Seismic Lifeline Study of Greater Vancouver Water District System and Assessment of Second Narrows Watermain Crossings

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Abstract: In 1993 a study of the Greater Vancouver Water District's (GVWD) water transmission system under various earthquake scenarios was completed (Kennedy/Jenks, 1993). The results of the screening level assessment indicate that failures of, and damage to, pipelines, pump stations, and reservoirs could occur, under moderate or higher levels of earthquake loading. Submarine and river crossings are vital components of the water supply system and are considered particularly vulnerable due to generally poor ground conditions and difficulty of submarine repairs. Based on the findings of the overview study, a program for detailed seismic assessment and upgrading was developed and is summarized herein.

A key element in the water supply system is the submarine crossing of the Burrard Inlet at Second Narrows by three large diameter pipelines. This paper also summarizes the geotechnical work carried out as part of the detailed assessment of these crossings. Key geotechnical issues include liquefaction, ground displacement, flow sliding, and buoyant uplift. Potential remedial measures to improve the reliability of these crossings are presented and operational considerations for the water distribution system are discussed.

Introduction

The Greater Vancouver Water District (GVWD) provides a safe and reliable wholesale supply of drinking water to 18 lower mainland municipalities, meeting the multiple needs of approximately 2 million residents. It is a key lifeline for the region and forms the backbone of the water supply system which brings potable water from the source to most residential, commercial and industrial taps in the member municipalities. Not only does the water supply meet drinking water and public health needs, it is essential for fire fighting, as well as ensuring a vibrant, expanding commercial, industrial and agricultural economic base for the region.

Historically, it was known that moderate earthquakes periodically occur in the coastal regions of southwestern British Columbia including the Lower Mainland. However, during the 1980's, studies indicated that the potential for a moderate or large earthquake in the region is much greater than previously anticipated. Rogers (1988) postulated that an earthquake along the Cascadia subduction zone would result in an earthquake of roughly M8 to M9. Paleoseismic evidence along the west coast of Vancouver Island, in the City of Richmond, and along the coast of Washington and Oregon suggest that such major seismic events have a recurrence interval of several hundred years. Although these events are predicated to occur 200 to 300 km from Greater Vancouver, the long duration and Peak Ground Acceleration of approximately 0.25 g would still result in substantial damage to the region's lifelines. These findings were accentuated in 1989 with the occurrence of the Loma Prieta earthquake in San Francisco. Even though seismic events are over relatively quickly, the loss of lifelines, in particular water supply, can cause major public health and safety problems, and disrupt normal activity in the region for weeks or months.

As a result of these developments throughout the 1980's and into the 1990's, the GVWD recognized the seismic risks associated with the water supply system in this region and the potential serious impacts on the residents, and commissioned a Lifeline Study of the regional water supply system. The purpose of the study was to assess the water system's vulnerability to seismic events and to develop recommendations for any necessary further steps such as remediation.

Water System History and Description

In 1889, a water main extending from an intake on the Capilano River on the North Shore of Burrard Inlet to the City of Vancouver went into operation. Built by the privately-owned Vancouver Waterworks Company, the 16 kilometre line represented the first step in what is now the regional water system. Prior to the construction of this line, all water in the city had come from wells or small lakes. Two years after completion, the water main was acquired by the city. It wasn't long after this that increasing demand identified the need for and highlighted the benefits of such projects.

In late 1892, New Westminster completed a pipeline from a small dam on Coquitlam Lake into its community. During this same time, the Seymour River was also tapped to supply drinking water, and by 1908 a new Seymour water system was constructed. The three sources to serve the region for the future were in place.

In 1924, the GVWD was created by provincial legislation at which time the population served was about 200,000 people. The GVWD was responsible for the water supply within Vancouver, South Vancouver and Point Grey (with these last two municipalities amalgamating with Vancouver in 1929). Nearly 50 years after its creation, in 1971, the GVWD became part of the Greater Vancouver Regional District. While still a separate legal entity, the District works as an integral part of the GVRD and is responsible for ensuring a safe and reliable supply of drinking water for the 18 municipalities it serves.

The water from mountain creeks and streams is collected in three protected watershed areas – Capilano, Seymour, and Coquitlam, located in the Coast Mountains immediately north of Greater Vancouver. There are dams on each river system to impound water and control water releases. Two of the main reservoirs, Capilano and Seymour, are supplemented by water released from smaller alpine lakes. From these storage lakes, water is distributed to 18 municipalities via an immense network comprised of six dams, 22 distribution system reservoirs, 15 pumping stations and over 500 kilometres of supply mains, shown in Figure 1. The average water consumption in 1998 was over 1,100 megalitres per day (ML/d), making the system one of the largest in Canada. The GVWD water treatment currently consists of primary disinfection with free chlorine at the three supply sources with a number of rechlorination stations throughout the region. Upgraded primary disinfection facilities at Coquitlam and Capilano will soon use ozone, and new Seymour facilities will use filtration followed by chlorination. Significant components of the system are operated and monitored from the Lake City Operations Centre where a modern Supervisory Control and Data Acquisition (SCADA) system monitors and stores data and controls the water flows and pressures.

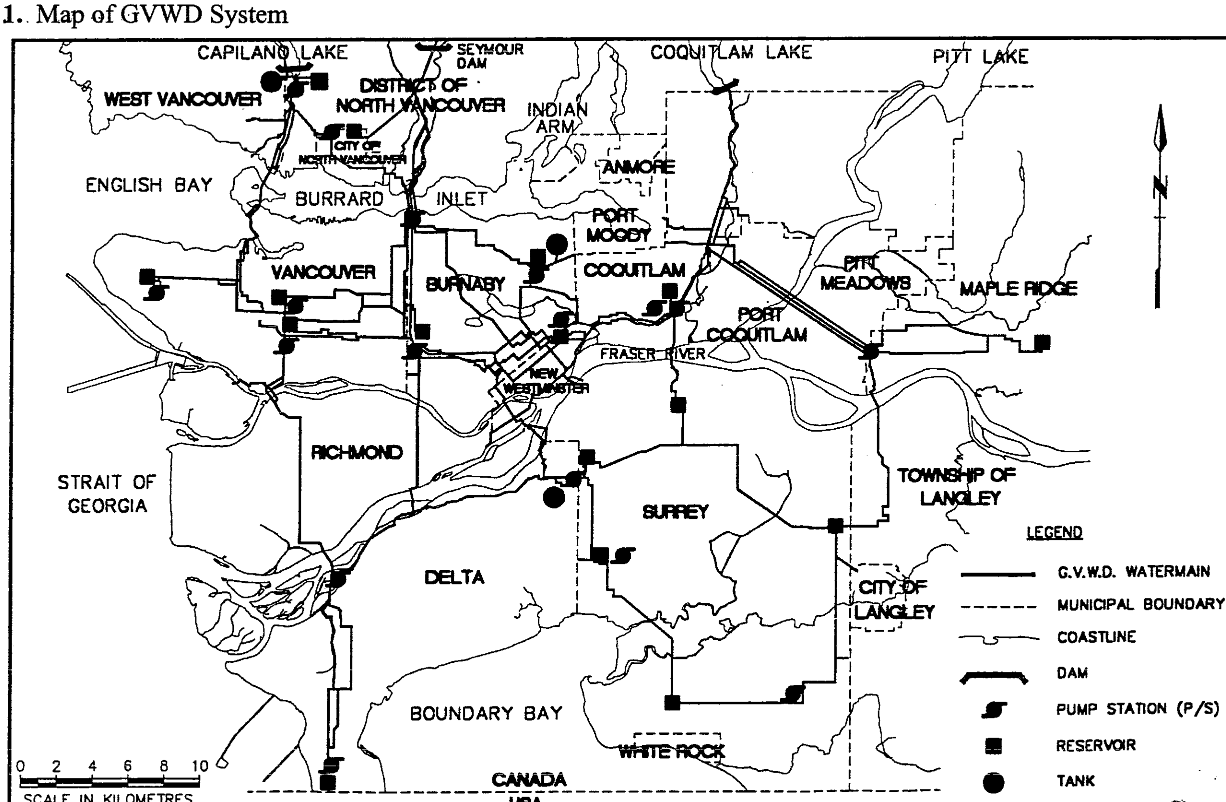
Overview of Lifeline Study

The lifeline study for the GVWD regional water system was commissioned and completed in 1993 by the team of consultants comprised of Kennedy/Jenks Consultants of Seattle and EQE Engineering and Design of San Francisco and the local firm Klohn-Crippen, with significant input by GVRD staff. The study included the following key steps:

- Development of proposed post earthquake performance objectives
- Verification and determination of seismic hazards in the service area
- Development of seismic hazard maps
- Assessment of seismic vulnerability of infrastructure
- Post earthquake system control and operation evaluation.

Each step is essential in the development of a lifeline study and ultimately in the preparation of a seismic impact mitigation plan.

Fig. 1. Map of GVWD System



Post earthquake system performance objectives

To develop performance objectives one must first understand how important each component of the system is to the long-term reliable delivery of potable water.

Water is stored in three major source reservoirs, Capilano, Seymour and Coquitlam. All 3 sources are required to meet high water demand in the summer months, however one of the sources can usually be taken out of service in the remainder of the year (October – April). With the exception of two out of fifteen pumping stations, water is moved through the system by gravity for much of the year outside the peak demand season. The transmission system consists of over 500 km of pipelines up to approximately 3 m in diameter. More than 90% of the mains consist of welded steel pipe while the remainder are comprised of other materials. The pipelines from the Capilano and Seymour sources cross Burrard Inlet either through tunnels or trenches at the bottom of the inlet. The mains from the Coquitlam source follow the Coquitlam River. Many of the mains in the system cross major rivers or water bodies such as the Fraser River and False Creek. Maintaining these crossings is essential to system operation and to providing water supply.

The system also includes 22 balancing reservoirs throughout the region which are essential for maintaining an adequate supply of water to member municipalities during peak hours. In addition, the system includes a number of primary and secondary disinfection facilities to ensure the water quality meets health guidelines and/or standards. Much of the system is now operated remotely from a central control room at Lake City, Burnaby, however manual operation is possible in the event of an emergency. It is clear that system operability is a function, not only of the performance of one component, but rather of all the components in a series (i.e. pump stations, pipelines, reservoirs).

Given an understanding of the system components and how they are inter-related, the primary functions of the water system were identified and prioritized as follows:

- Life Safety – prevention of death from infrastructure failure
- Fire Suppression – required immediately after a seismic event
- Drinking Water/Public Health – required within 3 days after an earthquake
- Other – i.e. Water for industrial use.

System performance was then put in the context of earthquake severity. In order to do this, two benchmark levels of earthquake were selected; Operating Basis Earthquake (OBE) and Design Basis Earthquake (DBE), where the OBE has a higher probability of occurrence than the DBE. Generic performance criteria were then set as follows:

OBE - No damage or minor damage, with continued normal system operation

DBE - Some damage and reduced operation of the system

Given the source and system redundancy and high reliance on gravity rather than pumping, it was proposed that a target basic level of service could be provided after a DBE. Subjective performance and policy objectives are detailed in Table 1.

At the time of the lifeline study a Maximum Credible Earthquake (MCE) was considered but not incorporated into the assessment; however, it must now meet MCE criteria for some critical infrastructure.

Service area seismic hazards

The performance objectives of the water system are tied to the level or intensity of the seismic event, OBE, DBE or MCE. These particular events are a function of location and both local and regional geology. A variety of combinations of magnitude, epicentral distance and peak ground acceleration can result in any one type of earthquake classification (i.e. DBE).

The Greater Vancouver area is situated over the active Cascadia subduction zone. Earthquake activity in this area is primarily related to the subduction of oceanic plates beneath the North American plate. Earthquakes that may present a hazard in this area are; deeper earthquakes within the subducting plate (intraplate earthquakes), shallow earthquakes in the continental plate above the subducting plate (crustal earthquakes), and thrust earthquakes on the subduction interface (interplate earthquakes).

Klohn-Crippen Consultants and B.C. Hydro, prior to the GVWD's lifeline study, undertook seismic hazard studies for the GVWD in the context of upgrading the system's two major dams, Cleveland and Seymour Falls. They used both deterministic and probabilistic methods to develop ground motions for OBE, DBE and MCE events consistent with the state-of-the-art in the early 1990's. This work was then reviewed and applied to the lifeline study, resulting in the selection of the following events for this study.

- The OBE has been selected as a "100-year" event; that is, a magnitude (M) 6.5 event with an epicenter about 70 km south of the Vancouver metropolitan area. This event results in peak firm ground accelerations of approximately 10 percent of gravity in the Vancouver area.
- The DBE has been selected as a "475-year" event; that is, a M7.0 event with an epicenter about 50 km south of the Vancouver metropolitan area. This event results in peak firm ground horizontal accelerations of approximately 20 percent gravity in the Vancouver area.

Table 1. Proposed post-earthquake system performance and policy objectives (1993)

Service Category/Priority	Operating Basis Earthquake	Design Basis Earthquake
Earthquake	40% chance of occurring in 50 years; 1/100 year return. PGA on rock 0.08 – 0.1g.	10% chance of occurring in 50 years, 1/475 year return. PGA on rock 0.18 – 0.23g.
Performance Objectives		
Pipelines	Limited to several failures	Minimize to several tens of failures.
Facilities	Continuous power service. Limit to minor, easily repairable damage to pump stations and reservoirs	Power restored in 72 hours. Control damage so only minor to 70% of pump stations and reservoirs, all remaining operable. Significant damage to remaining 30%, making them inoperable.
Policy Objectives		
1. Life Safety	Minimal life safety risk	Minimal life safety risk
2. Fire Suppression	Provide in all areas	Provide from 70% of reservoirs after valving off limited areas of damage.
3. Drinking Water/ Public Health	Provide continuous full service to all areas at winter demand rates. Maintain good water quality	Provide service to 70% of service area at 70% of winter flows; potable water made available at centralized locations, both within 72 hours. Boil-water order may be required.
4. Domestic, Commercial, Industrial Supply		
5. Property Damage		Provide full service to all but a few areas within 7 days at winter demand rates. Full service to all within 1 month at winter demand rates
6. Irrigation	Provide full service to all areas At summer demand rates within 7 days	Provide full service to all within 6 months at summer demand rates.

Seismic hazard maps of the lifeline area were then developed by overlaying the GVWD water system on a map of the regional surficial geology. To develop a liquefaction hazard map, each surficial soil unit in the region was classified in broad terms of liquefaction susceptibility (i.e. low, medium, high) based on the work of researchers Youd and Perkins. Variability of soil deposits and level of seismic event governs the extent to which liquefaction would occur in areas identified as susceptible. The magnitude of ground displacements were estimated using a semi-emperical model developed by specialists Bartlett and Youd.

In addition, the hazards of ground amplification and landslides, both related to earthquakes, were also evaluated. Ground amplification can occur when earthquake ground motions travel upward from a rock (firm ground) source through soil deposits, resulting in greater infrastructure damage. This analysis was undertaken using the early 1990's methods of Professor R.B. Seed. Areas in the region which are susceptable to seismically induced landslides were identified on the basis of topographic and existing geologic data.

Seismic vulnerability analysis

A screening level seismic vulnerability analysis was carried out for pipelines and other infrastructure, with the exception of the dams, which were considered high hazard and therefore addressed by a separate process.

For pipelines, two methods of analysis were undertaken: 1) an empirical system-wide estimate of damage for each level of earthquake and 2) an assessment of damage mechanisms for specific pipeline segments, which identifies those mains that are particularly vulnerable to liquefaction or flowslides and the type of failure (i.e. bending) associated with these segments. In order to do these analyses a detailed inventory of all pipelines, including key characteristics (i.e. joint types), and maintenance history, was compiled. For the system-wide analysis, this data was combined with the hazard map information into pipeline damage algorithms, to estimate the extent of damage for a broad range of damage mechanisms and pipeline materials. These algorithms had been developed by the consultant team on similar projects for other west coast cities and adjusted in accordance with the GVWD environment. The calculations provided estimates of pipe failures per unit length and total failures for the system under OBE and DBE scenarios. In making these determinations, it was estimated that for an OBE event, 5% of the ground susceptible to liquefaction will liquefy. For a DBE event, 35% was used. The results are summarized in Table 2.

Table 2 . Pipeline earthquake failure estimate

	Shaking			Liquefaction				Total
	Failure Rate (per 1,000')	Pipe Length (1,000')	Total Failures	Failure Rate (per 1,000')	Areal Distribution	Pipe Length (1,000')	Total Failures	Total Failures
Operating Basis Earthquake								
Steel Pipe (continuous)	0.0001	712	0.1	0.11	0.05	487	2.7	2.8
Concrete Pipe (segmented)	0.0004	140	0.06	0.39	0.05	81	1.6	1.6
Totals		852	0.1			568	4.3	4.4
Design Basis Earthquake								
Steel Pipe (continuous)	0.001	712	0.7	0.11	0.35	487	18.7	19.4
Concrete Pipe (segmented)	0.004	140	0.6	0.39	0.35	81	11.1	11.7
Totals		852	1.3			568	29.8	31.1

The second approach, pipeline segment analysis, considered liquefaction/lateral spreading, flow slides and landslides causing pipe barrel wrinkling, separation and possible future corrosion failure. The intent of this evaluation was to refine the damage assessment of pipeline segments taking into account structural characteristics and soil properties. The approach used a density function covering the range of strains over which wrinkling failure of pipe was likely to occur, and compares this information to estimates of net ground deformation and the length over which the ground deforms. For non-welded pipe an estimate of minimum joint separation was made. This was translated to minimum allowable ground displacement which was compared to estimated ground displacement. Vulnerability to flow slides and landslides was estimated by identifying mains which are located in liquefiable ground and on steep slopes (>8%) and mains through rugged terrain. Subjective vulnerability ratings of high or moderate were developed as follows:

Lateral Spread: High and moderate liquefaction associated failure probability

Flow Slides: Pipelines considered susceptible to flow slides have a high vulnerability

Landslides: Pipelines considered susceptible to landslides have a high vulnerability

The results of this analysis carried out for the key source watermains are shown in Table 3 with the highest (3) priority mains being most important to system operation.

In order to assess the vulnerability of the non-pipeline water system infrastructure, the system was modeled for four earthquake scenarios using the hydraulic network computer program "Waterworks". Based on the pipeline vulnerability assessment three scenarios each considered a single but different source out of service and the fourth scenario considered two sources out of service. Under each scenario different high vulnerability mains were considered to be out of service. The level of earthquake damage modeled was somewhere between an OBE and DBE event. A full DBE event was not modeled because the level of system damage was anticipated to be so severe that the model results would not be meaningful. It was also assumed that B.C. Hydro electrical power would be out of service and that even those pump stations with internal or backup power would be inoperable. A preliminary assessment was made of all balancing reservoirs/tanks and pump stations to determine their vulnerability to damage in both an OBE and DBE event. This data was incorporated into this model.

Table 3. Vulnerability summary (partial system only)

Pipeline	Priority (a)	Overall Vulnerability	Liquefaction			Land -slide
			Lateral Spread		Flow Slide	
			Wrinkling	Segmented		
Capilano Main No. 4	3	H	H	M		
Capilano Main No. 5	3	H	H		H	
Coquitlam Main No. 2	3	H	M			H
Coquitlam Main No. 3	3	H	H	H		H
First Narrows Crossing	3	H			H	
Second Narrows Crossing No. 1	3	H			H	
Second Narrows Crossing No. 2	3	H			H	
Second Narrows Crossing No. 3	3	H			H	
Seymour Main No. 2	3	H	H			
Capilano Main No. 7	3	M	M			
Seymour Main No. 3	3	M		M		
Annacis Main No. 2	2	H	M		H	
Annacis Main No. 3	2	H	M		H	
Angus Drive Main (Marpole Crossover)	2	H			H	

NOTES: (a) High Priority = 3; Low Priority = 1 H = High Vulnerability; M = Moderate Vulnerability

Anticipated seismic performance

The results of this system-wide study suggested the following conclusions:

OBE Event:

The GVWD system would be expected to generally remain operable in the OBE event. The estimated number of water main failures that could be expected to occur was less than 10. These water main failures should not seriously impact overall system performance, although some areas may be more adversely affected than others, however, further and more detailed studies would be required to confirm this. Some minor repairable damage would be expected to reservoirs and pump stations but all pump stations should remain operable.

DBE Event:

The GVWD system was predicted to be severely impacted by a DBE event. An estimated 30 water main failures would occur, making much of the system inoperable. There was considered to be a high probability that the transmission system delivering water south of Burrard Inlet would be inoperable as a result of failure of the Burrard Inlet crossings and failure of the land based mains from Coquitlam Lake along the Coquitlam River valley. There was also considered to be a high probability of failure at various submerged river crossings because of anticipated large ground movement.

It was concluded that all pump stations would likely be inoperable due to non-structural and, in some cases, structural damage or due to commercial power outage. All reservoir roofs/column supports were found to be vulnerable and some may collapse, although in most cases this should not impact the hydraulic integrity of the reservoir.

These conclusions imply that the post earthquake system performance and policy objectives identified in Table 1 would not be met. In order to meet the proposed performance objectives, several recommendations were made:

OBE Event:

- Anchor equipment and support pipe in pump stations and treatment facilities where needed.
- Develop and employ earthquake design standards for all new facilities which are not governed by the National Building Code of Canada (NBCC).
- Incorporate earthquake considerations in the capital improvement planning process.
- Develop, implement and practice an emergency operating plan that includes earthquake disasters.
- Conduct system hydraulic analysis for post earthquake system operation.

DBE Event:

- Improve Source Reliability – connections to all 3 sources are vulnerable. Therefore more detailed study, including the following, was recommended.
  - Refine the liquefaction/flow slide susceptibility of the Capilano River delta, the Seymour River delta and the Coquitlam River valley.
  - Refine the understanding of landslide potential along the Coquitlam River Valley water main alignment.
  - Establish site-specific seismic vulnerability based on the geotechnical information and develop a prioritized mitigation program for source reliability improvement.
- Improve Transmission System Reliability – the transmission system serving the east, central and western delta areas south of the North Arm of the Fraser and Pitt rivers was considered to be vulnerable to failure because of ground movement primarily due to liquefaction-induced lateral spread/flow slide. More detailed information and investigations would be required.
- Upgrade Reservoir Roofs and Support Structures.

GVWD Seismic Assessment and Upgrading Program

Prior to completion of the lifeline study, the GVWD had already begun a program of investigating the seismic resistance of its two major dams, Cleveland on the Capilano River and Seymour Falls on the Seymour River, and the system's largest and oldest balancing reservoir, Little Mountain, in Queen Elizabeth Park and undertaking seismic upgrades. In 1994, subsequent to the completion of the lifeline study, the District began to develop a program to address the major issues and recommendations identified in the report. The program includes the five major initiatives which are described below. Given the extent of investigations, design and upgrade anticipated, the program has been developed to be completed over approximately 20 years. Highest priority is being given to projects which address life safety, public health and fire protection.

Seismic Design Standards

Standards were developed to address the performance objectives of all major GVWD infrastructure during a DBE (1 in 475 year event) and an MCE (1 in 10,000 year event). All major new infrastructure is expected to survive an OBE (1 in 100 year event) without notable damage or interruption to service. The standards are now under revision to meet the current state of the art and to make them more comprehensive.

Improve Source Reliability

The first step in this initiative was to undertake a detailed Natural Hazards Assessment for all three source valleys, Capilano, Seymour and Coquitlam. Through a compilation of existing data, field verification and selective drilling, natural hazards which could impact GVWD infrastructure, were identified. Decision criteria comprised of definition of hazard type and magnitude, hazard probability rating, impact type and intensity were then used to classify and prioritize hazard areas. While the study indicated that most GVWD infrastructure has a low natural hazard risk, there are several areas within the source valleys that require further assessment.

Upgrade Pump Stations

A number of issues, primarily structural and mechanical anchorage weaknesses were noted during the lifeline facility assessment at most pump stations. Although most of the pump stations are not as high a priority for seismic retrofit as other infrastructure, a program is underway to address these concerns and the work has been completed at several locations.

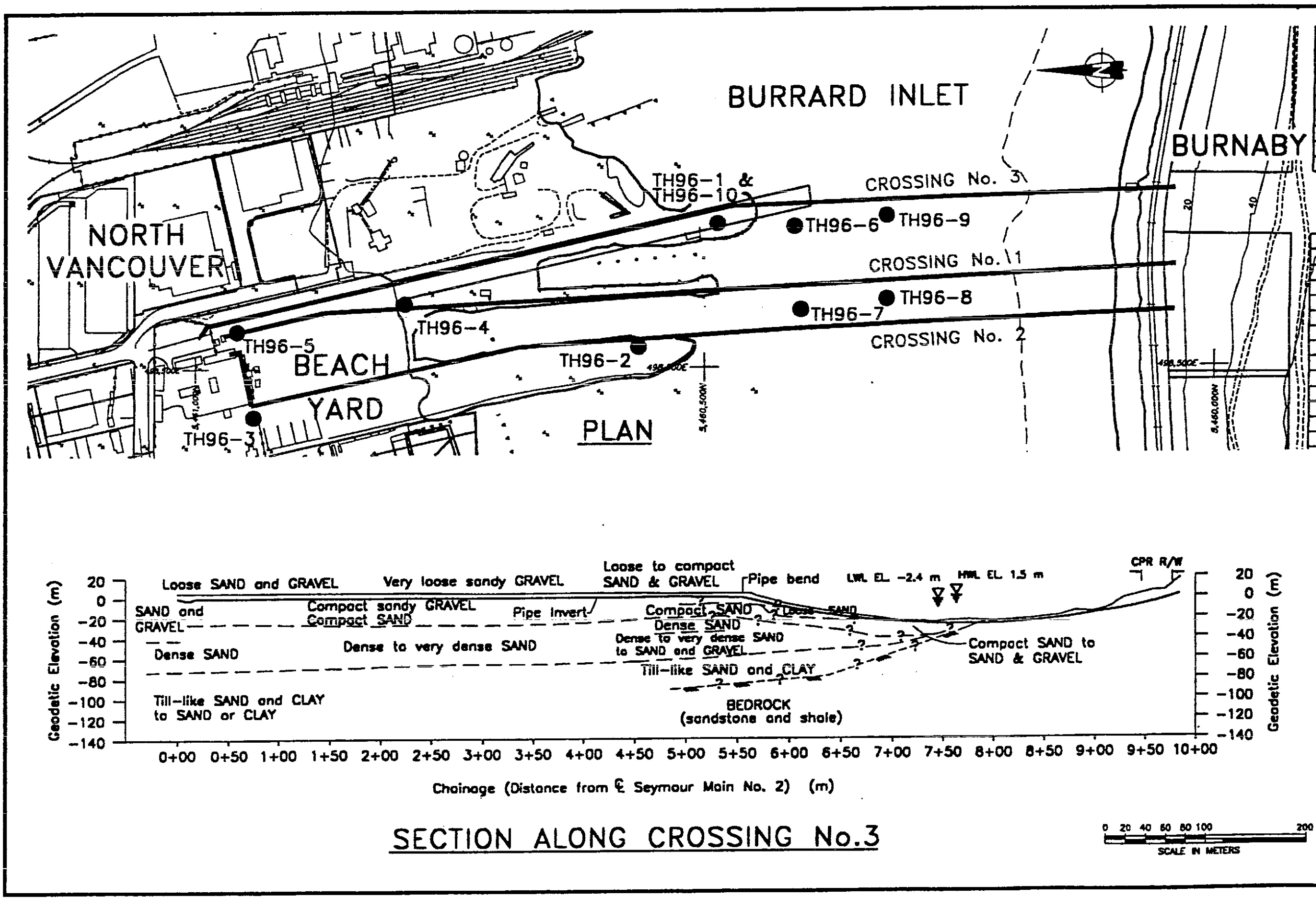
Upgrade Reservoirs

The lifeline study indicated that many of the reservoirs, most of which were built between 1910 and 1980, likely have insufficient structural capacity in the roof slab and supports to withstand a moderate earthquake. The reservoirs are typically large in-ground concrete lined basins with reinforced concrete roofs supported from beneath by beams and columns. These reservoirs range in storage volume from 5,000 m^3 to 137,000 m^3 and provide vital storage of water to help meet peak daily demands during the high summer draw periods. They also serve as local reserve of water in the event of an interruption in supply from the feeder mains.

A structural evaluation program has progressed with a preliminary assessment of the oldest reservoirs in the system. These were given the highest priority because they were considered the most vulnerable to damage during an earthquake, and because of their importance in terms of the Region's post-disaster water supply plan.

Several reservoirs, including Vancouver Heights and Central Park in Burnaby and Kersland in Vancouver, have now been upgraded and are capable of resisting even a major earthquake with minimal damage, allowing the facility to continue to operate at near full capacity. It is anticipated that seismic deficiences of all reservoirs will have been addressed within the next decade. Little Mountain reservoir, in Vancouver's Queen Elizabeth Park, the oldest, largest and one of the most important in the system, has been determined to likely require nearly a full replacement. The detailed design phase of this work is expected to begin later this year.

Fig. 2 Section Along Crossing No. 3.



Improve Transmission Reliability

The lifeline study indicated that many mains are located in liquefiable ground, with river crossings and adjacent segments being most vulnerable to significant damage even in a moderate earthquake. Given this finding, the primary focus of this work has been to investigate submerged crossings for seismic vulnerability as they would be difficult and likely take months to repair in the event of failure.

The 13 major submerged crossings have been prioritized; those closest to the source and those likely to have the largest impact on water supply capability in the event of a failure have been given highest priority. The work is being undertaken in stages commencing with a seismic vulnerability assessment, and if necessary, followed by preliminary and detailed design of mitigation measures, and subsequently by construction. To date several crossings have been, or are being, assessed.

In late 1997 a comprehensive assessment of the three submerged crossings at the Second Narrows of Burrard Inlet was completed. The complexity and value of this work is outlined in the following section.

Second Narrows Crossings Seismic Vulnerability Assessment

A key element in GVWD's water supply system is the submarine crossing of the 400 m wide Second Narrows of Burrard Inlet by three large diameter pipelines (Fig. 2). The lifeline study (Kennedy/Jenks, 1993) indicated that the three pipelines could be damaged by a significant earthquake, and it was recommended that the vulnerability of these pipelines be assessed by means of further detailed analyses. This study was undertaken by Thurber Engineering Ltd. and Sandwell Inc., with specialist input from the Centre for Engineering Research Inc. and from Dr. P. Byrne & Dr. D.L. Anderson from the University of British Columbia. (Thurber Engineering, 1997)

Table 4: Second Narrows Crossing Pipe Details

Crossing	Year Constructed	Diameter (OD), m	Wall Thick., mm	Type	Min. Yield Stress, Mpa	Ult. Yield Stress, Mpa
No.1	1948	1.30	32	Wrought iron pipe (ASTM A42-39) with mechanical couplings	152	303
No.2	1954	1.30	32	Wrought iron pipe (ASTM A42-39) with mechanical couplings	152	303
No.3	1977	2.01	13	Welded steel pipe (ASTM285 Class C), concrete-encased	207	379

Description of Pipelines

The watermain crossings commence at the GVWD's Beach Yard compound on the North Shore of Burrard Inlet in North Vancouver and pass beneath Burrard Inlet before coming up a steep bedrock slope on the south bank of the inlet (Fig. 2). The pipeline corridor is approximately 130 m in width, and is located about 500 m east of the Iron Workers Second Narrows Bridge.

Fig. 3 Crossing No. 2 Construction 1953 – View to North

The Beach yard contains numerous valves and piping connections which enable water from three Seymour supply mains and one Capilano supply main to be routed to the three Second Narrows submarine crossings. In addition, there is a control building which houses instrumentation that permits remote monitoring and regulation of flows and pressures from the Lake City operations centre.

Crossings Nos. 1 and 2 were built of wrought iron and laid one pipe section at a time in an excavated trench. Underwater sections were joined with mechanical couplings. Figure 3 shows a view of the harbour crossing during construction of Crossing No. 2 in 1953.

For Crossing No. 3, a trench was excavated on the bottom of the inlet in the north half leading into a tunnel excavated in sedimentary bedrock on the south half of the inlet. The pipeline was welded (double welded bell & spigot joints) and concrete encased on the south side of the inlet, and then launched by gravity from the south side combined with winching from the north side of the inlet.

On the north shore of the inlet, each of the crossings extends south from the Beach Yard towards the inlet along artificial sand spits formed during construction of the mains.

Subsurface Conditions & Geotechnical Investigations

Several geotechnical investigations had been carried out in the area between 1925 to 1992 for bridge and watermain crossings of the inlet, providing an indication of the local geology in the area. The general geology is simplified as follows:

Sandstone/siltstone bedrock is the predominant unit outcropping on the south side of the inlet. From the centre of Burrard Inlet, the bedrock dips down to the north and is more than 100 m below ground surface beneath the Beach Yard on the north shore of the inlet.

On the north side of the inlet and on the north shore, the overburden consists of loose to compact clean sand and gravel to a depth of up to about 30 m, underlain by dense sand or sand and gravel containing a trace to some silt, which in turn is underlain by dense till-like sand and clay over bedrock.

A geologic profile across the inlet is shown on Fig. 2. In order to supplement the geotechnical information for detailed assessment of the three watermain crossings, additional drilling was carried out in 1996.

These included five Becker Hammer testholes (168 mm OD) drilled onshore (TH96-1 to TH96-5) and four drilled offshore from a barge (TH96-6 to TH96-9). Both open and closed ended Becker drilling was carried out within 2 to 3 m of one another at each location, in order to provide soil samples for laboratory classification tests plus soil penetration resistance. The bounce chamber pressures of the closed ended Becker tests (i.e. Becker Penetration Tests or BPT) were recorded during penetration to allow blow counts to be corrected to equivalent N_{60} values in accordance with procedures recommended by Harder and Seed (1986).

In addition, a mud rotary drillhole (TH96-10) was drilled near the inlet shoreline adjacent to Becker drillhole TH96-1. Standard Penetration Testing (SPT's) was carried out at 1.5 m depth intervals, with energy measurement made during the SPT's to determine the hammer efficiency and enable energy correction of the blowcounts to the standard 60% transferred energy (i.e. N_{60}). PVC casing was installed in the hole and the annulus grouted to provide for downhole shear wave velocity measurements.

Near the shoreline (i.e. southern end of sand spits), the sands and gravels below the pipelines were found to be primarily compact with $(N_1)_{60}$ values typically in the range of 9 and 15 to depths of at least 15 m (see TH96-1 on Fig 4) The BPT and SPT values shown on Fig. 4 (TH96-1 and TH96-10) are in reasonable agreement, confirming the suitability of the Harder-Seed correlation. Profiles of $(N_1)_{60}$ are similarly shown for TH96-4, located about 300m to the north of the submerged pipelines, and TH96-9 & TH96-6 located within the inlet. These results indicate that the sand and gravel deposits are loosest along the submerged north inlet slope with $(N_1)_{60}$ values ranging from 5 to 15 to a depth of about 15 m. North of the sand spits and also near the centre of the inlet, somewhat denser sands and gravels are present, with $(N_1)_{60}$ values in these areas typically ranging from 15 to 25.

Results of Vulnerability Assessment

Liquefaction Analyses

Resulting from a review of area seismicity and parameters used on other local projects, the following earthquake shaking parameters were used in the assessment:

Table 5 Earthquake Parameters

Earthquake	PGA	Magnitude	Epicentral Distance (km)
1:100 year	0.1 g	6.5	50
1:475 year	0.2 g	7.0	35
MCE	0.5 g	7.5	15

Several attenuation relationships were evaluated and recommended acceleration spectra for the different return periods were established based on a review of state of the art and local practice. Five earthquake time history records from two earthquakes (1971 San Fernando & 1979 Yugoslavia) were selected as appropriate for the Vancouver area, considering duration and acceleration/velocity ratios, and were modified so that the resulting spectra closely matched the recommended spectra.

The performance of the soil strata during the three earthquake events were assessed using the computer programs SHAKE and FLAC

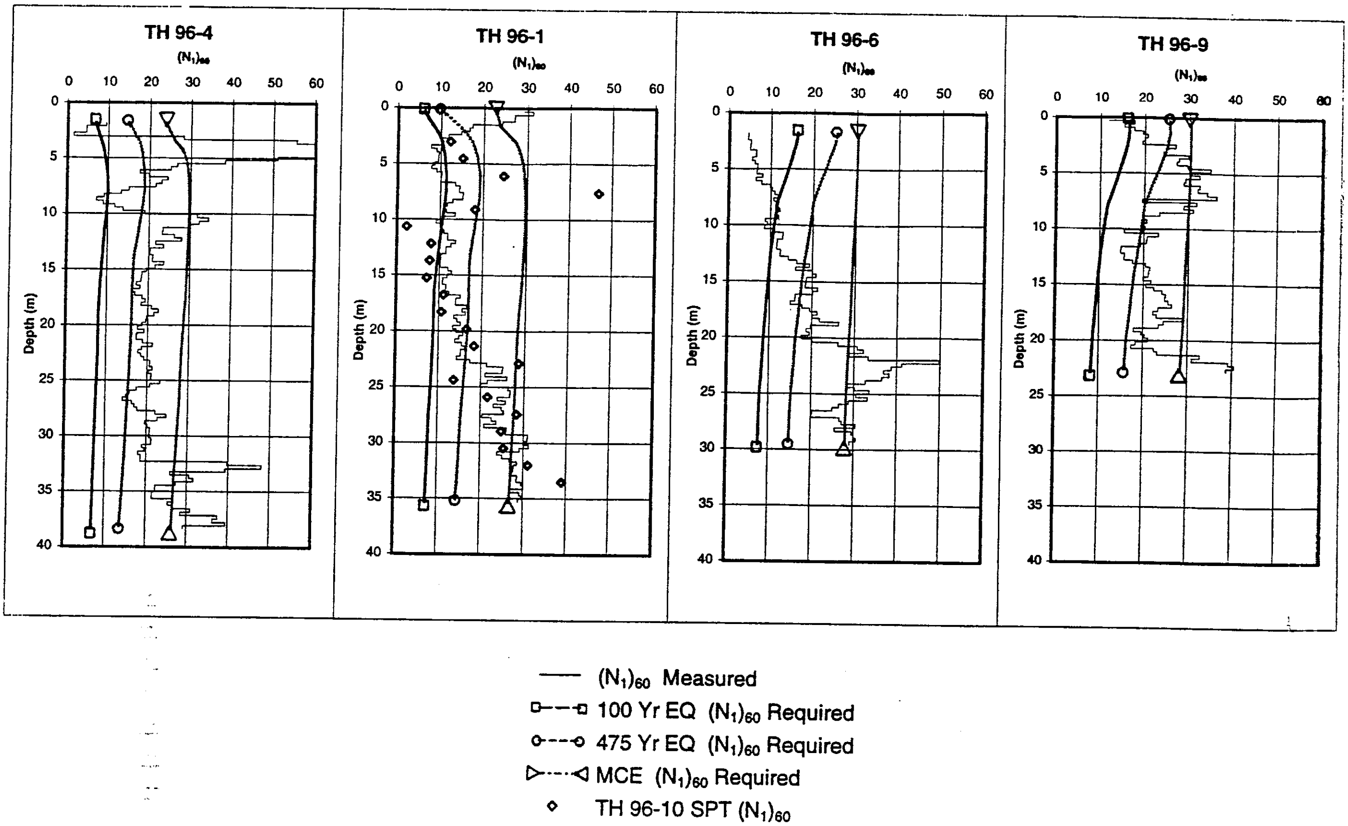
SHAKE models the upwards propagation of horizontal shear waves through a vertical soil column and is routinely used to assess the amplification of ground motions and induced cyclic stress ratio (CSR) in the soil for liquefaction assessment. Three onshore soil columns and one offshore soil column were evaluated for the three levels of earthquake shaking. Liquefaction susceptibility at each BPT test location was evaluated in accordance with Seed et al (1985) procedure which involves comparing the $(N_1)_{60}$ required to prevent liquefaction for a given CSR with the actual $(N_1)_{60}$ profile at the location.

The required $(N_1)_{60}$ values to resist liquefaction under three levels of earthquake shaking profiles are shown on Figure 4. The required $(N_1)_{60}$ values were computed from the average CSR of the five earthquake records. The plots show that for the 1:100 year event, liquefaction would be limited to less than 4 m thickness at the onshore areas and in the middle of the inlet, but could be up to 11 m thickness along the submerged slope of the channel.

The analysis for the 1:475 year event indicates that the thickness of liquefiable layer increases significantly to between 11 and 21 m, except at TH's 96-4 & 96-9 where the thickness of liquefiable layer is predicted to be 4 and 5 m respectively.

For the MCE event, the thickness of liquefiable layer is predicted to be very large (19 to 35 m) for all locations drilled.

Fig 4. Liquefaction Assessment Curves



Ground Movements

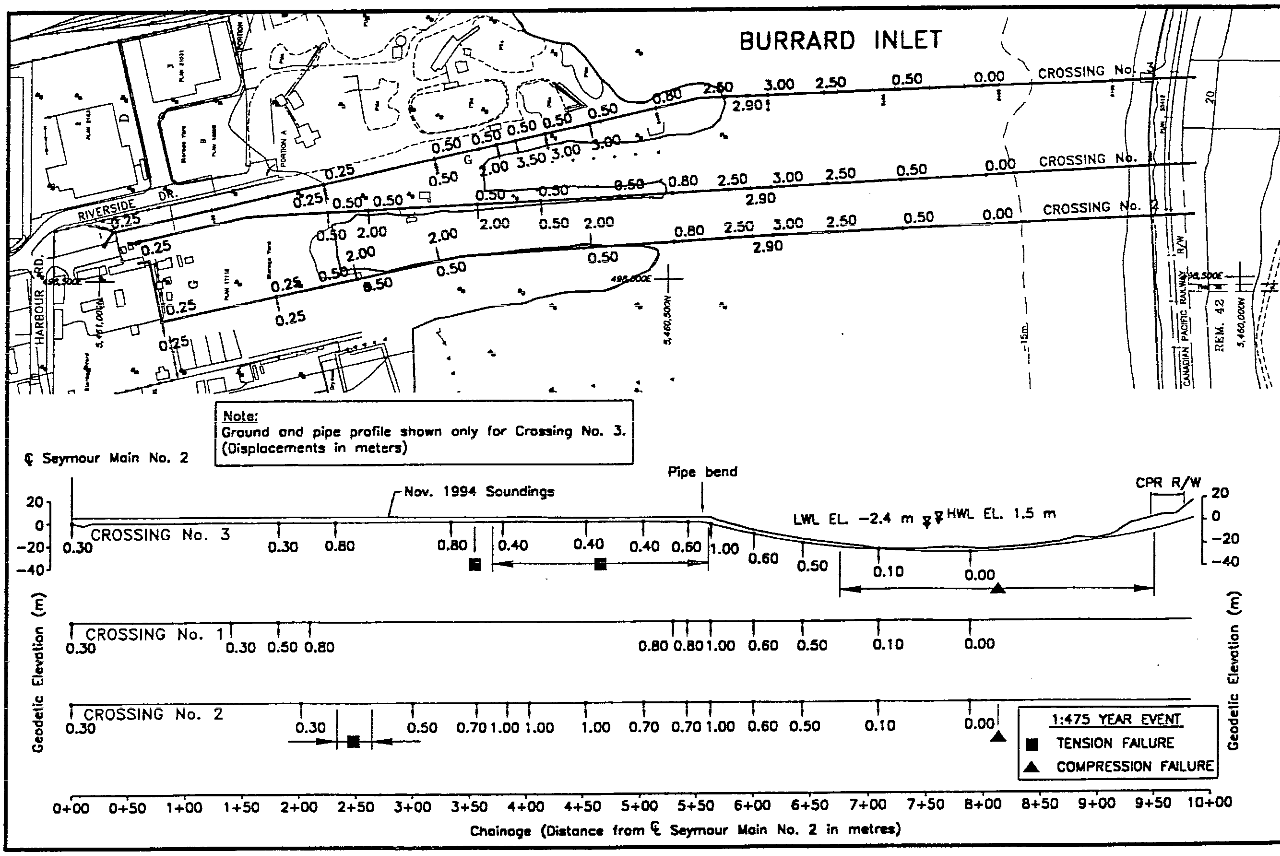
Displacement analyses were conducted using FLAC finite element analyses to give horizontal and vertical displacements with depth along the geologic profile, using inferred post liquefaction input parameters. Results were plotted in terms of the completed vertical and horizontal displacements for each pipeline for all three levels of earthquake shaking, and are summarized below.

Table 6 Soil Displacements Summary

Mode	Computed Soil Displacements along Pipelines (m)		
	1:100 year	1:475 year	MCE
Longitudinal	0.5	3	12
Lateral	0.5	3	4
Vertical	0.5	1	2

A plan and profile of displacements under the 1:475 year scenario are plotted on Fig. 5 to demonstrate the pattern of displacements. As expected, computed horizontal and vertical displacements are greatest near sloping ground and in the direction of the slope. Large differential horizontal movements occur in some areas as a result of abrupt changes in ground surface, such as in the vicinity of the spits.

Fig 5. Plan and Profile of Displacements – 1:475 Year Scenario



The Bartlett and Youd empirical method was used to compute the ground surface horizontal displacements and was used to calibrate the FLAC model, by adjusting the input parameters within a range of expected values.

Although ground surface displacements of up to 13 m were obtained in the analyses, the potential for a flow slide was not explicitly considered, and is difficult to evaluate due to the wide range in potential residual strengths (i.e. post liquefaction) based on penetration test data and field experience. However, when residual strengths somewhat lower than those selected for the FLAC model were used, considerably larger displacements resulted, even for the 1:100 year event. Because of the steep underwater slopes and in the absence of sophisticated laboratory testing to possibly clarify the prediction, it was concluded that the possibility of a flow slide should be considered even for the 1:100 year event. However, for the seismic vulnerability assessment it was decided to compare the response of the pipelines to the computed FLAC displacements for the three earthquake levels to assess their likely performance, and to acknowledge that flow sliding was a possibility, although uncertain.

Pipelines Performance

During a ground movement event, loads are induced in a pipeline by the relative motion between the pipe and the surrounding soil. Using the finite element program ABAQUS, each pipeline was modeled with soil-spring models to characterize this soil pipe interaction, using the FLAC computed ground displacement pattern as input to the model. Separate axial, transverse and vertical bi-linear compliance springs (i.e. elastic, perfectly plastic P-y curves) were specified for liquefied and non-liquefied soil mass.

The axial and compressive strains for the pipelines were computed and compared against the established pipe limits. The peak values of the axial strain in each of the Second Narrows Crossings for each movement scenario are summarized in Table 7.

Table 7 Peak axial tensile and compressive strains

Crossing	Strain Limits: +ve Tensile –ve Compressive	1:100 year	1:475 year	MCE
Crossing No. 1	+0.5 % -1.1%	+0.12 % -0.14 %	+0.39 % -0.61 %	**+0.92 %** **-4.00 %**
Crossing No. 2	+0.5% -1.1 %	+.28 % -0.21 %	**+0.64 %** **-1.52 %**	**+1.40 %** **-11.2 %**
Crossing No. 3	+0.5 to 2.0% -0.35 %	+0.40 % -0.29 %	**+0.88 %** **-1.38 %**	**+1.98 %** **-6.9 %**

Numbers in bold indicate strain limit exceeded

It should be noted that while a tensile limit of 0.5% has historically been used for pipelines to protect against fracture initiation at circumferential weld flaws, a higher tensile strain limit of up to 2% is considered feasible for continuously welded modern steel pipe such as for the Crossing No. 3 pipeline. However, since the performance of double lap-welded joints is uncertain based on past experience, a range of tensile limits was used for Crossing No. 3. From the results given in Table 7, the assessment of pipeline vulnerability, excluding the effect of an earthquake induced flow slide on the underwater slopes, can be summarized as follows:

- For the 1:100 year earthquake, the three crossings are predicted to maintain their integrity and remain functional.
- For the 1:475 year earthquake, Crossing No. 1 is predicted to remain functional, Crossing No. 2 is expected to fail at the vertical bend at the north shoreline, and Crossing No. 3 is expected to buckle in the underwater section north of the tunnel and to undergo tensile failure on the north shore. The locations of the overstressed sections of pipe are shown on Fig. 4.
- For the MCE event, loss of pressure integrity is predicted for all three pipelines.

  In addition to the above, it was determined that buoyant rise of the underground chambers and settlement of the thrust blocks in the Beach Yard could cause damage or pipe failure for any of the earthquake levels examined.

Remedial options & operational considerations

Based on the results of the above analyses, it was determined prudent to consider installation of three new valve chambers in the Beach Yard to improve redundancy in enabling water from any of the four source pipes to be routed to any of the three submarine crossings, should one or two of the submarine pipelines fail.

Long-term options to improve the post earthquake reliability of the Second Narrows Crossings include:

Ground improvement around the valve chambers and along the watermains or along the foreshore to prevent soil liquefaction. To prevent the possibility of underwater flow sliding, considerably more densification would be required on the underwater slope. Vibro-replacement is considered to be the most feasible option of ground improvement in these soil conditions. Additional soil densification and/or infilling the basin between the sand spits would be required to minimize potentially large transverse pipe movements.

- A new deep crossing consisting of a vertical shaft in the Beach Yard, with horizontal tunnel in till and bedrock. Ground densification would be required to prevent liquefaction around the shaft.
- Consideration of other options such as rerouting the crossing pipeline(s) onto future remediated bridge crossings in the area
- Improving the ability to route water to an existing alternate water supply crossing located at the First Narrows of Burrard Inlet, provided that the seismic performance could be confirmed to be superior to that of the Second Narrows Crossings. Seismic vulnerability assessment of the First Narrows Crossing is currently underway to confirm the performance of this key crossing.

Summary

The potential seismic risk to GVWD's water supply system has been evaluated by means of a Lifeline Study completed in 1993, which assessed the system's vulnerability to seismic events. The study involved an assessment of seismic hazards, a screening level seismic vulnerability analysis, and identification of performance objectives for various system components. Results of the study indicated that the GVWD system is expected to generally remain operable after a 1:100 year earthquake event, but would be severely impacted by 1:475 year, or greater, events. An upgrading program was initiated in 1994 to be completed over a period of about 20 years.

As part of the seismic assessment and improvement program, a seismic vulnerability assessment of the GVWD's three Second Narrows submarine watermain crossings was undertaken in 1997. These crossings are key elements in GVWD's water supply system. The seismic vulnerability assessment indicates that two of the three submarine pipelines are expected to be damaged by ground movements under a 1:475 year earthquake, and all three are considered likely to be seriously damaged under the MCE scenario. Although unlikely, flow sliding could result in loss of all three pipelines even under a 1:100 year event. Upgrading of the onshore facilities is planned to be completed over the next several years, although due to the very large expense involved, offshore improvements will depend on the detailed seismic assessment of an alternate watermain crossing at First Narrows which is currently underway.

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