



**LITTLE MOUNTAIN RESERVOIR RECONSTRUCTION TO MEET
MAXIMUM CREDIBLE EARTHQUAKE PERFORMANCE CRITERIA**

A. P. Sukumar¹ John Sherstobitoff² Frank Huber³

SUMMARY

The Greater Vancouver Water District (GVWD) provides a safe and reliable wholesale supply of drinking water to approximately 2 million residents in the Greater Vancouver area, located in British Columbia, Canada. It owns and manages the water transmission system that is a key lifeline for the region that brings potable water to most taps in the area and is essential for fire fighting. Little Mountain Reservoir is located at the highest geographic point in Vancouver, and is the largest and most important distribution reservoir in the GVWD system. It was originally constructed as an earthen berm contained open basin in 1910 and a pre-cast concrete roof structure, subsequently used for parking and recreational purposes, was added in the mid 1960s. Investigations determined that the concrete roof structure and the surrounding earthen containment berm did not meet modern seismic criteria and even a moderate earthquake would likely have caused collapse of the roof and breaching of the berm resulting a sudden release of stored water. It was decided to reconstruct the reservoir on the same site as a monolithic concrete structure independent of the containment berm, to meet GVWD's seismic performance criteria.

The paper outlines details of GVWD's seismic performance criteria and site specific response spectra, and the design of the 20,000 square meter footprint monolithic structure with no expansion joints that met the requirements of the 0.5g Maximum Credible Earthquake criteria.

The preliminary design phase evaluated several preferred options of rebuilding the existing seismically deficient reservoir and finally a radical, but simple concept was developed to meet the structural and seismic objectives. The structure is a monolithic six-sided concrete "box" 180m x 120m x 10m in size with no expansion joints, incorporating a central dividing wall to create 2 reservoir cells. Seismically, this provides an extremely robust structure. The roof is an economical design as its primary function is to support the rooftop usage loads, rather than lateral seismic loading. The 'box' was designed to fit completely within the confines of the existing reservoir perimeter wall to avoid disturbance to the adjacent vegetation and minimize change to the current park setting. Excavating vertically into the embankment for the new vertical side walls created increased storage volume. Furthermore, by raising the roof level 600mm, a total volume increase of 25% was achieved. The 'box' no longer relies on the embankment for

¹ Senior Engineer, Structural Group, GVRD, Vancouver, Canada. a.sukumar@gvrd.bc.ca

² Manager of Buildings and Infrastructure, Sandwell Engineering Inc., Vancouver, Canada.

³ Division Manager, Engineering Support and Technical Services, GVRD, Vancouver, Canada.

any support to contain water during an earthquake and any potential for catastrophic release of water is eliminated.

Demolition of the existing reservoir began in September 2002 and construction of the new facility was completed by December 2003 on schedule and within budget. The project was designed to commission one-half of the reservoir before the high-demand summer period by completing a majority of construction in a compressed 8½ month schedule. Sustainability principles were integral throughout the design, demolition and reconstruction of the Little Mountain Reservoir,

INTRODUCTION

The Greater Vancouver Water District (GVWD) owns and operates the water supply, transmission and treatment system that provides a safe and reliable supply of drinking water to approximately 2 million residents in the Greater Vancouver area, located in British Columbia, Canada. Water is collected from three protected watershed areas and distributed to various municipalities in the region through a network comprised of six dams, 22 balancing reservoirs, 15 pumping stations and over 500 kilometers of supply mains. The average water consumption in 2003 was approximately 1.2 million cu.m per day, making the system one of the largest in Canada.

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 [1] 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 predicted to occur 200 to 300 km from Greater Vancouver, the long duration at even a relatively low acceleration would still result in substantial damage to the region's lifelines. Furthermore, Greater Vancouver is prone to shallow crustal earthquakes of magnitude M6 to M7.5 with shorter duration but potentially much larger firm ground accelerations than the subduction event. These findings were accentuated in 1989 with the occurrence of the Loma Prieta earthquake in San Francisco. Even though most seismic events are of relatively shorter durations, 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 began to develop a seismic evaluation and upgrade program for the system.

GVRD'S RESERVOIR SEISMIC UPGRADE PROGRAM

An important component of the GVWD system is the network of balancing reservoirs, which 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 m³ to 175,000 m³ and provide vital storage of water to help meet peak daily demands during the high summer draw periods. They also serve as a local reserve of fire fighting and drinking water in the event of interruption in supply from the feeder mains. At the time of construction of many of the GVWD's reservoirs, there was minimal awareness of the seismicity of the region and subsequently there was little or no seismic consideration included in the design of these reservoir structures. Since that time, the potential for a large earthquake to occur within the region has become widely accepted and seismic design criteria for structures have become much more stringent.

In the late 1980's, a structural evaluation program focusing on the assessment of potential seismic deficiencies of each of the GVWD distribution system reservoirs commenced with the preliminary assessment of the oldest and biggest 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. To date GVWD has undertaken seismic upgrading of 6 major balancing reservoirs using a variety of approaches including interior shear walls, seismic dampers, exterior buttresses, wall thickening, and complete demolition and reconstruction, in the case of Little Mountain Reservoir. Examples of some of these methods are shown in Figures 1 & 2.

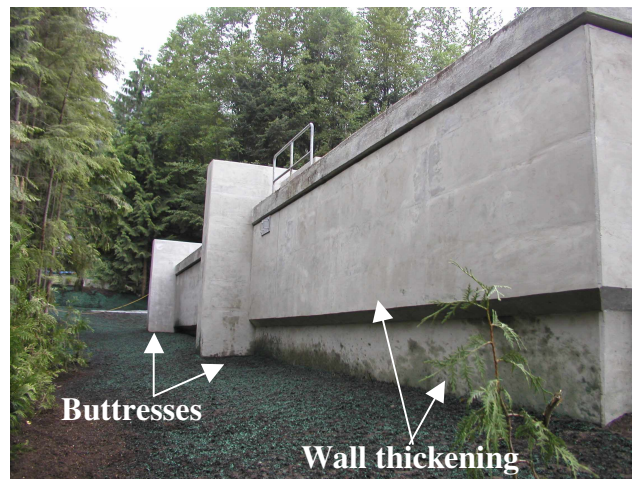


Fig. 1 Prospect Reservoir Seismic Upgrade

Seismic upgrade work includes thickening exterior of the reservoir walls and constructing new buttresses (2 for each side) to transfer seismic forces to the foundation



Fig. 2 Cape Horn Reservoir Seismic Upgrade

Seismic upgrade work includes thickening the interior of reservoir walls and using viscous dampers to dampen the seismic roof forces before transferring to the walls.

SEISMIC ASSESSMENT OF THE LITTLE MOUNTAIN RESERVOIR

Little Mountain Reservoir, located at the topographic high in the centre of Queen Elizabeth Park, in the City of Vancouver, is the oldest and largest distribution reservoir in the GVWD system. It is also the largest local source for emergency post-earthquake drinking water and fire fighting, located at an elevation

to enable gravity feed to the system. Originally constructed as an open basin in 1910, with a perimeter earthen embankment, a roof structure was added in 1967 to protect water quality. A large portion of the roof structure has been used as a public parking lot for park users, and the remaining reservoir surface was covered with decorative features and walkways, which were constructed by the Vancouver Board of Parks and Recreation a number of years ago. Figure 3 shows the Little Mountain Reservoir site prior to the reconstruction including the newly installed water mains and GVWD's Kersland reservoir, which was seismically upgraded in 1997.



Fig. 3. Little Mountain Reservoir – Site Plan

A series of consultant's reports compiled between 1990 and 1993 have indicated that the reservoir structure and the perimeter embankment do not meet modern seismic criteria and even a moderate earthquake would likely cause a sudden release of water. It was also determined at the time that the reservoir roof structure was structurally deficient even under normal, non-seismic loading conditions. Therefore, immediate repairs were undertaken that improved the normal load carrying capacity in the short term but were not intended to improve the seismic resistance of the structure. This work was completed in 1994. Subsequently, a consulting assignment was commissioned for development and assessment of a range of options for seismically upgrading the reservoir. Based on this work it was concluded that a retrofit of the existing structure or a complete replacement of the existing structure would be of similar cost, each approach having its own unique advantages and disadvantages.

Further design work was then put on hold in 1996 because adequate operational flexibility did not exist at that time which would enable the reservoir to be taken out of service for an extended period to carry out construction. With the completion of new water mains in 1999, operational flexibility of the regional water supply system was much improved. Therefore, in 2000, preliminary and detailed design were carried out by GVWD's prime consultant Sandwell Engineering Inc. First, a short list of options was developed, based on previous work, to determine whether a new reservoir or a seismic upgrade was more appropriate.

Numerous options were evaluated to mitigate deficiencies of the existing reservoir, including:

- Repair and retrofit of the existing reservoir, leaving the existing structure essentially intact.
- Total replacement of the existing reservoir with a new reservoir located essentially within the same footprint.
- Construction of a new reservoir(s) elsewhere in the park and outside the current footprint to replace the capacity of the existing reservoir.
- Variations and combinations of the above, including increases in storage volume as part of the remediation work.

The repair and retrofit options could economically address only specific deficiencies. The replacement options could address the specific deficiencies and in addition, economically provide operational and functional improvements to the reservoir. After extensive consultation with various stakeholders, it was determined, given the features desired in the future reservoir, that a new reservoir would be the best solution. Rationale included:

- Similar cost to upgrade
- Shorter construction schedule
- State-of-the-art technology throughout
- Operational flexibility
- Increase in capacity for emergency purposes.

Salient features of the final design:

- Seismic resistant roof design capable of supporting up to 12.0 kPa live load suitable to support loaded buses, truck access and vehicle parking.
- New seismic resistant concrete perimeter walls to replace the soil embankment as the structural element to retain the water volume while minimizing landscaping impacts.
- A 100 year life, resistant to the maximum credible earthquake (MCE) for Vancouver, i.e., 0.5g, with a 1/10,000 year return period. (Note that new buildings are designed for, 0.23g, with a 1 in 475 year return period)
- Provision of a seismic shut off system for the new piping to ensure containment of water following an earthquake, should existing buried water mains fail
- Optimized water quality by specific configuration and design of new inlet/outlet piping.
- Address constructability issues within the park, and minimize construction impact to the park users, local residents, park related businesses, and other park stakeholders.
- Reservoir designed with two independent cells, each with its own inlet and outlet piping
- Environmental issues addressed relating to noise, dust, vegetation, wildlife, surface runoff, and traffic
- Completion of the valve chamber and Cell 1 in the low demand period of September to May, to ensure at least 50% of the capacity was available for the following summer
- Sustainability principles to be incorporated throughout

SITE SPECIFIC GEOLOGY

Available geological information and previous site investigation work indicate that the perimeter embankment fill around the existing reservoir is underlain by foundation soils comprising dense till-like soils and weathered tertiary bedrock. Standard Penetration Tests carried out in these foundation materials

encountered refusal, which is defined as more than 50 blows for a penetration of less than 300 mm, indicating that the native materials are dense to very dense. Borehole investigations, also indicate that the bedrock surface dips gently to the south and southeast.

It is believed the original construction specification called for a zoned earth fill embankment containing an impermeable core, upstream sand and gravel shoulders, a downstream zone of rock fill and a concrete lining on the interior slope and on the base. In a study carried out in 1993, 12 bore holes were drilled into the embankment and downstream toe area, and the results indicated that the berm was not constructed as specified. Two zones of materials were encountered – a silty, gravely sand zone which forms the majority of the fill, and a zone with similar materials containing broken rock. The latter forms a surficial zone on the downstream slope.

Standard Penetration Test results gave values between 2 to 10 blows per 300 mm for the embankment fill materials, with some higher blow counts encountered due to the presence of broken rock. It was concluded that the embankment fill is generally very loose to loose. Further, this earlier study concluded that it was not clear from the borehole information or the observations from test pitting, that the broken rock was placed with sufficient density to provide adequate shear strength. During this 1993 study, piezometers were installed in most of the boreholes. Standpipe piezometers were installed initially in the sandy fill material or where there are hydraulic connections to this material. Pneumatic piezometers, with a more rapid response time, were used in the deeper natural ground. It was noted that the piezometer readings responded to changes in reservoir water levels, indicating reservoir leakage through the embankment.

SEISMIC DESIGN CRITERIA

The Little Mountain Reservoir is located within Seismic Zone 4 as defined by the 1995 National Building Code of Canada (NBCC) [2]. The estimated magnitude of the peak ground acceleration at the site from historical earthquakes that occurred between 1911 and 1990 was investigated. This data indicated that the maximum peak ground acceleration the site has experienced since 1911 is about 4 percent of gravity (.04g).

The Reservoir was analyzed and designed based on the seismic design performance levels specified in the GVRD Seismic Design Criteria [3]. The reservoir is a level 1 facility - defined as “critical to system operations, has little or no redundancy and may result in substantially reduced service for an extended period if a failure occurs”

The NBCC foundation factor F is not applicable here as site specific response spectra developed by Klohn Crippen [4] in 1992 for the 1:475 earthquake and the MCE, both with 5% damping, are used to establish the seismic acceleration (0.5% damping is used for the sloshing response). To remain consistent with the seismic upgrades recently completed for GVWD’s Vancouver Heights, Kersland, Cape Horn, Prospect and Central Park reservoirs, the MCE mean response spectra with 5% damping, for was maintained as the design spectra for this project. The spectral acceleration at various periods for MCE and in comparison to NBCC levels are shown in Figure 5. The design peak firm-ground acceleration for Operating Basis Earthquake (OBE) is associated with an earthquake of magnitude 7.5, an epicentral distance of 30 to 70 km, and a focal depth of 20 km. For the MCE event, an earthquake of magnitude 6.5, an epicentral distance of 10 to 20 km, and a focal depth of 10 km are assumed to be representative.

Based on previous studies, Table-1 shows seismicity levels that were chosen for seismic design at the reservoir site, to satisfy GVRD’s seismic design criteria:

Table-1 Seismic Design Criteria

Earthquake Designation	Probability of Annual Exceedance	Return Period (years)	Peak Ground Acceleration
Operating Basis Earthquake OBE Serviceability Criteria $R = 1$; $U = 0.6$, $I = 1$. Reservoir fully functional; elastic structural response; no additional leakage; no structural damage; some movement of embankment soil possible.	.0021	475	0.23 g
Maximum Credible Earthquake MCE Ultimate Design Criteria $R = 2$ (wall bending), $U = 0.6$; $I = 1$. Reservoir functional after minor repair; no catastrophic leakage of water; possible local shallow failure and significant movement of embankment independent of the reservoir.	.0001	10,000	0.5 g

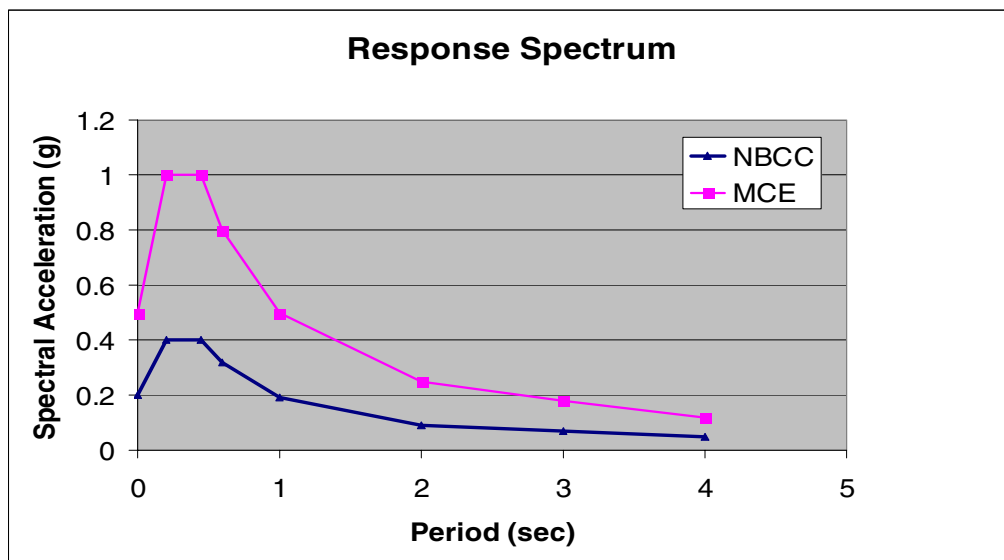


Fig. 5. Site Specific Response Spectrum used for LMRR

Vertical accelerations were assumed as 2/3 of horizontal acceleration which is standard practice in Vancouver and consistent with observed ground motions in recent earthquakes in California and Japan. The vertical acceleration was combined with the horizontal acceleration to calculate the hydrodynamic forces on the walls of the structure. The 0.5% damped spectra for convective response was calculated from 5% damped spectra using the Newmark and Hall [5] method. The spectral displacement, which is assumed constant for periods greater than 4 seconds, was used to calculate the sloshing response of the reservoir, which had a first mode response in the order of 20 seconds. Analyses indicated that while the foundation materials at the site are not liquefiable, the embankments were expected to liquefy and fail under the design OBE loading.

STRUCTURAL DESIGN CRITERIA

The reservoir was designed for water levels varying from full to empty with or without external backfill under static and dynamic conditions. The roof slab provides lateral support for the top of the perimeter and central walls. The preliminary design assumed that the reaction at the top of the wall was due to a simply supported wall (i.e. no or limited base fixity) with water on the inside and no soil on the exterior. The effect of the foundation stiffness was checked using the lower bound soil springs provided by the geotechnical engineer and found that the footing is not stiff enough to significantly reduce these roof reactions. In accordance with BS 8007 [6] the crack widths and steel stresses are calculated after the neutral axis is shifted for the combined tension and flexural moments.

Crack Control Criteria

The structure was designed to meet the following crack criteria:

1. B.S. 8007, max crack width = 0.2 mm flexural and tension, except top of roof slab in contact with waterproofing to CSA A23.3, exterior exposure, $Z=25$ kN/mm (approximately 0.33 mm flexural cracks) or
2. ACI 350 [7], normal exposure, $Z=115$ for all concrete except top of roof slab in contact with waterproofing to CSA A23.3, exterior exposure, $Z=25$ kN/mm.

Environmental Loads

The structure was designed for the following environmental loading differential and overall temperature ranges as follows:

- Exterior ambient: Minimum = -17.8°C , Maximum = 33.3°C
- Daily mean low = -14.5°C , Daily mean high 25.9°C
(based on Environment Canada records for Vancouver Airport for 1937 to 1999)
- Interior fluid contents: Minimum = 3°C , Maximum = 18°C
(based on water temperature measurement by GVWD at source)
- Moisture gradient: Based on interior 100% humidity and exposed concrete 70% relative humidity.
- Ground snow load $S_s = 1.7$ kPa and associated rain load $S_r = 0.3$ kPa

The roof slab was reinforced for dead and live load flexure with a minimum of 0.6% reinforcing to provide crack control and ductility under early thermal and shrinkage stresses. This percentage of reinforcing is the quantity of reinforcing which will have a tensile capacity greater than the tensile cracking strength of the concrete.

Typically in buildings, 0.2% temperature reinforcing is used, however this is inadequate in water retaining structures. It is noted that a water reservoir elsewhere had 5 mm wide thermal and shrinkage cracks in the roof slab because it was provided with only 0.2% temperature reinforcing in accordance with CSA A23.3 [8]. Later editions of CSA A23.3 recognize that 0.2% reinforcing may be inadequate and Clause N7.8.1 of the commentary states, *“The amounts of shrinkage and temperature reinforcement given in the previous standards have been found to be inadequate in preventing wide shrinkage and temperature cracks in slabs.”*

DESIGN OF WALLS

The dynamic soil pressures on the walls were calculated for a range of displacements. The soil pressure was selected to correspond to the flexibility of the wall. Detailed design was done using non-linear FLAC analysis to estimate the pressures for the MCE event. A ductility factor $R=2$ was used on all seismic wall forces except convective forces where $R=1$ in accordance with the recommendations of ACI 350 (draft). A

$U=0.6$ was used on all seismic loading conditions as per NBCC provisions. These preliminary ductility factors were reviewed when the final response spectra was selected. Radiused walls at the corners act primarily as hoop tension members in resisting the hydrostatic forces and were designed to PCA [9] recommendations for circular concrete tanks modified by Vitharana, Priestley, & Dean [10]. The maximum concrete hoop tension has been calculated to be approximately 2.2 MPa for 30 MPa concrete with a wall thickness of 600 mm.

The reservoir walls were designed for the combination of hydrostatic, hydrodynamic, static soil and dynamic soil pressures as noted below.

1. For exterior perimeter walls the following load combinations were used:
 1. DL + HYDROSTATIC + ROOF LIVE LOAD * #
 2. DL + STATIC SOIL + LIVE LOAD SURCHARGE + ROOF LIVE LOAD* [reservoir empty]
 3. DL + HYDROSTATIC + HYDRODYNAMIC (impulsive and convective) #
 4. DL + STATIC SOIL + DYNAMIC SOIL [reservoir empty]

* Roof live load applied in alternate bays to maximize wall moments
no support from soil assumed
2. For the dividing wall load combinations were:
Load cases 1 and 3 from above with water on either side or both sides simultaneously

The stability of the wall footing is based on either the maximum elastic seismic force or the wall overstrength moment to minimize damage to the foundations. The shear capacity at the base of the wall is based on the overstrength flexural moment to ensure a ductile behaviour in the event of seismic loads greater than the design forces.

Sloshing Uplift

Hydrodynamic forces were based on the Isaacson method [11] with the following modification. Vertical accelerations on the fluid forces were included in accordance with the New Zealand Standard [12] and combined with the impulsive and convective forces by the SRSS technique. Under seismic loading there is potential for uplift forces due to sloshing waves in the reservoir. Typically, water reservoirs are constructed with adequate freeboard to eliminate these uplift forces on the underside of the roof slab. However in order to maximize the available storage volume the freeboard was reduced to a minimum, consistent with operational requirements. The predicted sloshing height exceeds the design freeboard. Therefore the roof panels were designed for the sloshing wave impact uplift forces in accordance with values measured during testing by Kurihara, Musuko, and Sakari [13].

SHAPE OF THE NEW RESERVOIR - A SEISMIC DESIGN FEATURE

The size and shape of the proposed reservoir was established from the requirement of constructing the new reservoir within the bounds of the existing one. The new perimeter walls were built as close as practical to the existing walls to maximize reservoir volume. Allowing 150 mm \pm 75 mm for shotcrete on the face of the excavation as temporary support, 300 mm for back filling with a drainage layer of pea gravel and no-fines concrete behind the wall and a 600 mm thick wall, the dimensions of the new reservoir are 185.4 m in the east-west direction by 113.8 m in the north-south direction (inside of wall to inside of wall) with circular corners. A north-south wall divides the reservoir into two cells and has been located approximately mid-length between the new east and west perimeter walls. With an average water depth of 8.75 m, a total capacity of 174,700 cu.m, a 25% increased capacity was achieved. The control

valve chamber is a three storied building attached to the reservoir. The seismic control system inside the facility was designed to ensure that all relevant valves shut down if a major seismic event occurs.

Figure 4 shows a schematic rendering of the reservoir upgrade work

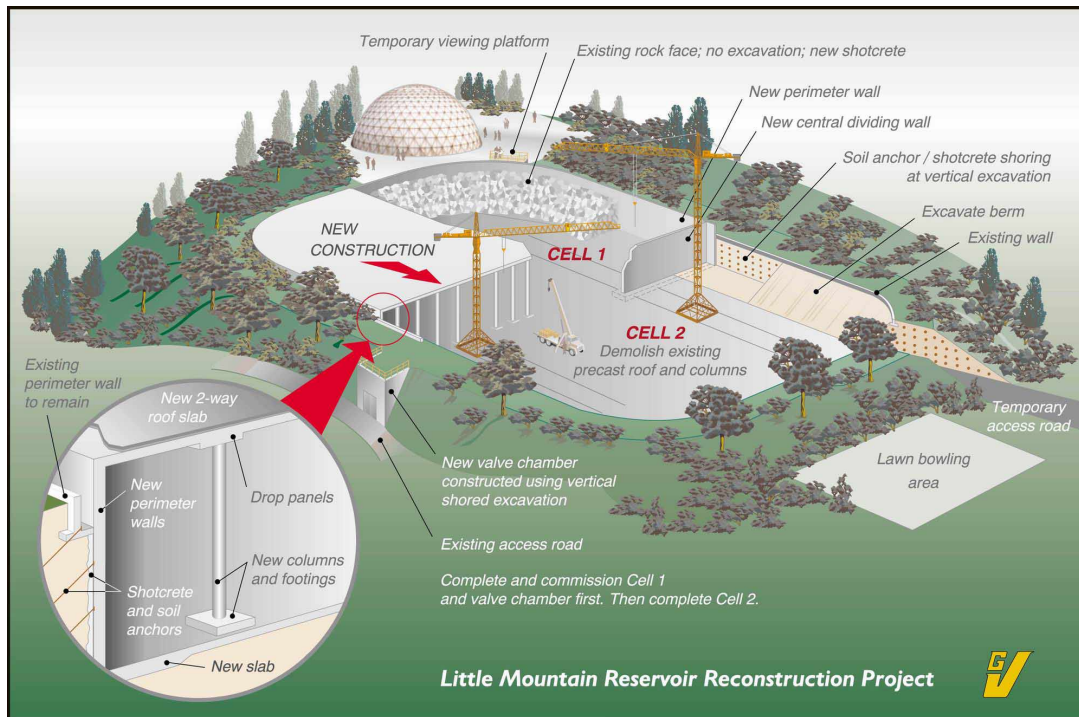


Fig. 4. Little Mountain Reservoir Seismic Upgrade - Project Schematic

DESIGN PHILOSOPHY AND ANALYSIS METHODOLOGY

The reservoir was designed as a fully monolithic reinforced concrete structure and the various structural elements were proportioned to withstand the combined action of vertical dead and live loads on the roof, lateral soil and water pressures on the walls, seismic loading and thermal movements. The reservoir roof is a 300mm thick flat slab with 250mm thick drop panels resting on 610mm diameter columns spaced approximately 7.32m apart, forming a square grid within the interior of the reservoir. The columns rest on 550mm thick reinforced concrete spread footings which are cast monolithically with the base slab, that varies in thickness from 250mm to 150mm. Along the perimeter, the roof slab rests on a 600mm thick wall, which in turn rests on a 500mm thick strip footing, which was designed to permit the wall to rotate at the base. The wall footing was also cast monolithically with the base slab. A 600mm thick central dividing wall separates the reservoir into two equal cells.

The perimeter walls are rigidly connected to their foundations. Where founded on till, the foundations rotate about the footing centerline by locally compressing the till at the edges. Where founded on rock, the foundations rotate by “rocking” on their edges. In this case, the foundations and a part of the base slab were debonded from the rock face to allow for this “rocking”. The base of the dividing wall was detailed as a hinge joint to allow it to rotate with respect to the foundation. The wall reinforcement was passed through the centre into the footing to provide the transfer of shear and minimal moment into the footing and the wall section is reduced to 200 mm for 25 mm above the footing to facilitate the rotation.

A three-dimensional finite element computer model of the reservoir was developed using the software SAP2000 (Fig.6). The reservoir roof and wall thermal movements were determined by subjecting this model to the design temperature range for expansion and contraction. The effect of the perimeter soil embankments was modeled by lateral wall springs. A compressible material added to the outside of the perimeter walls to an approximate depth of 4 m provides additional flexibility to absorb repeated expansion movements. This material was modeled by adding lateral springs of appropriate stiffness. The global effect of the thermal movements was analyzed using this three dimensional model. This model was also used to determine the global seismic response and the fundamental period of the structure, which is in the order of 0.25 sec.

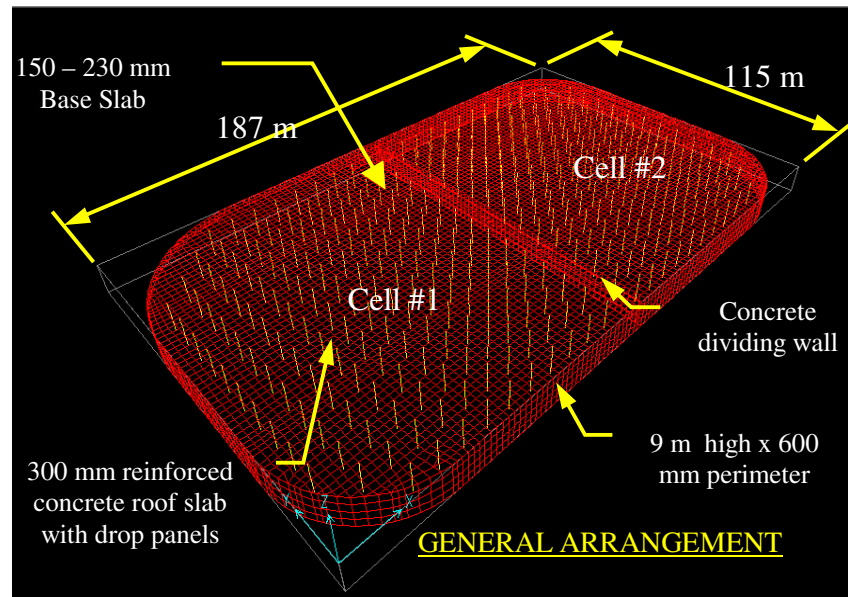


Fig. 6. SAP MODEL OF THE RESERVOIR STRUCTURE

A series of idealized two-dimensional computer models of the reservoir structure were also developed using SAP2000. These models, 7.32 m wide by 7 bay long slice of the structure, were used for the detailed analysis and design of the reservoir roof, walls, columns and base slab under the action of the design loads. The thermal effects are included by prescribing temperature ranges that produce the displacements determined from the three-dimensional model. Special cases including varying perimeter wall heights and curved walls were modeled individually.

The effects of the different design loads, including thermal effects, were superimposed in combinations that produce the maximum moments and shears at the various critical sections of the reservoir wall, roof and columns, for both static and seismic loads. Appropriate load factors were applied to the critical load combinations for the serviceability and ultimate limit states for the static loads and for the ultimate limit state for the seismic loads, as specified in the 1998 edition of the BC Building code.

Limiting concrete cracks to less than 0.2mm width, as per the provisions of BS 8007, was the governing criterion for the serviceability limit state at all critical sections, except for tension at the top of the roof slab, where the governing criterion is the crack control provision of CSA A 23.3 clause 10.6.1. The effect of axial forces is included in the crack control calculations for the roof and base slabs. Limiting concrete and steel stresses to the factored material stress (0.6×35 Mpa for concrete and 0.85×400 Mpa for steel) was the governing criterion for the ultimate limit state for both static and seismic loads. The reservoir walls, roof and base slabs and columns, were proportioned to resist the most severe condition. The base

of the reservoir walls were reinforced for shear perpendicular to the walls assuming $R=1$ for the MCE seismic condition, to account for the “overstrength” of the wall in bending.

Seismic loads arising from the self-weight of the structure, the superimposed dead loads on the roof (including 25% snow load allowance) and the out of plane soil and water pressures on the walls, are transferred to the orthogonal walls as in-plane shear loads in the direction of the earthquake, by the roof slab acting as a diaphragm. These in-plane shears, along with the out of plane shears at the base of the orthogonal walls, are transferred to the reservoir base slab and are dissipated into the ground by friction between the underside of the base slab and column foundations, and the supporting soil. In the north-west corner of the reservoir, where the perimeter wall is founded on rock at a higher level than in the rest of the reservoir, the out of plane shears are transferred at the base of the wall directly to the rock, by dowels embedded into the rock. The in plane shears in this area are transferred to the reservoir base slab through the reinforced shotcrete sloping face.

Insulated Structure

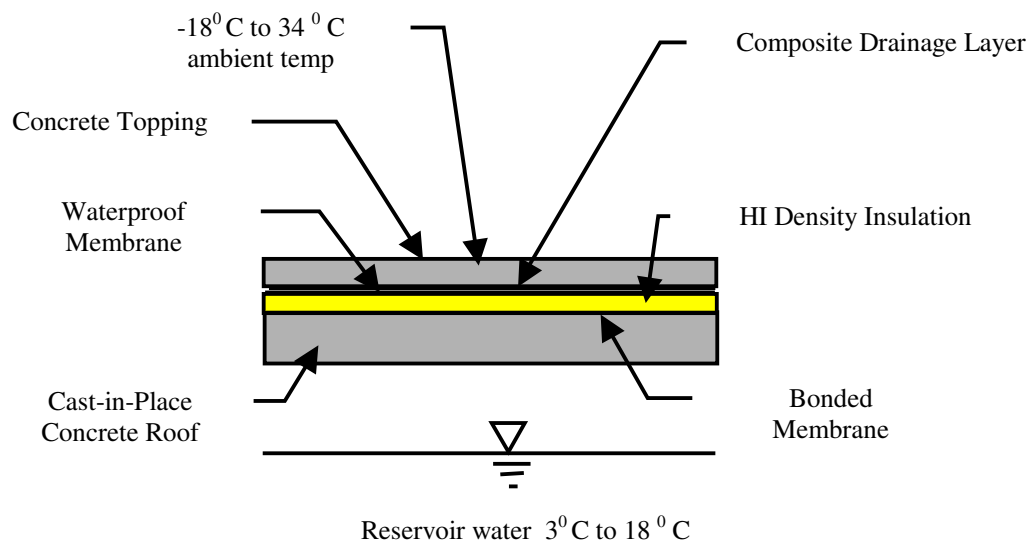


Fig. 7. RESERVOIR ROOF INSULATION
(schematic section of the roof structure)

To achieve what is believed to be the largest joint free concrete structure and the largest water storage reservoir in British Columbia, two innovative features were incorporated. First, the roof needed insulation to eliminate thermal effects once the structure was complete (Fig. 7). The specified rigid insulation was also capable of sustaining future repetitive loading of buses and other vehicles. Second, the construction sequence required the structure to be completed in a manner to leave a gap between the roof and the walls (Fig.8). Closing the 400 m long 1.2 m gap by structurally tying the roof to the remainder of the structure needed to be completed within a strict temperature range of 5°C to 18°C to ensure no unacceptable thermal strains were built into the structure. This was successfully achieved by specifying continuous temperature monitoring of the concrete during construction, use of cooling water on hot days, and temporarily insulating the roof and dividing wall during cold days. Furthermore, special compressible material was installed around the perimeter to enable the structure to expand and contract freely within the 5°C to 18°C range. The reservoir base slab and walls were poured in a checkerboard pattern to minimize the effect of thermal strains due to the heat of hydration.

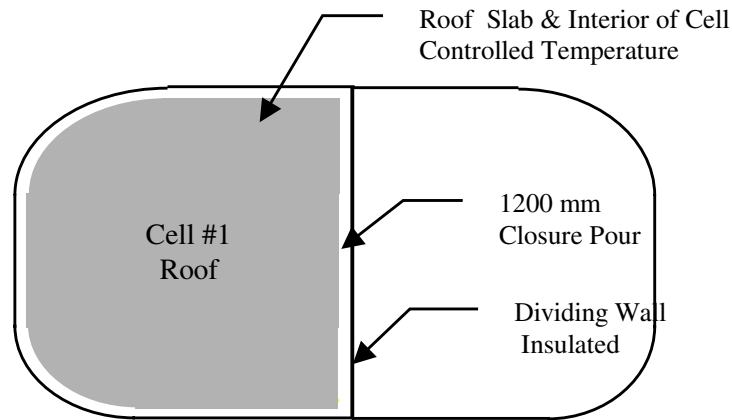


Fig. 8. THE RESERVOIR ROOF STRUCTURE (Cell #1 Schematic)

CONCRETE MIX DESIGN

The use of maximum feasible volume of fly ash was a key consideration in the design of this structure to ensure durability and to minimize future maintenance. Replacing cement with flyash also provides environmental benefits in terms of reduced green house gas emissions due to reduction in cement consumption. Research and case studies have addressed the issues related to usage of cement in concrete production by appropriately replacing it with suitable supplementary cementing materials (SCM). With regard to the permeability and durability, works by Malhotra and Mehta, [14] shows superior long-term performance for high volume fly ash concrete (HVFA) concrete. The amount of fly ash replacement was specifically designed for each component of the reservoir structure to enable placement compatible with the rebar detailing and placement size. A total of 16 concrete mixtures were developed with varying proportions of fly ash. In the Little Mountain Reservoir Reconstruction project, replacement of approximately 44% of cement has resulted in a reduction of 3700 (+/-) tonnes of CO₂ emissions, for the 28,000 cu.m of concrete placed. The quality of concrete finish achieved and the insignificant amount of cracks developed were attributed to the successful application of concrete with large volumes of fly ash (Fig. 9). Smooth finished concrete surfaces are an important requirement in reservoir construction to avoid potential bacterial growth in bug holes.

CONCLUDING REMARKS

The structural concept of the reservoir is a monolithic roof/wall/slab construction with no expansion joints, that is very efficient to resist earthquake loads from external soil pressures, internal sloshing effects, and inertial loading of the structure itself, in combination with the normal hydrostatic loading. Widely spaced construction joints were achieved during construction using full height (9m high) wall placements some 14.6m in length, base slab placements of 14.6m by 14.6m in plan area, and roof slab placements of some 850 sq m or 260 cu. m. each. The limited amount of joints, all water-stopped and sealed, is intended to reduce leakage potential and long-term maintenance.

Figure 10 shows a view of the site during construction in July 2003. Demolition of the existing reservoir was completed in October 2002 and after separating the rebar and concrete on site, they were recycled. The first Cell of the reservoir was commissioned in June 2003 as scheduled. The second cell was commissioned in November 2003 and the project was completed on schedule and within budget.



Fig. 9. CONCRETE FINISH (COLUMNS AND ROOF SLAB)

Little Mountain Reservoir has been integrated into the Park environment, and forms the foundation for a centre piece addition to the public recreation facilities of Queen Elizabeth Park, aimed at long term sustainability. The reservoir has been reconstructed by utilizing state-of-the-art technology to safely resist the maximum credible earthquake for the region, with needed additional capacity for emergency response, modern operating infrastructure and by adopting innovative and sustainable construction practices.



Fig. 10. LITTLE MOUNTAIN RESERVOIR CONSTRUCTION IN PROGRESS

In accordance with the Sustainable Region Initiative of the Greater Vancouver Regional District, the project team adopted methodologies that would address environmental, social and economic issues in a balanced manner. Extensive public consultation was undertaken prior to construction, to address the needs of affected parties, and it was continued through the demolition of the existing reservoir and construction of the new facility. The partnership-driven consultation approach and carefully crafted mitigation plans were designed to minimize social and environmental impacts. Partnership of agencies with diverse public mandates was achieved by communicating and sharing a vision of an integrated facility and the project was implemented by a partnering approach between the staff, consultants and contractors.

REFERENCES

1. Rogers, G.C. "An Assessment of the Megathrust Earthquake Potential for the Cascadian Subduction Zone", *Canadian Journal of Earth Sciences*, 1988, Vol. 25: 844-852.
2. National Building Code of Canada, National Research Council of Canada, 1995.
3. GVRD Seismic Design Criteria, CR-02-02DS-GEN-0100.
4. Klohn-Crippen, "Little Mountain Reservoir, Review of Seismic Retrofit Options"; Report to GVWD, 1994
5. Newmark, N. M. and Hall, W. J., *Earthquake Spectra and Design*, EERI Monograph, EERI, Oakland, 1982.
6. British Standards Institution, BS8007:1987, Design of concrete structures for retaining aqueous liquids, 1987.
7. American Concrete Institute (ACI), ACI 350R-89, Environmental Engineering Concrete Structures, 1989 and ACI 330R-92, Guide for Design and Construction of Concrete Parking Lots, 1992.
8. Canadian Standards Association (CSA), A23.1-00, Concrete Materials and Methods of Concrete Construction & A23.3-94, Design of Concrete Structures.
9. Portland Cement Association (PCA), *Handbook-Circular Concrete Water Tanks*.
10. Vitharana, Priestley, & Dean, *ACI Structural Journal*, May/98, American Concrete Institute.
11. Isaacson, Michael, "Assessment of Earthquake-Induced Hydrodynamic Loads on Reservoirs, *Report Prepared for GVWD and Sandwell Inc.*, January 1997.
12. Priestley, M.J.N., Davidson, B., Honey, G., Hopkins, D., Martin, R., Ramsey, G., Vessey, J., Wood, J., "Seismic Design of Storage Tanks", *Recommendations of a Study Group of the New Zealand National Society for Earthquake Engineering*, 1986.
13. Kurihara, C., Masuko Y., Sakari, A., "Sloshing Impact Pressure in Roofed Liquid Tanks", *Fluid Structure Vibration and Sloshing. ASME Pressure Vessels and Piping*, Vol 232, pp 19-24
14. Malhotra, V.M, "Making Concrete Greener with Fly Ash", *Concrete International*, Vol.21, No.5, May 1999, pp.61-66.