resilience of these connections during seismic retrofitting that primarily targeted the concrete structure. It is not possible to estimate the likelihood of damage of inlet/outlet pipes without conducting a site-specific assessment.
- The seismic vulnerability was evaluated by comparing the minimum seismic performance targets/demands considered for the design or seismic retrofitting against those expected from the 2020 NBC for a post-disaster structure. However, it is possible that the actual design or retrofit measures are more robust and may exceed the minimum requirements stipulated in Metro Vancouver's Seismic Design Standards or applicable codes of the day. Therefore, the actual damage could be less severe than that listed in Tables 13 and 14. Note that similar structures have performed satisfactorily in past earthquakes especially if they are not located in ground prone to soil liquefaction and permanent ground movements. For a more accurate estimate of the damage level, a detailed site-specific assessment is required.
- Most of the anticipated failures are associated with structural deficiencies. Major geotechnical hazards such as soil liquefaction are not anticipated at most facilities. However, additional geotechnical analyses are required at s.15(1)(l) to evaluate the potential for cyclic softening of the underlying clay-like soils.

6 FUTURE ACTIONS AND RECOMMENDATIONS

6.1 FUTURE ACTIONS

In the following section, certain action items have been identified to improve the reliability of the regional-scale study.

6.1.1 FOR TRANSMISSION MAINS

- Based on the findings of this report, it is understood that Metro Vancouver will conduct hydraulic analysis to identify supply points that are most likely to be resilient to a major seismic event and to provide a basic level of service to member jurisdictions. Subsequently, we expect more detailed analysis to be conducted for higher risk pipes to better predict the extent of damage. PSI analysis undertaken for this study should only be relied for this regional-level assessment.
- Our study has flagged several water or bridge crossings which were assigned a low or high vulnerability rating based on the available information. AECOM (2021) has already conducted a detailed condition assessment of these water crossings. We recommend this study be expanded to assess the seismic vulnerabilities of these crossings. Site-specific assessments have already been undertaken for some these crossings; specially the vulnerability of major river crossings is already known and quantified. Nonetheless, there are uncertainties related to the vulnerability of watermains that cross shallower rivers, streams and creeks (Category 2) where the extent and consequence of damage is not as significant as that of a major river crossing. Given the number of Category 2 crossings, we do not expect site-specific seismic assessments to be conducted for each crossing; instead, it may be feasible to categorize crossings into certain groups based on common characteristics and analyze their vulnerability in a more generalized manner. A more comprehensive assessment will also allow approaches and recommendations to be developed for remediation and/or post-earthquake recovery strategies for vulnerable crossings. We have categorized the crossings into four groups mainly based on the available O&M drawings and site photographs. A more detailed assessment which includes a site visit and more detailed review of ground conditions may change the category of some of these crossings.

6.1.2 FACILITIES

- As stated in Section 5.2.3, one of the most common and critical failure modes of a reservoir is the failure of the outlet and inlet pipes at their connections. The likelihood of damage is difficult to estimate from a desktop review. We recommend this to be evaluated in a separate study in which the inlet/outlet pipes and their connections are reviewed to assess their vulnerability. Special attention should be given to pipes that are difficult to access and those lacking isolation devices.
- We expect the seismic demand to increase in all facilities with the release of 2020 NBC compared to those were considered for seismic retrofitting or design. Additional structural analysis is recommended to estimate the seismic vulnerabilities of these structures due to this increase in seismic demand. Priority should be given to critical reservoirs (identified as Level 1 reservoir) which were retrofitted in the 1990's and early 2000s. Pump stations and reservoirs with structural irregularities can also be included in this initial round of assessment since such irregularities are restricted in current design codes for post disaster structures unless detailed analyses are undertaken to demonstrate their feasibility. The initial round of structural analysis may target only a few structures to gain an understanding of the extent of the damage and to verify the predictions made in this regional-scale study. The decision to expand the study to other facilities will be influenced by the findings of the initial assessment.

- Besides the structural vulnerabilities, there are several “non-structural “vulnerabilities that can impact the seismic performance. Commonly a site walkthrough is conducted to review the condition of non-structural components and verify the details shown on drawings. The intent of the walkthrough is to identify components vulnerable to seismic loading. Additional seismic design details related to non-structural components are also given in CSA S832 – Seismic Risk Reduction of Operational and Functional Components (OFCs) of Buildings. The checklists included in Heubach (2002) may be considered for checking the seismic deficiencies during a site walkthrough. A site walkthrough is also recommended because a desktop review cannot identify the current conditions of a facility. For example, deterioration of a given component will increase its seismic vulnerability and, in concrete tanks, cracks that appear to show corrosion stains can be an indicator of significant structural degradation. Once the vulnerable components have been identified, the consequence of various failure modes should be determined. In some instances, remediation measures may be obvious while in other cases, detailed assessments are required to quantify the likelihood of failure and consequence. We understand that non-structural upgrades have been already completed for some reservoirs and pump stations.
- s.15(1)(l) [redacted]
- As stated in Section 5.2.4, additional geotechnical assessments are recommended for Grandview and Barnston/Maple Ridge pump stations to assess concerns related to potential cyclic softening of clay-like layers and its impact to the foundations. If deficiencies exist, it will be difficult and expensive to retrofit the foundations.
- Besides those facilities considered in this assessment, there may be several other ancillary facilities that are critical to the overall seismic performance of Metro Vancouver water supply system. For example, the recovery of the water distribution system was severely hampered during the 1985 Mexico City, 1991 Philippines, 1995 Kobe and 2001 Nisqually earthquakes because administration buildings used to house water utility staff and store engineering records were damaged and inaccessible after these earthquakes (Heubach, 2002). We recommend such buildings to be identified and included in future seismic vulnerability assessments.

6.2 MEASURES TO INCREASE SEISMIC RESILIENCY

6.2.1 PRIORITIZATION STRATEGY

Developing a seismic resilient water distribution network may take decades through the normal asset renewal process. In the interim, a prioritization strategy can be developed to identify the facilities and watermains critical to meeting the overall performance requirements. This may include maintaining a robust backbone system, which can supply the key health and safety related needs to communities shortly after a seismic event, while more extensive repairs and upgrades are being implemented on the remainder of the system. The opinions on prioritization strategy for a water supply network can differ and require input from different stakeholders. For example, some have recommended upgrades be performed based on pressure zones such that if an earthquake occurs before the upgrades are completed, at least the upgraded pressure zones will remain functional. However, this strategy may severely compromise the performance of remaining pressure zones.

In general, it is expected that the financial burden associated with a capital investment to upgrade all components of the entire pipe system would overwhelm short-term capital improvement budgets. For instance, it is difficult for a municipality to justify upgrades solely based on earthquake resilience considerations when they face many other serviceability needs. Life-safety issues should be given the highest priority and should be addressed first. Afterwards, the seismic upgrades are typically prioritized according to the overall improvement to the post-earthquake system performance and cost effectiveness. Capital improvement programs and maintenance issues should also be considered during prioritization. It is often cost-effective to integrate seismic retrofitting with capital improvement and maintenance programs. For example, if a component is scheduled to be replaced, performing expensive seismic upgrades to this component prior to its replacement may not be cost-effective.

Detailed development of a prioritisation strategy is outside the scope of this study. We anticipate that the findings of the hydraulic modeling will be a key factor in the prioritization strategy.

6.2.2 DESIGN CONSIDERATIONS

Typically, the incremental cost of designing and constructing for a higher seismic demand is generally a small percentage of the total project costs when new installations, capacity upgrades and asset renewals are involved (NIST 2012). This situation may arise due to the release of the new 2020 NBC and its requirements. Although the higher seismic demands given in the 2020 NBC may not be adopted before the next version of BC Building Code, Metro Vancouver may consider mandating designers to adopt the higher seismic demands to reduce the need for future seismic retrofits to new facilities and transmission mains designed under the current code.

Over the years, several best-practice recommendations have been proposed by various agencies and researchers to improve the seismic resiliency of different components. We understand that some of the standard design details were developed prior these developments. It is recommended that a review of these standard drawings be completed to identify modifications that would improve the seismic resiliency of components (e.g., detailing of outlet/inlet pipe connections to reservoirs).

Replacement of vulnerable pipe sections with more ductile pipes and connections may improve the post-seismic performance depending on the magnitude and direction of ground displacement. However, replacing the entire pipe may be cost prohibitive and unnecessary in relation to the overall gain. In certain circumstances, inclusion of earthquake resistant joints at strategic locations of the pipe may be sufficient to protect the entire pipe from damage. These connections will improve the flexibility of the pipe and its ability to accommodate larger ground displacements. For example, following the 1994 Northridge earthquake, Valencia Water Company installed flexible expansion joints at many bridge crossings to improve their displacement capacity (Abercrombie, 2013).

With the construction of five major river crossings (out of which three are in design phases), the key concerns related to most of the Greater Vancouver areas will be addressed. However, we understand that there are a few areas that require additional robust water crossings to meet the needs of the localities (e.g., s.15(1)(l) [redacted]. For these water crossings, the greatest challenge is to design and install the crossings to withstand the forces from permanent ground displacements. This is more critical in instances where deep ground displacements have been predicted using numerical modeling, sometimes in non-liquefiable soils. To eliminate the impact from these ground displacements, deep tunnel profiles have been adopted along with deep shafts at either end. As a result, these seismically resilient major river crossings tend to be very expensive. Deep liquefaction and the associated ground displacements are controversial topics with differing opinions. For example, Prof. Leslie Youd, who is considered as a prominent researcher on lateral spreading and the developer of the Youd et al (2002, 2009) method, recommended pipes be buried two times below the free face height, measured from the ground surface, to limit the impact from lateral spreading regardless of the liquefaction status (see Figure 6-1). In some instances, this will result in a much shallower pipe profile and a significantly less expensive water crossing. Considering the potential cost implications related to the deep tunnel profile, the validity of numerical model predictions should be investigated further with assistance from relevant subject matter experts. If shallower pipe profiles are found to be feasible at these crossings, the savings could be utilized to upgrade more water crossings and/or introduce redundancy to the network.

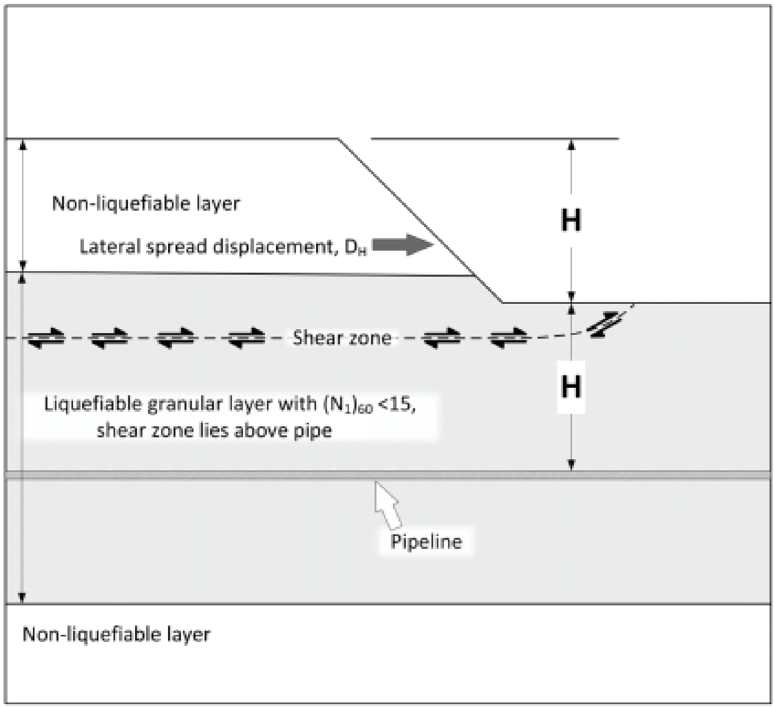

Figure 6-1: **Maximum depth of lateral spreading (adopted from Youd, 2018).**

The design of facilities should also consider the ease and speed of undertaking repairs after an earthquake. For example, in water treatment plants, drains and pumps should be sized to allow rapid dewatering of the units, and chemical feed equipment and lines should be accessible for inspection and repair (US Environmental Protection Agency, 1974). The components that are intentionally designed to be weak, in order to protect other components, should be easily accessible after an earthquake.

6.2.3 SEISMIC RESILIENCY MEASURES

Apart from undertaking seismic upgrades of facilities and transmission mains, there are several other strategies that should be considered and incorporated into a resiliency plan. As stated earlier, a seismic resiliency plan will include both long and near term mitigation measures. The near term mitigation measures are expected to include less expensive strategies such as improvement of emergency preparedness and response procedures, and implementation of post-earthquake operation and control strategies.

This includes adding isolation devices and controls to the system such that reservoirs do not completely drain empty if there is damage. For example, during the 1995 Kobe earthquake, shutoff valves have been installed in 21 distribution reservoirs (out of 122) if the acceleration exceeds 0.25g (Matsushita 1995). Kuraoka and Rainer (1996) reported that 18 of these valves were activated by the earthquake, saving approximately 33,000 m^3 of water. Water in the remaining 101 distribution reservoirs was drained within a short time due to extensive damages to the distribution and service pipes. Los Angeles Department of Water and Power (LADWP) and Seattle Public Utilities (SPU) have initiated programs to incorporate shutoff valves at strategic locations to isolate damaged pipelines. We understand that SPU has already installed seismic isolation valves in more than half of their newer reservoirs. Shutoff valves should be located outside the areas likely to experience PGD and equipped with sufficiently sized pipe/valve outlets to install temporary bypasses, if needed. The main concern related to the use of automatic shutoff valves is cutting off water needed for fire fighting. Therefore, the isolation strategy should be discussed with the local fire department and municipality. Valves which are earthquake-actuated or manually or remotely controlled, can be used to isolate areas or divert flows. Use of electrically operated valves should be carefully reviewed due to concerns related to the interruption to the power supply.

Developing an emergency preparedness and response plan is a key component of the seismic resiliency strategy. This will identify the critical materials and suppliers to repair components that are damaged from the seismic event. If problems are anticipated in the supply chain immediately after a major earthquake, spare parts should be stockpiled at strategic locations. A majority of transmission pipes are located at deeper depths and repairs may require shoring and dewatering. The availability of shoring, heavy machinary and skilled labour to repair such pipes could become a concern after an earthquake and should be considered in the plan. Further, it is best practice to use standard components and designs as much as possible for new constructions and minimize the use of specialized components and sizes which may not be readily available during an emergency response. In addition, construction drawings and location plans should ideally show the components that were intentionally designed to be weak to protect the rest of the network.

Incorporating sufficient redundancy in the network is another key method of improving the overall seismic resiliency. Further details related to redundancy in the system should be evaluated in subsequent phases of the project depending on the results of hydraulic modeling.

The items described above are not intended to represent a comprehensive list of measures that could be adopted by Metro Vancouver to improve the seismic resiliency. If requested, more details will be given in a separate phase of the project.

6.2.4 SEISMIC RESILIENCY OF DISTRIBUTION NETWORK

The current seismic assessment includes Metro Vancouver's transmission mains only. However, past earthquakes have indicted that emergency response and post-earthquake recovery largely depends on the level of damage experienced by distribution mains and service pipes because of large number of damages expected in these pipes. Currently, there is no unifying standard for designing distribution networks for seismic events. Seismic design practices differ from one local municipality to another. At present, only a couple of municipalities have specific seismic design and performance requirements for water pipes.

The American Society of Civil Engineers (ASCE) Utility Engineering and Surveying, in collaboration with the ASCE Infrastructure Resilience Division, Task Committee on Seismic Design of Buried Water and Wastewater Pipelines is developing a Manual of Practice (MoP) for seismic resistant design of buried water and wastewater pipelines. The MoP is expected to cover transmission, interceptor, trunk, collection and distribution pipelines. Although fire hydrants and service pipes are not covered, design considerations to address these ancillary components will be included. According to Heubach et al. (2019), "*these guidelines are intended to evolve into a national standard and are intended for use throughout the United States and Canada*."

According to this MoP, utilities will be categorized into four main types based on their importance (i.e., Pipeline Criticality I to IV). This will form the basis for seismic design and material selection. For example, guidelines have been provided to identify pipe segments requiring more detailed site-specific analysis based on their criticality and the prevailing soil conditions. The performance criteria for different pipes will be established in terms of required flexibility (e.g., joint deflection for segmented pipelines), ductility (e.g., strain capacity), and strength for continuous and segmented pipes. Preliminary recommendations were included in some publications such as Wham et al. (2019). Pipe manufacturers are expected to test their products and demonstrate the ability to meet different strength and flexibility requirements. If the MoP is adopted by local municipalities, Metro Vancouver's Seismic Design Criteria may require an update to comply with the performance requirements expected from transmission mains.

7 CLOSURE

We trust the information provided in this report meets with your immediate requirements. Please contact the undersigned should you have any questions or comments regarding the information provided herein.

Prepared by:

PROFESSIONAL PROVINCE OF BRITISH COLUMBIA ENGINEER
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APPENDIX

A SEISMIC HAZARD COMPARISONS

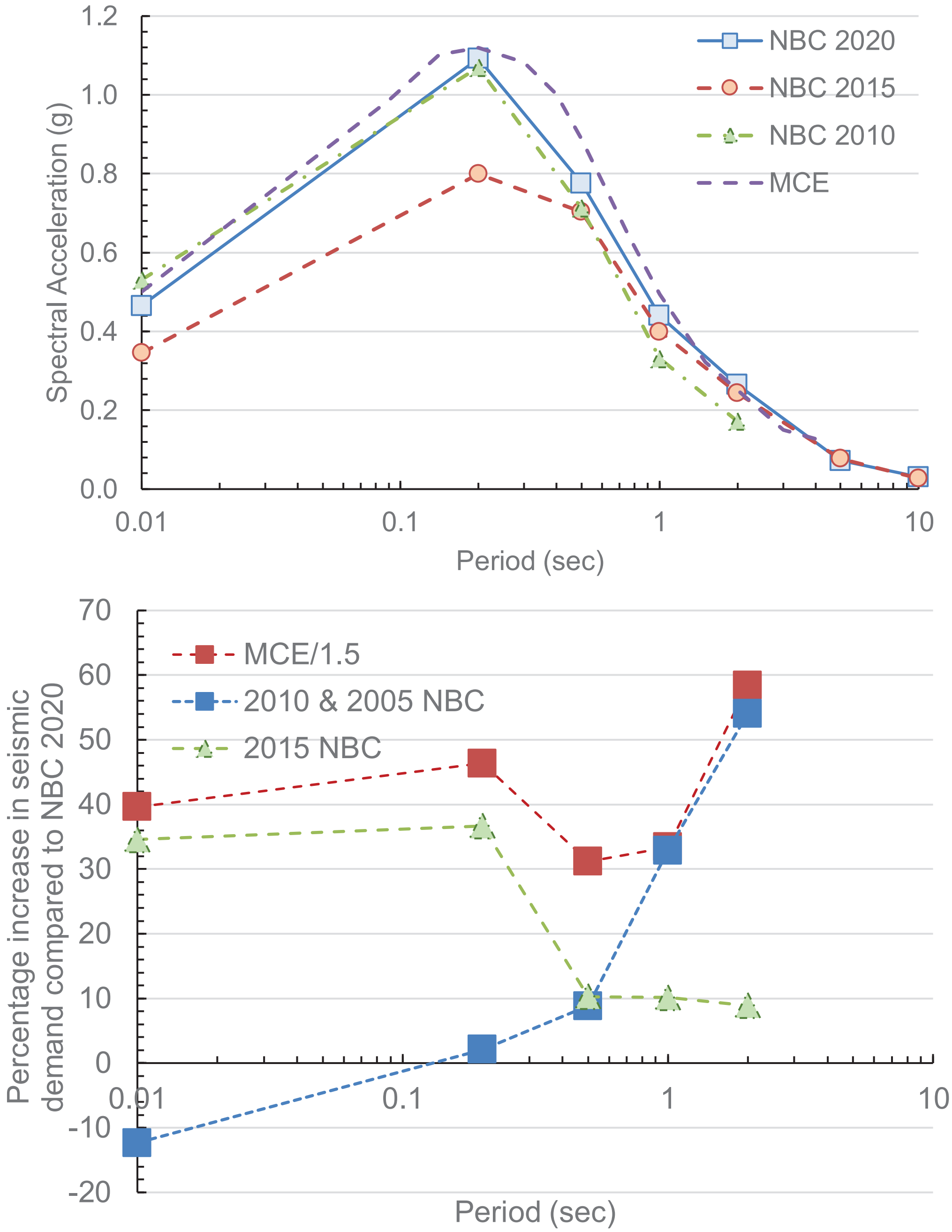


Figure A1: **Comparison of firm-ground spectral accelerations from different design codes – Cloverdale, BC**

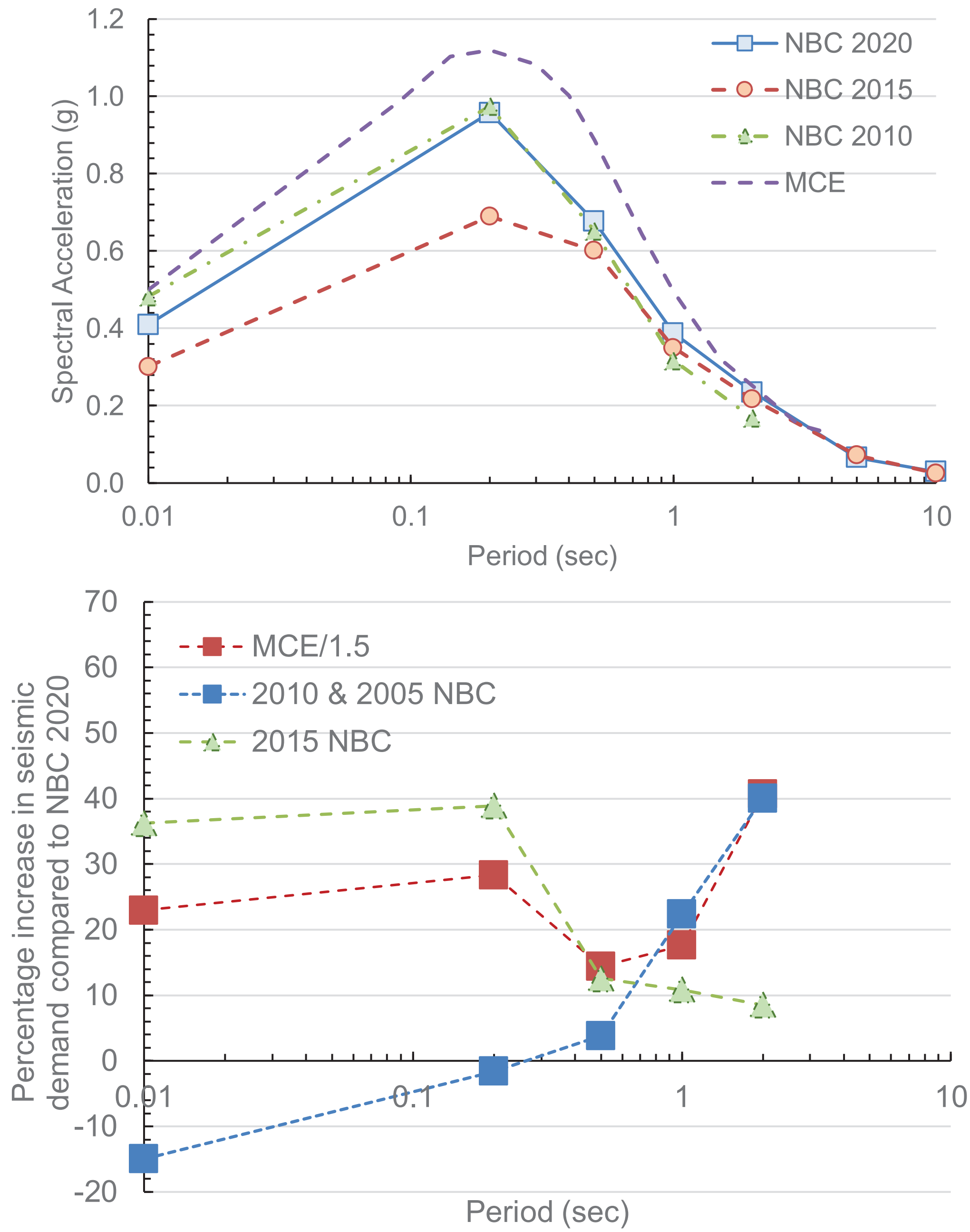


Figure A2: **Comparison of firm-ground spectral accelerations from different design codes – Haney, BC**

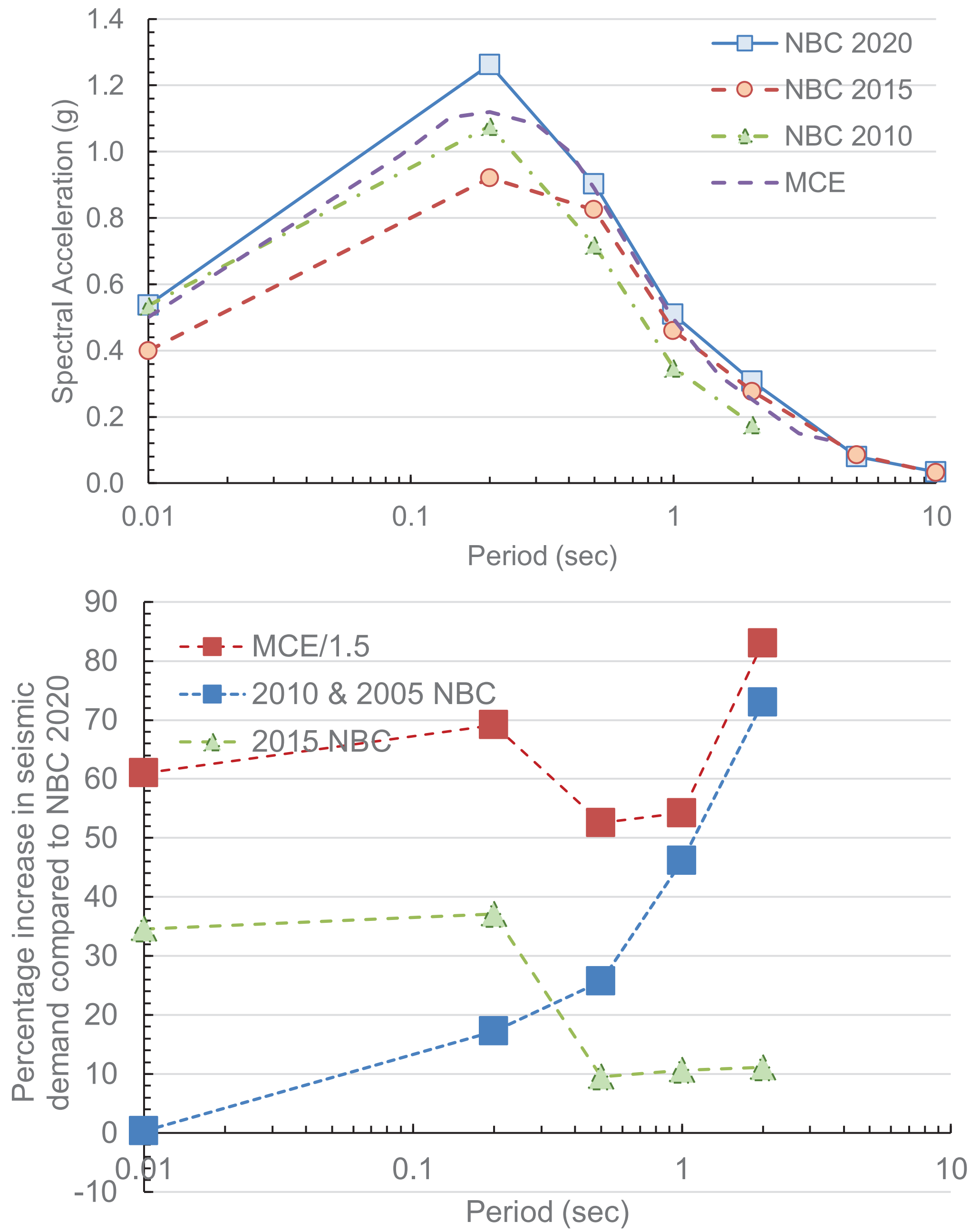
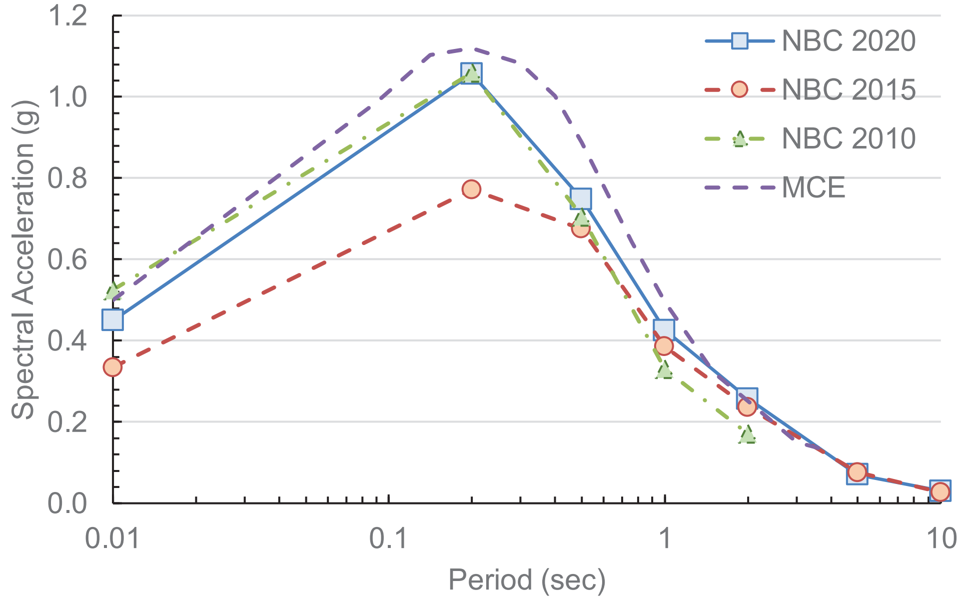


Figure A3: **Comparison of firm-ground spectral accelerations from different design codes – Ladner, BC**



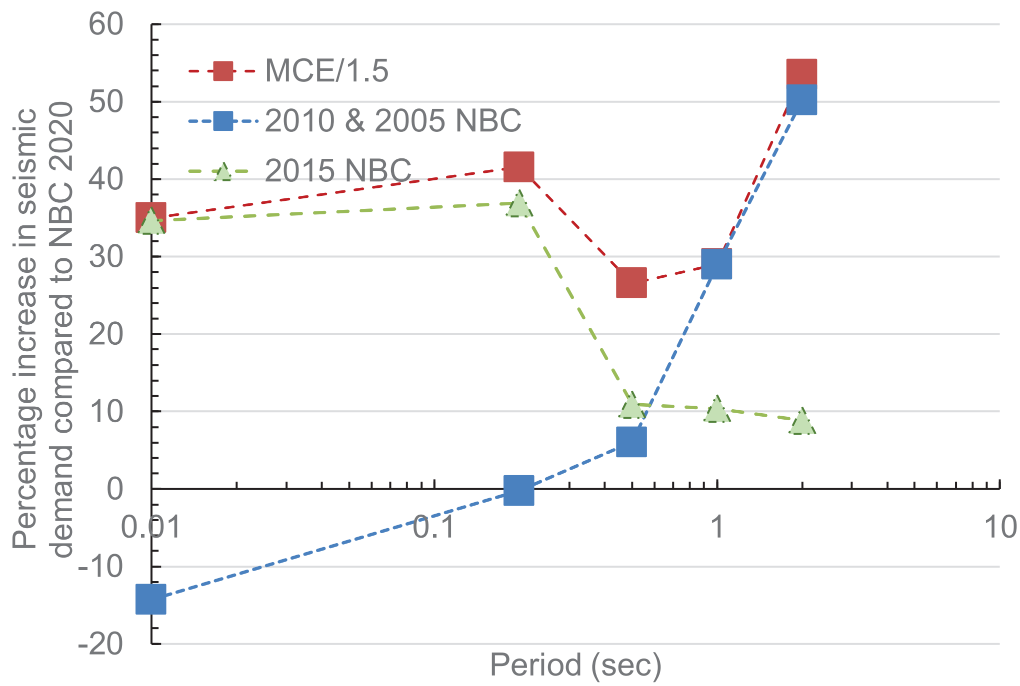


Figure A4: **Comparison of firm-ground spectral accelerations from different design codes – Langley, BC**

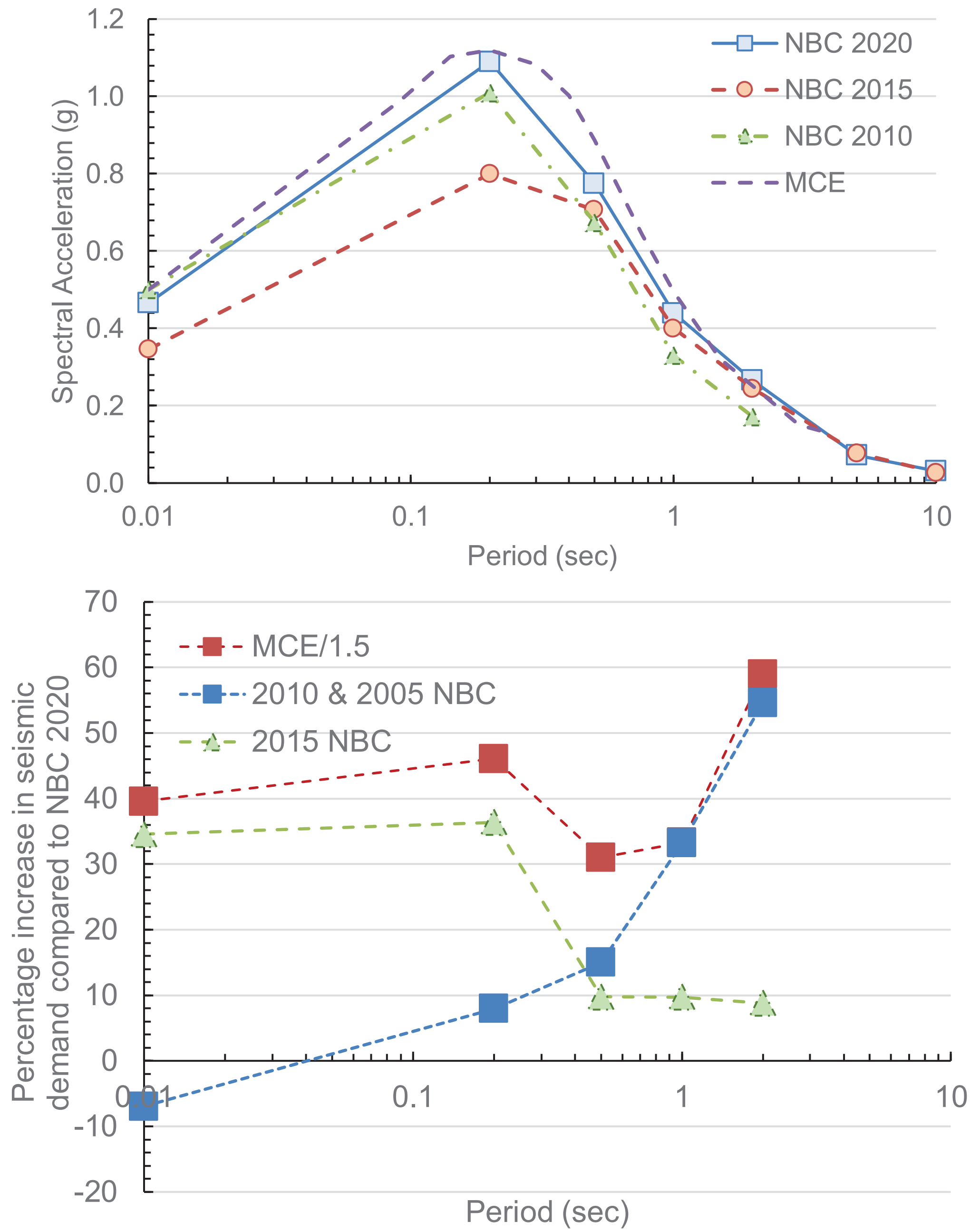


Figure A5: **Comparison of firm-ground spectral accelerations from different design codes – New Westminster, BC**

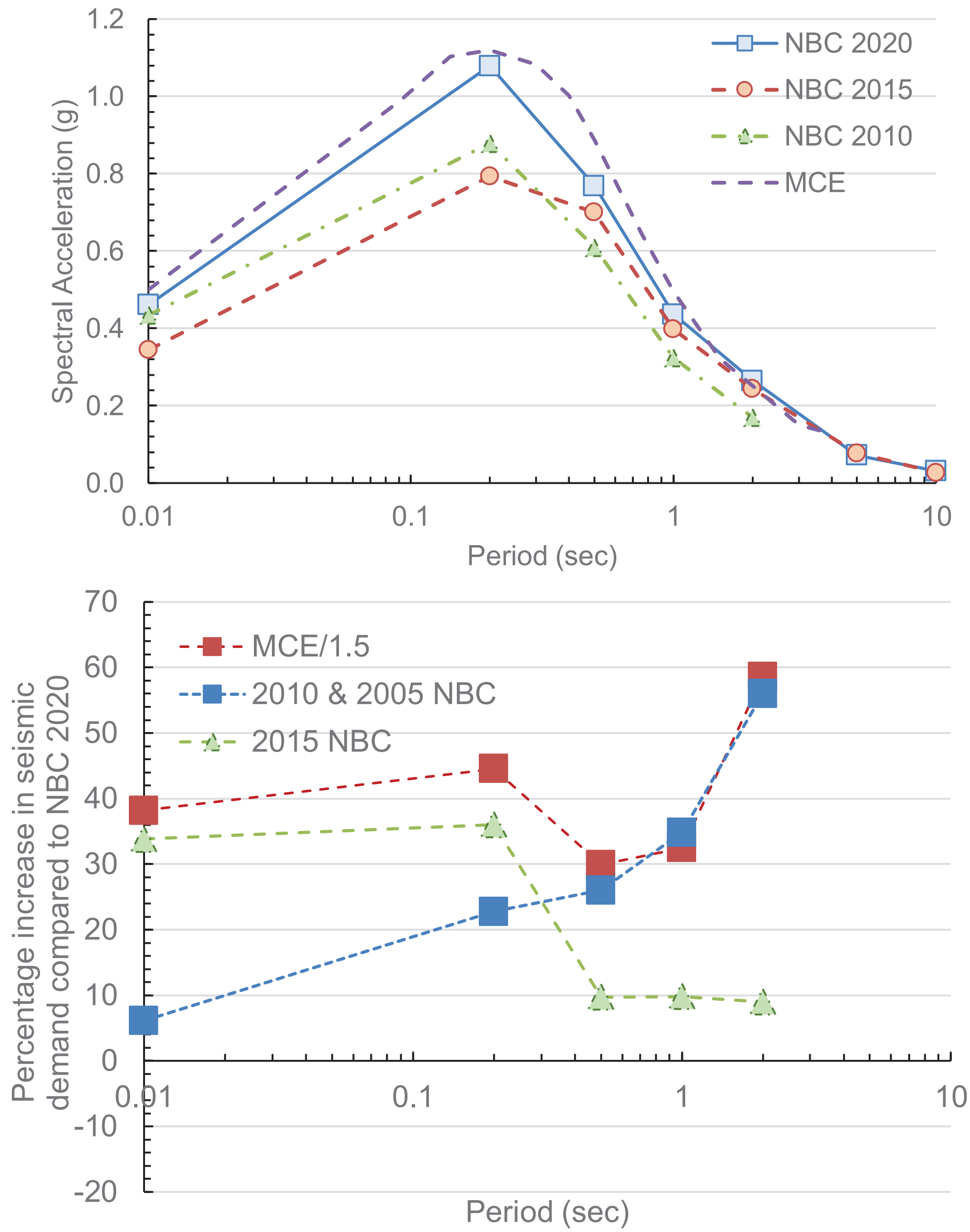


Figure A6: **Comparison of firm-ground spectral accelerations from different design codes – North Vancouver, BC**

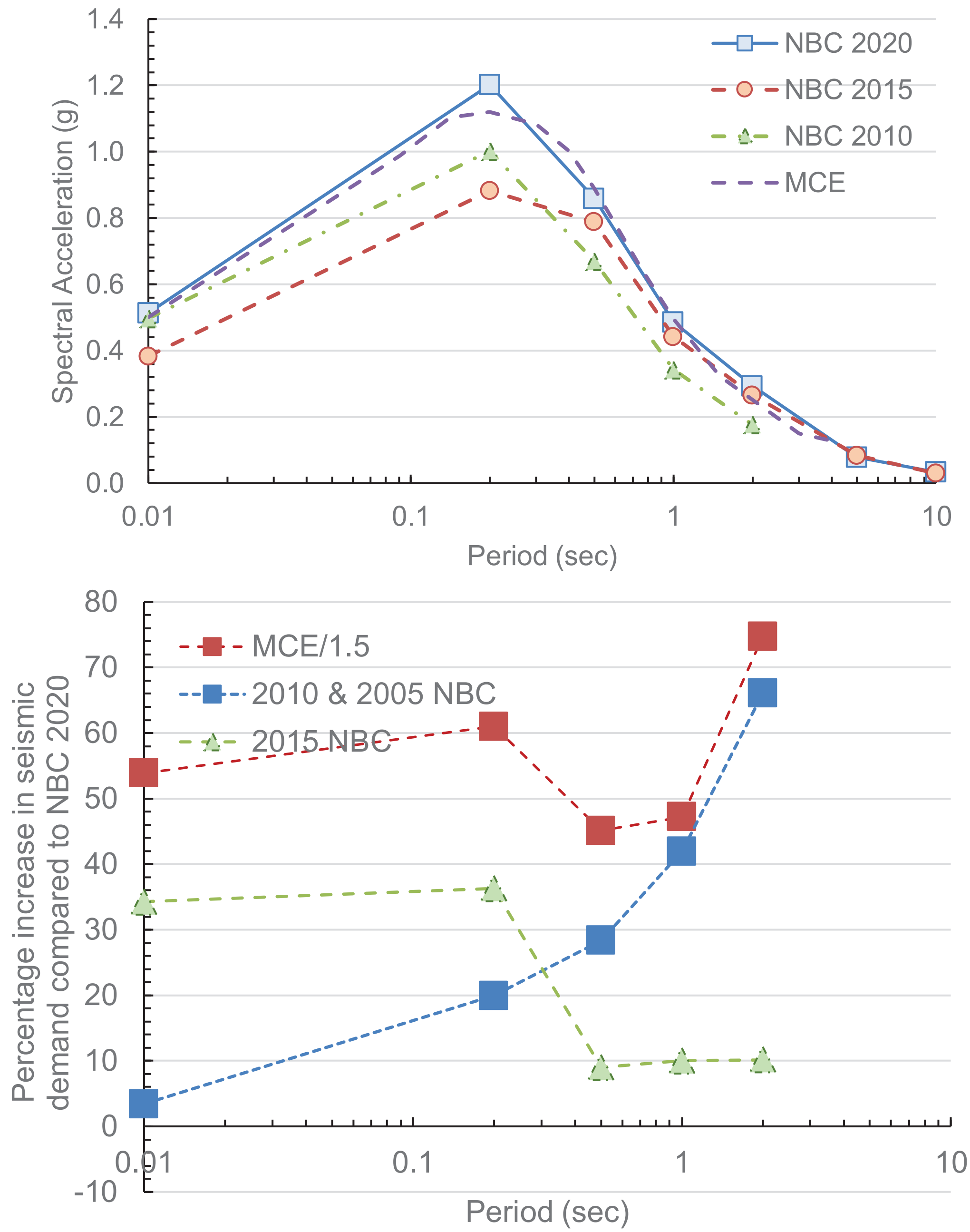


Figure A7: **Comparison of firm-ground spectral accelerations from different design codes – Richmond, BC**

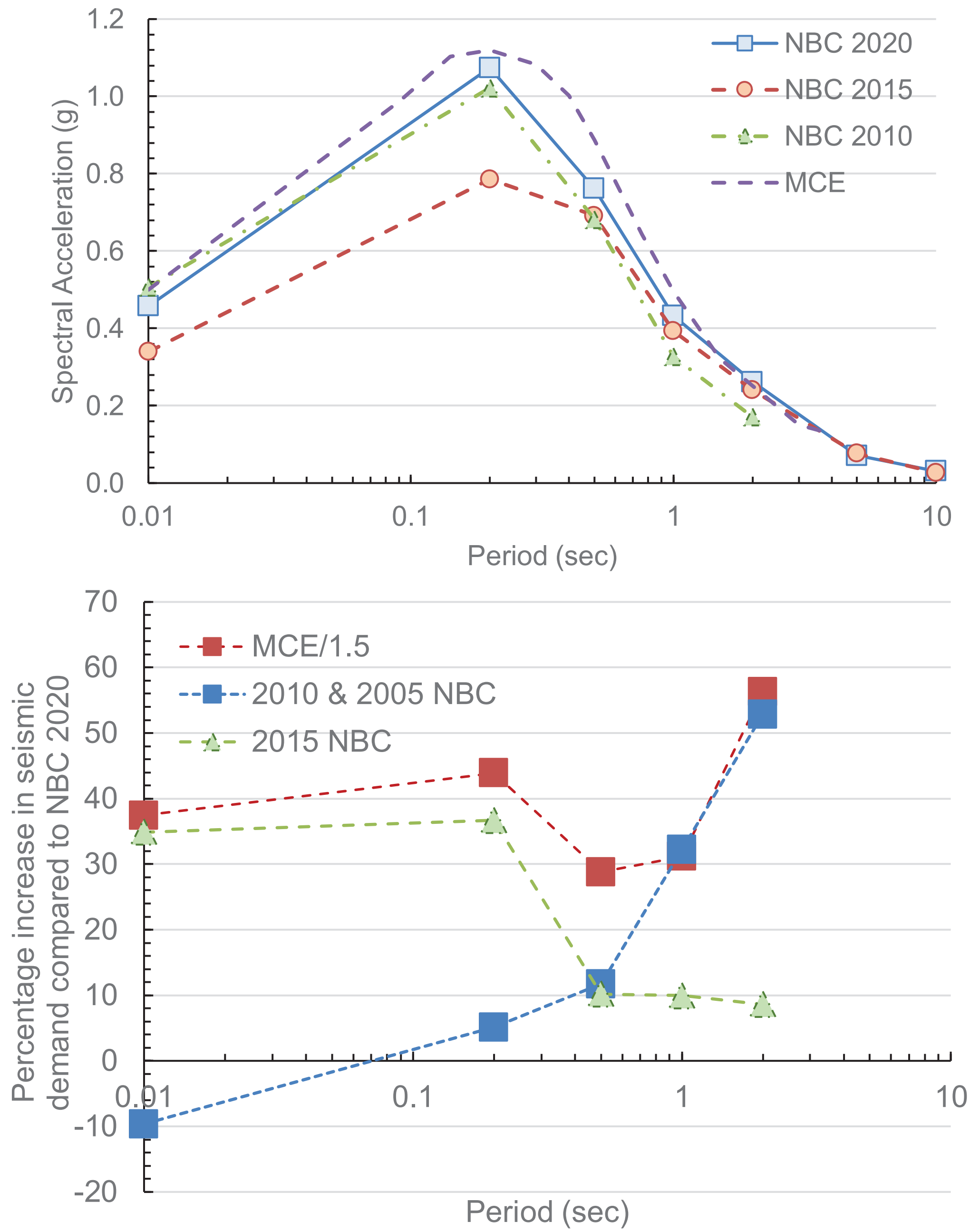


Figure A8: **Comparison of firm-ground spectral accelerations from different design codes – Surrey (88 Ave & 156 St), BC**

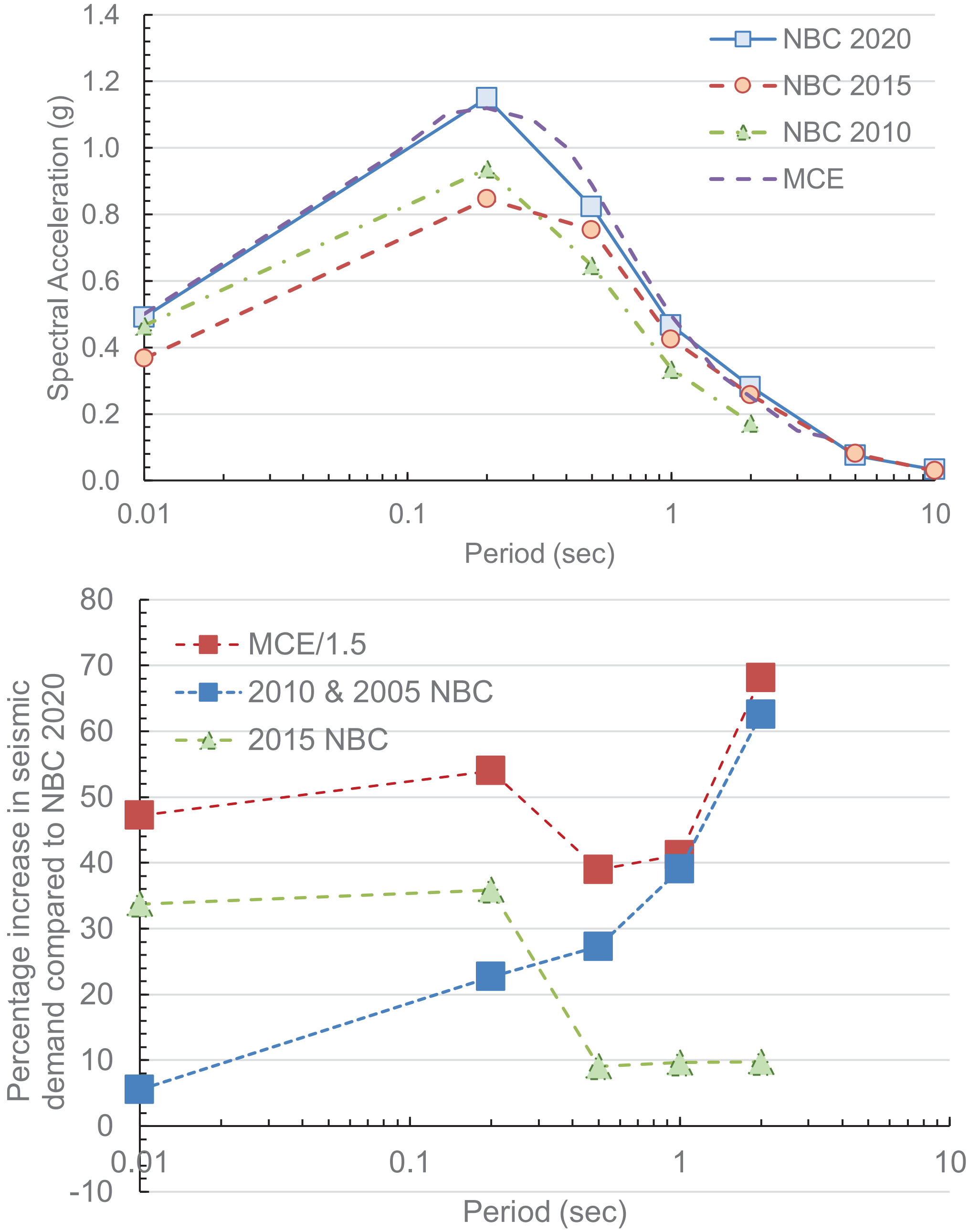


Figure A9: **Comparison of firm-ground spectral accelerations from different design codes – Vancouver (City Hall), BC**

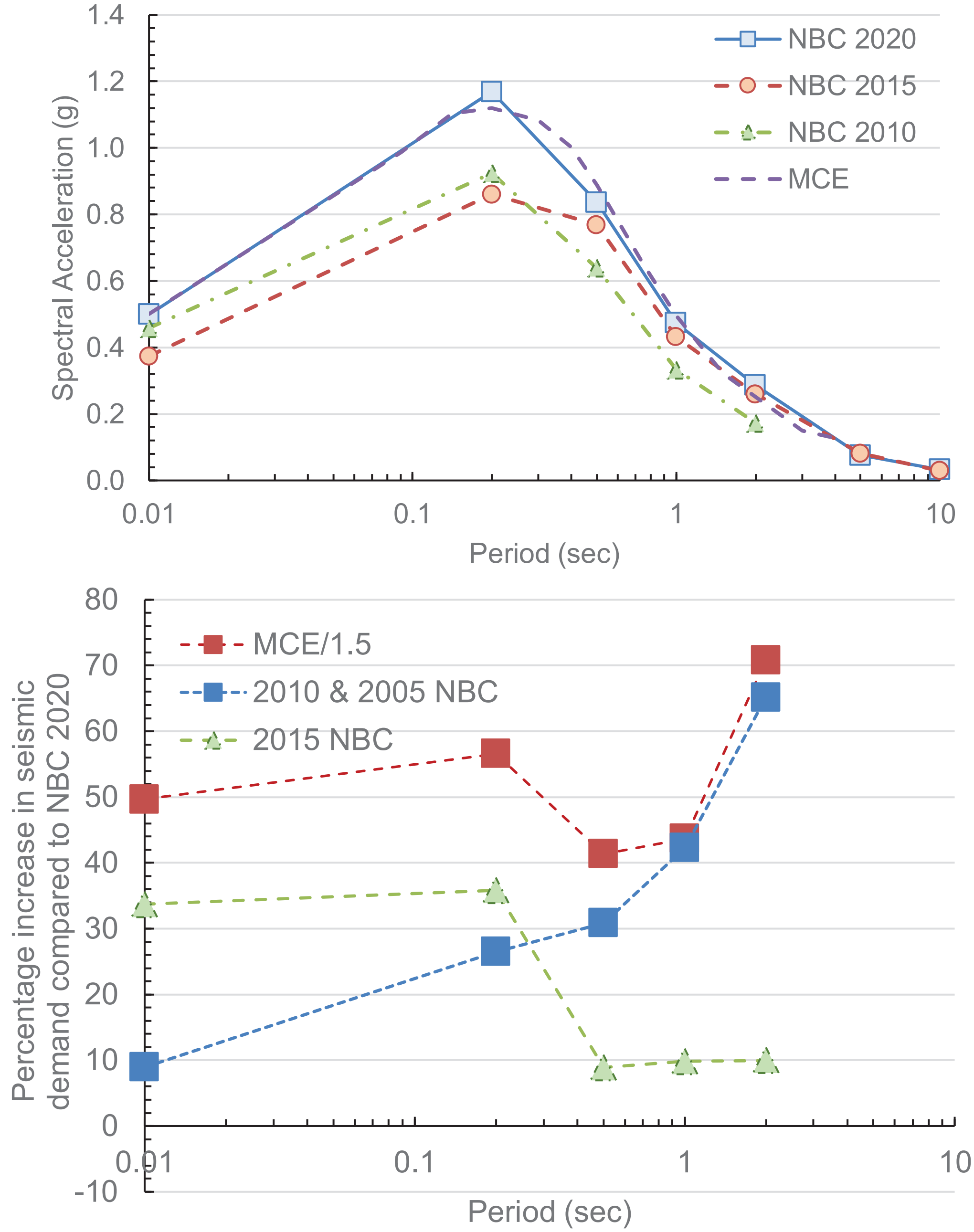


Figure A10: **Comparison of firm-ground spectral accelerations from different design codes – Vancouver (Granville & 41 Ave), BC**

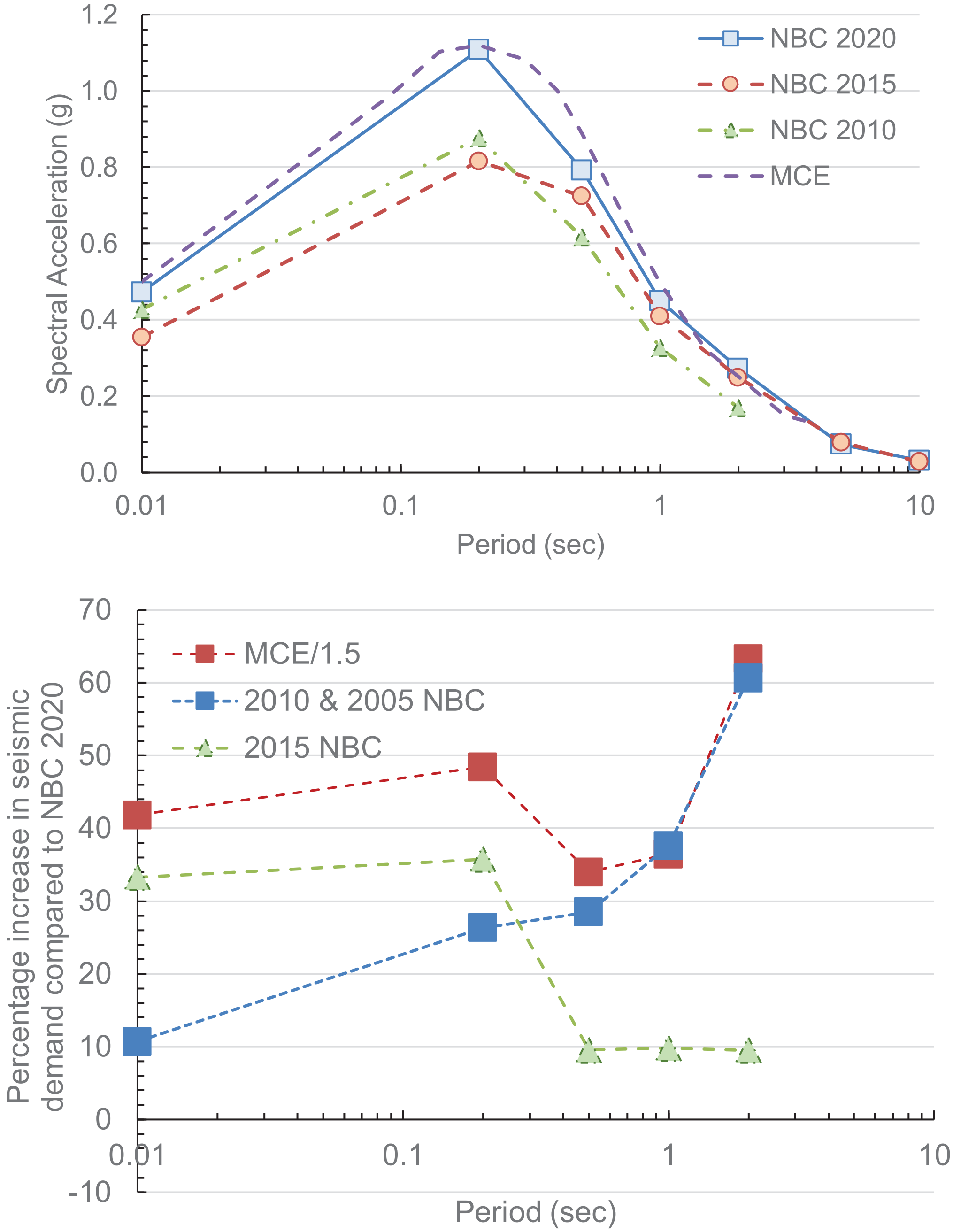


Figure A11: **Comparison of firm-ground spectral accelerations from different design codes – West Vancouver, BC**

APPENDIX

B SEISMIC HAZARD MAPS

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APPENDIX

C PIPE DAMAGE RATES

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APPENDIX

D SEISMIC EVALUATION SHEETS FOR FACILITIES

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APPENDIX

E HIGH-LEVEL STRUCTURAL ASSESSMENT (THREE RESERVOIRS)

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APPENDIX

F

SEISMIC VULNERABILITY MAPS

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APPENDIX

G TERMS OF REFERENCE

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5 INTERPRETATION OF THE REPORT

a. **Nature and Exactness of Descriptions**: The classification and identification of soils, rocks and geological units, as well as engineering assessments and estimates have been based on investigations performed in accordance with the standards set out in Paragraph 1 above. The classification and identification of these items are judgmental in nature and even comprehensive sampling and testing programs, implemented with the appropriate equipment by experienced personnel, may fail to locate some conditions. All investigations or assessments utilizing the standards of Paragraph 1 involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and all persons making use of such documents or records should be aware of, and accept, this risk. Some conditions are subject to changes over time and the parties making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or when the Client has special considerations or requirements, the Client must disclose them to WSP so that additional or special investigations may be undertaken, which would not otherwise be within the scope of investigations made by WSP or the purposes of the Report.

b. **Reliance on information**: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site investigation and field review and on the basis of information provided to WSP. WSP has relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, WSP cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in the report as a result of misstatements, omissions, misrepresentations or fraudulent acts of persons providing information.

c. **Additional Involvement by WSP**: To avoid misunderstandings, WSP should be retained to assist other professionals to explain relevant engineering findings and to review the geotechnical aspects of the plans, drawings and specifications of other professionals relative to the engineering issues pertaining to the geotechnical consulting services provided by WSP. To ensure compliance and consistency with the applicable building codes, legislation, regulations, guidelines and generally-accepted practices, WSP should also be retained to provide field review services during the performance of any related work. Where applicable, it is understood that such field review services must meet or exceed the minimum necessary requirements to ascertain that the work being carried out is in general conformity with the recommendations made by WSP. Any reduction from the level of services recommended by WSP will result in WSP providing qualified opinions regarding adequacy of the work.

6 ALTERNATE REPORT FORMAT

When WSP submits both electronic and hard copy versions of the Instruments of Professional Services, the Client agrees that only the signed and sealed hard copy versions shall be considered final and legally binding upon WSP. The hard copy versions submitted by WSP shall be the original documents for record and working purposes, and, in the event of a dispute or discrepancy, the hard copy versions shall govern over the electronic versions; furthermore, the Client agrees and waives all future right of dispute that the original hard copy signed and sealed versions of the Instruments of Professional Services maintained or retained, or both, by WSP shall be deemed to be the overall originals for the Project.

The Client agrees that the electronic file and hard copy versions of Instruments of Professional Services shall not, under any circumstances, no matter who owns or uses them, be altered by any party except WSP. The Client warrants that the Instruments of Professional Services will be used only and exactly as submitted by WSP.

The Client recognizes and agrees that WSP prepared and submitted electronic files using specific software or hardware systems, or both. WSP makes no representation about the compatibility of these files with the current or future software and hardware systems of the Client, the Approved Users or any other party. The Client further agrees that WSP is under no obligation, unless otherwise expressly specified, to provide the Client, the Approved Users and any other party, or any or all of them, with specific software and hardware systems that are compatible with any electronic submitted by WSP. The Client further agrees that should the Client, an Approved User or a third party require WSP to provide specific software or hardware systems, or both, compatible with the electronic files prepared and submitted by WSP, for any reason whatsoever included but not restricted to an order from a court, then the Client will pay WSP for all reasonable costs related to the provision of the specific software or hardware systems, or both. The Client further agrees to indemnify and hold harmless WSP, its officers, directors, employees, agents, representative or sub-consultant, or any or all of them, against any claim or any nature whatsoever brought against WSP, whether in contract or in tort, arising or related to the provision or use or any specific software or hardware provided by WSP.