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Seismic Upgrade Techniques For Concrete Reservoirs

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ABSTRACT

Recent earthquakes have revealed a considerable vulnerability of urban water systems. The western part of British Columbia with Vancouver as a major urban center is the seismically most active region of Canada. To mitigate a risk of water supply disruption in a post-earthquake situation, the Greater Vancouver Water District has initiated seismic assessment and subsequent retrofitting of critical water storage facilities (reservoirs) in the Greater Vancouver area. Three reservoirs upgraded to date were originally constructed between 1928 and 1974. A set of performance criteria for various earthquake recurrence intervals, which exceed the seismic provisions of the current 1995 National Building Code of Canada, has been considered taking into account post-earthquake operability and the potential for catastrophic release of water. Analyses identified major deficiencies relative to the performance criteria. The reservoirs are generally of similar construction consisting of a reinforced concrete flat plate roof structure supported by the columns (behaving as a moment frame) independent of perimeters concrete cantilever walls that range from fully exposed to mostly buried. The paper outlines several retrofitting schemes considered for upgrade of the deficient reservoir structures, such as: i) new reinforced concrete shear walls, ii) modification of existing frame by installing new beams, and iii) seismic dampers installed at the roof-to-wall connection. Benefits of the retrofitting schemes have been compared in terms of expected performance such as post earthquake damage, under the design level seismic events and corresponding construction costs. Retrofit costs are compared to a new reservoir currently being designed and constructed to equivalent performance criteria.

INTRODUCTION

The Greater Vancouver Water District (GVWD) supplies water to approximately 1.8 million people, about half the population of British Columbia. The water distribution network includes 18 service reservoirs, critical for providing network storage and additional capacity during peak day demand. In 1992 the GVWD initiated a program to evaluate the vulnerability of those reservoirs that present a high life safety hazard or were identified as critical with respect to their ability to distribute water throughout the region. This paper presents the evaluation and upgrade of three reservoirs listed in Table 1.

Table 1. Reservoirs – Basic Information

Name	Year Constructed	Storage Volume	Year Upgraded
Vancouver Heights (original/expansion)	1928/1968	46 ML	1996
Kersland (Units 1 & 2)	1955/1959	79 ML	1997
Central Park	1974	36 ML	1998

SEISMIC EVALUATION AND RETROFIT CRITERIA

The GVWD considers transmission of water an essential service - a "lifeline" which is expected to sustain earthquake effects with minimal disruption. The GVWD developed their own performance - based seismic design criteria for the water transmission system as noted below:

- <u>EQ - 1 Service Level Earthquake (SLE)</u> - a seismic event with a probability of annual exceedance of 0.01 (a "100 year return period"), with an estimated firm ground PGA of 0.08 g. Reservoir is expected to demonstrate elastic response with no damage.

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- EQ - 2 Operating Basis Earthquake (OBE) - an M7 to M7.5 event occurring at a distance of approximately 30 to 70 km from the site, with an estimated firm ground PGA of approximately 0.19 g, corresponding to a probability of annual exceedance of .0021 (a "475 year return period"); equivalent to NBCC 1995 design level earthquake. Reservoir is expected to remain operational but may experience minor cracking and moderate leakage, and it should be repairable within a year.
- EQ - 3 Maximum Credible Earthquake (MCE) - a near-field M 6.5 event occurring at a distance of approximately 10 km from the site at a depth of 10 km, with an estimated firm ground PGA level of 0.5 g. Reservoir might experience extensive damage, however, no sudden, catastrophic release of water is expected to occur from the containment structure.

A set of response acceleration spectrum curves corresponding to the mean confidence level was developed by GVWD for both the OBE and MCE for the purpose of seismic analysis. In addition, a set of three design spectrum-compatible artificial time histories was generated to serve as input for time domain dynamic analyses (Nikolic-Brzev and Sherstobitoff, 1999).

Earthquake-induced impulsive and convective sloshing forces on the perimeter walls were generally evaluated using traditional procedures, except in case of the Central Park project, where the effects of sloping sides and partial height walls were considered as proposed by Isaacson (1997). Dynamic soil-structure interaction effects were evaluated using Mononobe-Okabe type procedures for yielding walls and recent work by (Wu, 1996) for rigid walls (relevant at reservoir corners). Non-linear dynamic analysis using the program FLAC was also used to determine movement of the perimeter walls due to deformation of the supporting soil media.

DESCRIPTION OF A TYPICAL RESERVOIR STRUCTURE

The structures generally consist of a basin excavated into the existing dense "till-like" soil, lined with concrete over the flat central area and sloping sides. At the top of the slopes are short cantilever concrete walls that retain backfill and support the perimeter of the roof slab. The two-way roof slab is supported by internal columns and the perimeter walls. The roof slab is structurally independent of the walls in order to accommodate thermal expansion. Typical details are presented in Figure 1.

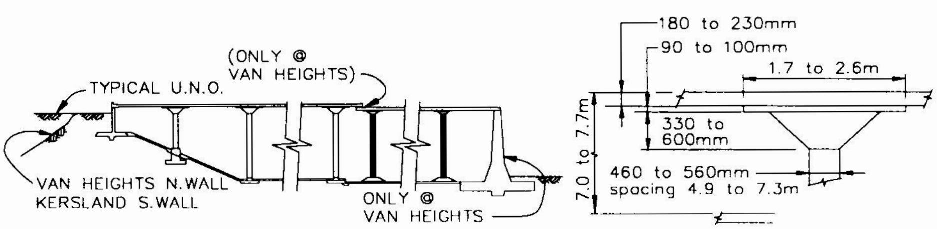

Figure 1. Typical reservoir cross section and details.

THE VANCOUVER HEIGHTS RESERVOIR CASE STUDY

The 1928 structure has no expansion joints within its plan area and was structurally connected to a full height water retaining cantilever east wall. The ground sloped steeply away from the buried north wall. The 1968 expansion increased the plan area with 3 new structurally independent roof sections. Proximity of dense housing located downhill of the north and east walls of the reservoir made evaluation and retrofit of these walls to the EQ-3 criteria especially critical.

The seismic evaluation of the 1928 portion indicated a number of critical structural deficiencies, such as: i) no bottom reinforcing in the roof slab at column locations, ii) roof slab connected only to the east wall, thus creating high torsional eccentricity, iii) columns of a limited flexural capacity and nominal ductility performance, iv) a deteriorated roof slab with limited shear capacity, v) a water retaining east wall grossly under reinforced, with potential for catastrophic failure at MCE loading, and vi) north wall with poor foundation conditions and inadequate flexural reinforcing. The 1968 portion was significantly better, with deficiencies similar to iii) above, and with short perimeter columns (high relative stiffness compared to the interior columns), attracting a significant load without adequate capacity or ductility.

Concepts were developed to upgrade the roof structures only to an OBE (importance factor I=1.5) level and the potentially hazardous north and east walls to the MCE level. The philosophy was that even if the roof structure suffered local collapse or serious damage at the MCE level, the perimeter walls would be able to maintain water retaining function.

In the 1928 portion the roof was made structurally independent from the east wall by removing a 70 m strip of roof slab 150 mm wide. The roof slab, independent from the east wall, was torsionally symmetric, however its strength and ductility still remained inadequate. Options to retrofit the roof structure included new shear walls, slab-to-roof concrete or steel trusses, and fibre wrapping of the columns. The square pattern of shear walls was selected as the most effective and economical. The steel and concrete trusses were effective, but estimated to be 10% and 25% more costly. Fibre wrapping would increase ductility but could not offset the extremely low level of column flexural reinforcing and inadequate splice lengths.

The pattern of shear walls is illustrated in Figure 2. The shear walls were designed as ductile structures with an R value of 3.5 (per NBCC 1995), with height/length aspect ratio in the range of 1.5. The walls were located to optimize the limited roof slab diaphragm capacity. To collect/distribute the shear to the roof slab, concrete "drag struts" were provided complete with adhesive anchors doweled to the underside of the slab. To achieve adequate sliding and overturning resistance in the foundations, 12 m long #18 soil anchors were used tensioned to 700 kN.

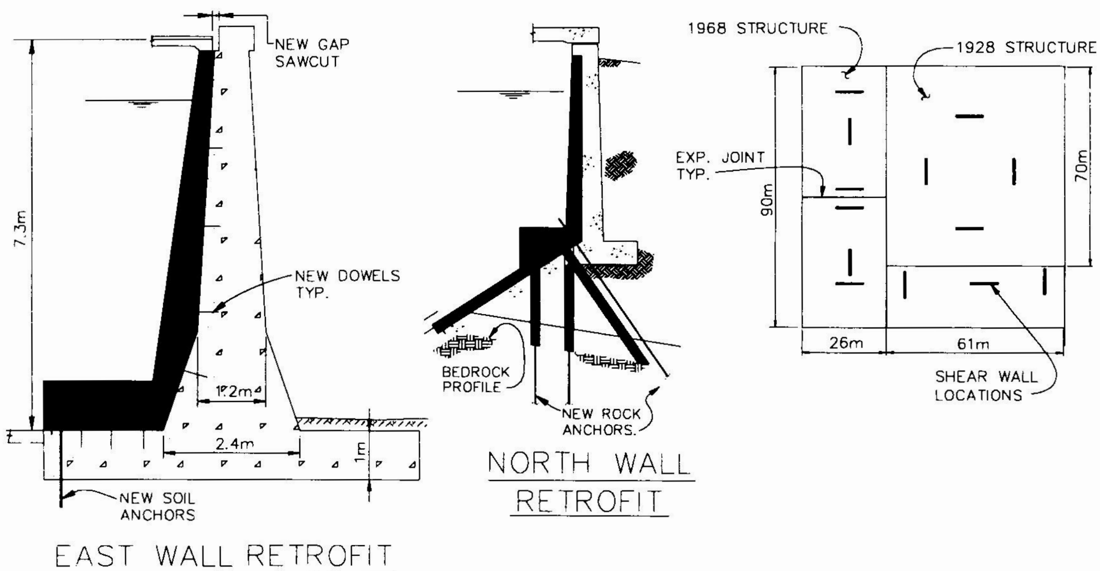

Figure 2. Vancouver Heights reservoir plan indicating new shear walls locations and retrofit of the existing walls.

The east wall was retrofitted with a new heavily reinforced layer of concrete on the interior face, doweled to the existing concrete (see Fig. 2). The interior portion of the footing was thickened and 12 m long post-tensioned anchors similar to those used in the shear walls were provided to increase sliding resistance, reduce footing toe pressures, and enhance the overturning capacity. The north wall was retrofitted in a similar manner, using soil anchors as tension-compression piles founded on bedrock to provide complete vertical and lateral support (see Fig. 2).

Analysis of the upgraded roof structure indicated that deformations (including soil anchor flexibility, dynamic soil deformation, inelastic shear wall deformation) were reduced to a level to ensure elastic response of the existing roof structure during a design earthquake event. The damage for the OBE x 1.5 event is expected to be limited to local cracking/yielding at the shear wall bases and local slab damage around the shear wall drag struts, thus meeting criteria for an EQ-2 event. The damage state for the walls for the MCE is expected to be limited to extensive cracking and possibly local yielding at the base of the walls, thus meeting criteria for an EQ-3 event.

KERSLAND RESERVOIR CASE STUDY

The 1955 roof structure (Unit 1) consisted of a one-way slab system with one expansion joint within its plan area, as shown in Figure 3. The 1959 expansion (Unit 2) followed the typical flat slab construction practice and consisted of 5 structurally independent roof sections. The ground slopes away from the buried south walls, similar to the north wall of the Vancouver Heights Reservoir. In this case, however, housing is remote from the south walls and a sudden release of water from the south walls was not considered to be catastrophic and the criteria related to an EQ-3 event were not deemed appropriate in this case.

The seismic evaluation of the 1955 portion indicated the following deficiencies: i) roof joists and beams with inadequate flexural capacity to act as a moment frame, ii) columns with inadequate flexural capacity, iii) south wall with inadequate flexural reinforcing. In addition to the above listed deficiencies ii) and iii), the 1959 portion had no bottom reinforcing steel in the roof slab at the column locations.

Due to the large variation in compaction of backfill against the south walls and varying ground water level, two extreme soil structure interaction scenarios were allowed for in the design: i) liquefaction and flow slide failure of the soil causing total loss of support, and ii) local deformation of the soil creating a "gap" between the wall and the soil.

The retrofit objective was to upgrade all components of the reservoir to an OBE (I = 1.5) level. The philosophy was that loss of water resulting from a 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.

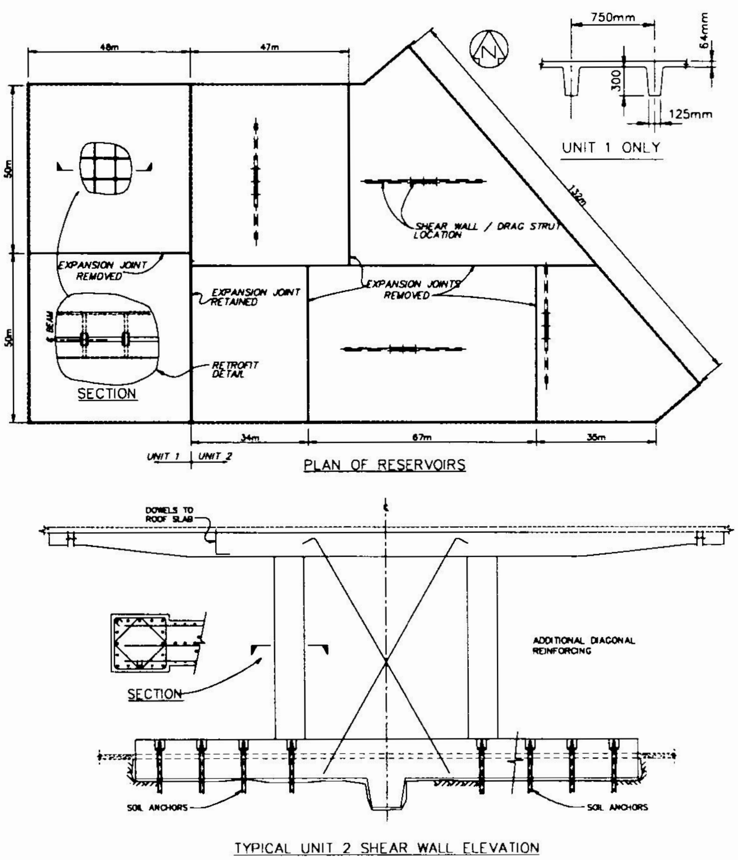

Figure 3. Kersland reservoir plan and shear wall elevation, including typical details.

For the 1955 roof structure, a shear wall option was discounted due to the very limited in-plane shear capacity of the roof slab. The favored option traded-off less than target performance against lowest cost. New beams added at the one-third height of all columns in both directions created a stiffer moment frame that limited deformations such that the existing structure remained elastic. The new beams were sized and detailed such that yielding will be initiated in the beam, just outside the new connections to the columns. To ensure transfer of very high stresses in the new beam-column joint, a 2 m high by 250 mm thick heavily reinforced concrete jacket was required. Due to the limitations of the existing beams and columns it was possible to meet the EQ-2 criteria with the importance factor (I) value of only 1.2. To obtain the I value of 1.5, a two fold cost increase would have been required.

The moment frame option was assessed for the 1959 roof structure but proved to be uneconomical. The selected shear wall option was essentially the same as used for the Vancouver Heights Reservoir but with the following differences: i) all 5 roof slab segments were connected together to create one large roof diaphragm; this was feasible due to the better condition of the slab, and significantly higher levels of in-plane reinforcing, ii) each of the four shear walls were designed to sustain significantly higher loads with an aspect ratio of approximately 0.95; to ensure a flexural failure mode and to prevent shear failure, diagonal shear reinforcement was provided.

The walls in the 1955 section were adequate for an OBE (I = 1.2) level, similar to the capacity of the retrofitted roof structure, and as such no further reinforcing was added. A 2 m high section of the south wall in the 1959 portion, which was thickened to augment the deficient flexural capacity at mid height. The new vertical reinforcing was not connected to the existing footing, as this would have increased the overstrength moment capacity of the wall to a point where the footing capacity and bearing capacity would have been deficient.

The damage state for the 1955 roof for the OBE x 1.5 event is expected to be concentrated primarily in the new beams, with limited damage to the existing columns and beams. The damage state for the 1959 roof structure at the OBE (I = 1.5) level is expected to be similar to that described for the Vancouver Heights Reservoir.

CENTRAL PARK RESERVOIR CASE STUDY

The roof structure has one expansion joint within its plan area, as illustrated in Figure 4. The critical deficiencies in the roof structure were as follows: i) discontinuous bottom slab reinforcement at the column locations, ii) lack of shear reinforcement at a critical slab perimeter around the columns, iii) inadequate lateral confinement of the column reinforcement, and iv) inadequate rebar development length at the column-to-slab connections.

Concepts were developed to upgrade the roof structure to an OBE (I = 1.5) level and the perimeter walls to the MCE level. Although the reservoir is buried on all four sides to within less than a meter of the roof and catastrophic release of water is not possible, upgrade to the MCE level was deemed necessary as this reservoir is considered critical in the redundancy and transfer ability of the overall water transmission system.

Several seismic upgrade options were evaluated, similar to those outlined in the previous sections. An alternative, less conventional option - incorporating seismic dampers - was ultimately selected because of lower construction costs and a considerably reduced construction schedule as compared to the other options. The damper option was estimated to be 12% less costly than the least cost shear wall option. Also, the majority of construction work can be carried out exterior of the reservoir, leaving the reservoir operational. Details of the damper retrofit scheme are discussed by Nikolic-Brzev and Sherstobitoff (1999).

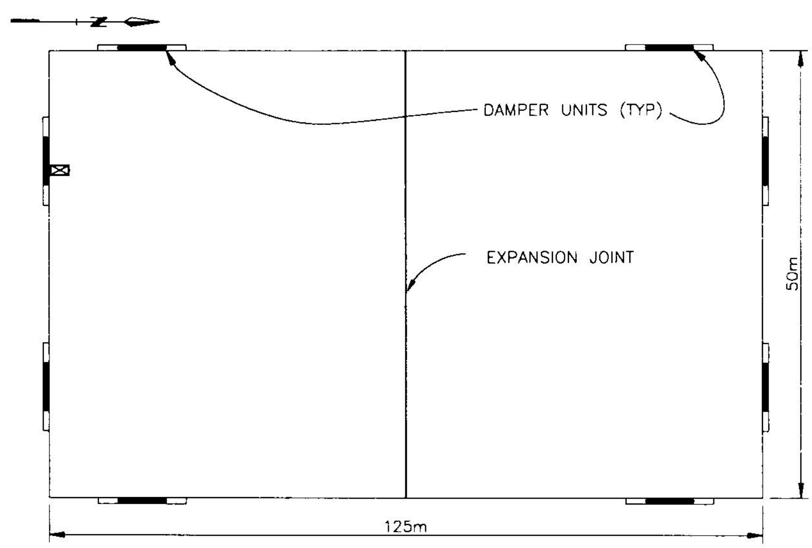

Figure 4. Central Park reservoir plan including damper locations

The roof structure was retrofitted by connecting the two halves at the expansion joint to create one roof diaphragm, and connecting the perimeter of the slab via dampers to the perimeter walls. Four dampers on each side, in pairs of two, were aligned parallel to the walls. Both friction and viscous dampers were evaluated, and both were able to provide adequate damping and energy dissipation to limit the deformation of the existing roof structure to an acceptable level (approximately 30 mm to maintain elastic behaviour). Long steel plates with adhesive anchors distributed the damper loads to the roof slab, and a new concrete beam, doweled to the existing wall distribute loads to the walls.

Dynamic time domain non-linear analysis of the soil supporting the perimeter walls indicated that the soil deformations for an MCE event are expected to produce movement at the top of the perimeter wall in the order of 50 mm horizontal (inwards) and 7 mm vertical (downwards). This relative deformation, plus expected thermal deformation, must be accommodated by the dampers themselves. Failure by the dampers or their connections to accommodate this relative movement could result in the roof structure behaving like the original structure and suffering extensive damage or collapse. Ultimately non-linear viscous dampers were selected for the project. Local wall reinforcing, similar to that described for the Kersland Reservoir south wall, was required on 3 perimeter walls and applied using silica fume shotcrete.

STRUCTURAL UPGRADE COST SUMMARY

A cost summary is presented in Figure 5 for the three reservoirs discussed, using costs based on the completed construction projects. The retrofit costs are presented in comparison to the estimated "all-in" costs (including site prep, piping, etc.) of the new Grandview reservoir, designed to the same criteria and constructed in 1998.

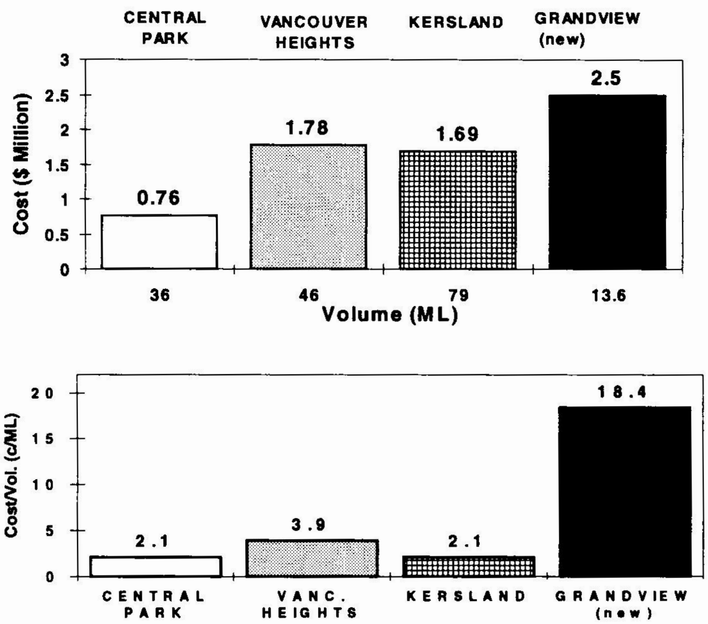

Figure 5. Cost estimate vs. total volume and cost per unit volume for the reservoirs studied.

CONCLUSIONS

The seismic upgrade of concrete reservoirs, to higher than current building code performance criteria, can be accomplished economically in a variety of ways. Three case studies were reviewed incorporating new internal shear walls, an enhanced internal moment frame, or new external seismic dampers for the roof structure upgrade and wall thickening (cast-in-place or shotcrete) for the perimeter wall upgrade. The upgrade costs in the case studies varied from 11% to 20% of the cost of a new reservoir with the same general seismic performance objectives.

Based on the three case studies compared, the use of seismic dampers in an upgrade provides the highest seismic performance, (i.e. least post-earthquake damage) at the least cost.

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