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Seismic Upgrading of Water Storage Facilities in British Columbia

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ABSTRACT

Water storage facilities are classified as critical lifeline structures. Many reservoirs in British Columbia were designed prior to the implementation of modern seismic standards and are vulnerable to damage or collapse in a major earthquake. The paper describes techniques the author has used to evaluate and retrofit existing water storage facilities to post disaster standards.

Modelling techniques include dynamic finite element computer analysis of the soil-structure and fluid-structure interaction. Dynamic fluid forces are modelled based on the Housner method, modified to include dynamic amplification of the impulsive forces. Hydrodynamic sloshing forces are also evaluated utilizing recent research from Japan. Critical structures were evaluated for both an operating basis earthquake (OBE) and a design basis earthquake (DBE) to ensure they will remain functional.

Lessons learned from retrofitting of existing reservoirs are applied to both the design of new water storage facilities and the retrofit of older facilities. The paper demonstrates that the seismic resistance of many existing reservoirs can be upgraded cost-effectively.

INTRODUCTION

The objective of this paper is to illustrate some techniques we use to retrofit water reservoirs and compare the cost of retrofitting with the cost of replacement to demonstrate that many reservoirs can be economically upgraded. The paper also describes some of the analysis methods we use in evaluating earthquake response of the reservoirs.

Dynamic fluid pressures are calculated utilizing the Housner method (TID-7024, 1963) modified for flexible walls and including the effects of vertical accelerations. There are three predominant types of dynamic fluid pressures on water reservoirs: impulsive, convective and vertical acceleration effects. Structure response can amplify the impulsive and vertical acceleration generated fluid pressures. We calculate the amplified impulsive forces by modelling the structure with added effective-fluid-masses attached to the walls. Typical at grade or below grade reservoirs do not amplify the convective forces and convective pressures are calculated from the fundamental sloshing mode assuming rigid walls. The fluid pressures produced by vertical accelerations are modelled by the New Zealand method (Priestley et.al.1986). The vertical acceleration pressures, impulsive pressures and convective pressures are combined by the SRSS method. Figure 1 illustrates the impulsive, convective and vertical acceleration pressures on a typical rigid reservoir wall.

We calculate the elastic earthquake forces on the reservoir and its components and assigned force reduction (or ductility) factors based on their capacity to absorb energy. This method allows us to develop capacity demand ratios for each component and failure mechanism.

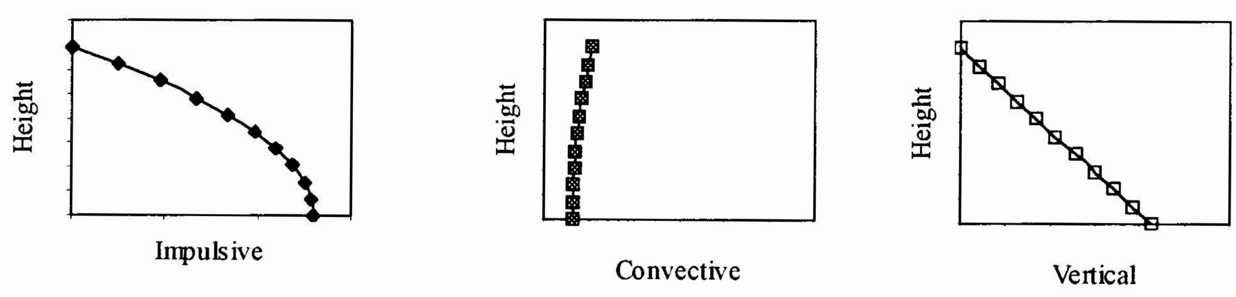

Figure 1

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CASE HISTORIES

Helgesen Reservoir

The Capital Regional District's Helgesen Reservoir supplies water for domestic use and firefighting in the community of Sooke on Vancouver Island. It is a rectangular concrete reservoir with a storage capacity of 2275 cubic metres of water. The reservoir was upgraded in 1997 to meet the performance criteria shown in Table 1.

Table 1

Design Earthquake	Return Period	Performance Level	
		Collapse Prevention	Operational
OBE	1:475		YES
DBE	MCE	YES	

Collapse prevention includes no uncontrolled release of water. The operational performance level includes limiting earthquake forces to the elastic or near-elastic range to minimize damage to the structure and designing the mechanical and electrical systems to remain operational. We adopted maximum credible earthquake (MCE) ground motions for the design basis earthquake (DBE) at this site because the post-disaster requirements of the building code specified short period spectral accelerations of almost the same magnitude as the MCE.

The roof slab was constructed as a conventional two-way flat slab and provides lateral support for the walls. The operating water level was 300 mm from the underside of the roof, which provided inadequate freeboard for the predicted sloshing waves. We calculated sloshing wave impact forces on the underside of the roof slab based on Japanese testing (Kurihara et.al. 1992 see Appendix). The roof was predicted to fail, with the resulting loss of support for the walls causing an uncontolled release of water.

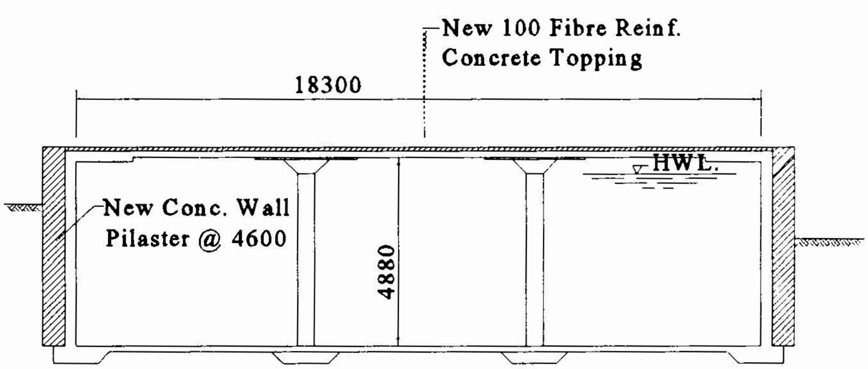

Figure 2

The roof was upgraded by casting a fibre-reinforced bonded reinforced concrete topping on the existing reservoir roof. The fibre-reinforcing increased the capacity of the system to absorb impact energy and the conventional reinforcing provided additional flexural capacity to resist uplift forces. The reservoir walls were also deficient in shear and additional capacity was added by casting reinforced concrete pilasters anchored to the roof and base slab to provide additional shear capacity in the system (see Figure 2).

The seismic upgrading for this reservoir cost approximately $100,000.

Towers Reservoir

The Towers reservoir, operated by the Greater Nanaimo Water District, is a 800 cubic metre storage capacity, circular reinforced concrete standpipe. It supplies potable water and provides fire protection to a pressure zone in the City of Nanaimo. The reservoir was retrofitted in 1998 to the post-disaster criteria of the National Building Code-1995 with an effective peak ground acceleration (PGA) of 0.3 g. The reservoir consists of a concrete circular shell with a flat concrete roof slab supported by the perimeter wall and a single centre column.

The reservoir shell was not connected to the foundation and the only lateral restraint for seismic forces was a 90 mm high curb. We analyzed the reservoir for seismic forces based on the modified and corrected Housner model. The reservoir shell is predicted to uplift 200 mm in the design earthquake. This amount of movement will fail the waterstop at the base of the wall and allow the shell to slide off the foundation.

We provided uplift restraint by casting a new reinforced concrete base slab anchored to the walls which resisted overturning by utilizing the weight of the contained water. The new slab is predicted to uplift 25 mm under the design earthquake, which moderately reduces forces in the vertical wall reinforcing. The piping connections were relocated to the exterior and flexibility was provided with articulated couplings (see Figure 3).

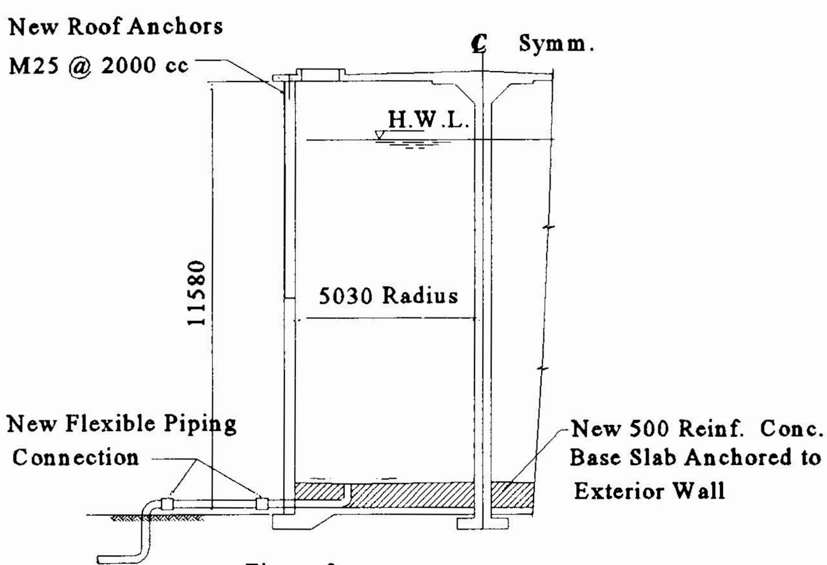

Figure 3

The roof was laterally unrestrained except by nominal resistance provided by the centre column. The addition of dowels in sealant filled holes provided lateral roof restraint with some accommodation for differential thermal movements. Other upgrading included new lateral supports for the interior overflow piping. Forces on the overflow piping were calculated by utilizing an added effective mass of water equal to the diameter of the pipe (Newmark-1970). We also lowered the operating water level to minimize sloshing impact forces on the underside of the reservoir roof slab.

The seismic upgrading cost of the project was $70,000.

Noon's Creek Reservoir

Noon's Creek Reservoir is a circular concrete water tank located in the City of Coquitlam. The tank has a storage capacity of 2275 cubic metres and was seismically upgraded in 1998. The performance criteria adopted for this reservoir was a design basis earthquake 1:1000 year event with topographic amplification effects due to its location on a steep hillside.

The concrete roof is a flat slab with drop panels and bears on four internal columns and a sliding joint at the top of the wall. The roof is laterally supported by four internal columns constructed integrally with the roof. The columns had limited ductility and the column-to-roof connections and column-to-footing connections were predicted to fail. The roof was upgraded by providing anchoring to the reservoir walls with dowels which permitted differential thermal movement. They also permitted the roof to uplift to release energy due to sloshing wave impact.

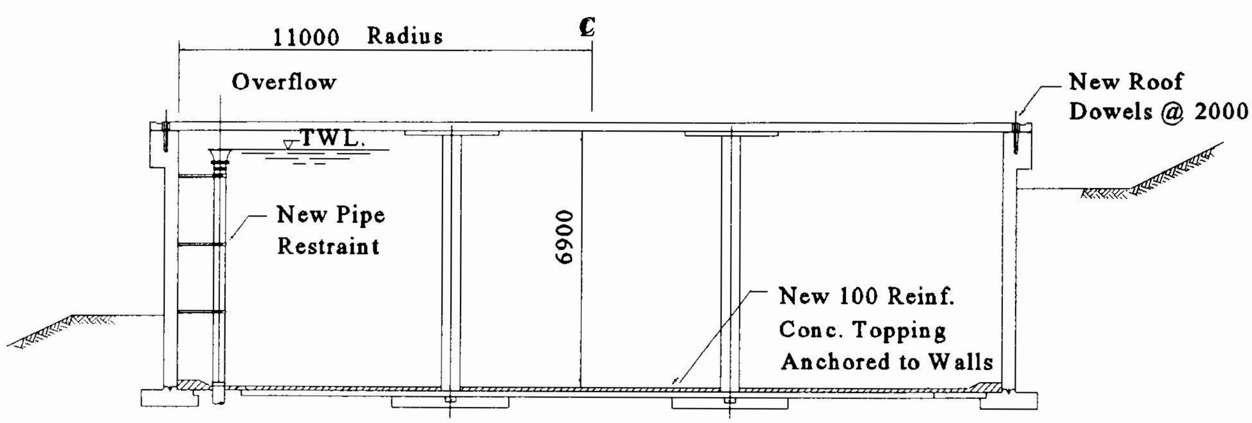

Figure 4

The reservoir floor slab was constructed with no structural connection to the perimeter footing. Therefore lateral forces on the tank walls were resisted by the perimeter footing bearing against the floor slab. This produces ring tension in the footing and base of the shell. Unbalanced static and dynamic soil pressures and dynamic fluid pressures produced large sliding forces on the footing, which were restrained by the floor slab. This overstressed the footing and base of the shell in ring tension. Finite-element modelling predicted shear in the wall-to-footing joint to exceed the capacity.

The solution was a reinforced floor topping which was anchored to the walls and footing to transfer the seismic forces directly from the shell into the base slab utilizing a shear mechanism rather than ring tension (see Figure 4).

The seismic analysis predicted that even with a monolithic floor slab the reservoir would slide. We calculated 50 mm of movement utilizing a Newmark sliding block analysis. (Franklin et.al. 1977). The piping connections were upgraded to accommodate this movement. The reservoir seismic upgrading cost was $50,000.

<u>Knox Mountain Reservoir</u>

The Knox Mountain Reservoir in the City of Kelowna is a twin cell reservoir with a combined storage volume of over 10,000 cubic metres. Each cell was constructed over fifty years ago with a masonry rubble wall on the downhill side. The water level had been reduced to reduce the seismic demands on the walls. To restore the reservoir to its original capacity and reduce leakage the deteriorated masonry walls were replaced with new reinforced concrete walls in 1998 (see Figure 5).

By adopting a capacity-demand procedure we were able to provide a high level of seismic resistance to the new walls at minimal additional cost. The performance criteria selected for this reservoir was no collapse or uncontrolled release of water for the maximum credible earthquake (MCE) with a PGA of 0.2 g.

We detailed the base of the wall as a ductile hinge and utilized capacity design principles to prevent a non-ductile failure in the adjacent components. The ductility capacity of the wall was assessed based on testing at UC Irvine (Haroun, et.al. 1994). The shear demands on the wall were calculated based on over-strength moment of the ductile hinge and the base of the wall reinforced with shear links in the ductile hinge zone. The foundation and rock anchors were also designed for the over-strength moment capacity of the wall.

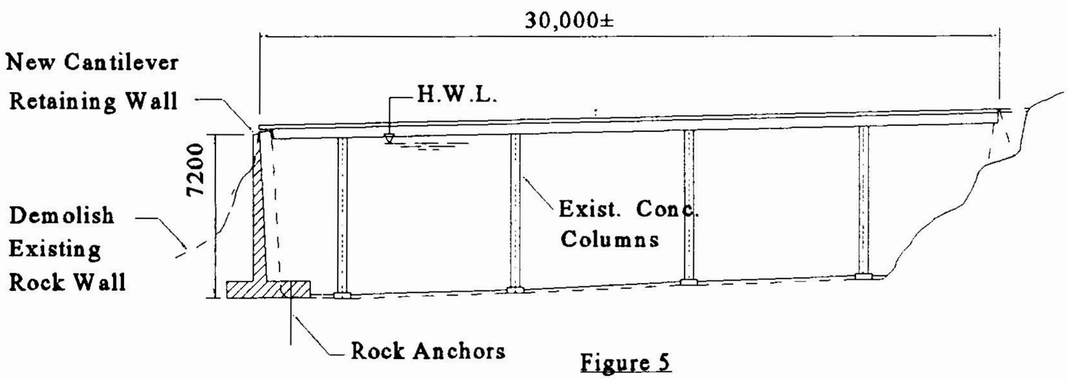

Figure 5

We provided a minimum vertical reinforcement of 0.8 % of the gross section to ensure ductile performance. (Priestley et al. 1996) This ensures that the reinforcing flexural capacity exceeds the cracking moment of the concrete by a suitable margin and includes an allowance for an increase in the concrete tensile strength under dynamic loading conditions. The foundation dowels were spliced above the ductile hinge zone. The wall to foundation joint was detailed to ensure the moment capacity of the joint would exceed the ductile hinge capacity. This included providing additional hook embedment, providing a development length on the hook tail in the footing and orienting the hooks to limit tensile forces in the joint.

The seismic upgrading portion of this project cost approximately $250,000.

SUMMARY

Table 2 summarises the cost of reservoirs which we have retrofitted including one currently in detailed design. The retrofit price per cubic metre of storage capacity has varied from $5/cubic metre to $100/cubic metre, with an average of $12 cubic metre. In general, the smaller capacity tanks were proportionately more expensive to upgrade than the larger tanks. Other factors affecting the cost were the original structural concept, material condition, type and age of construction, the seismic performance level, and the seismic zone.

Table 2

Name of Reservoir	Storage Capacity (m^3)	Approximate Replacement Cost	Upgrading Cost	Upgrading as % Replacement Cost
Foster	13700	$3,000,000	$50,000	2%
Knox	10000	$2,500,000	$250,000	10%
Dilworth	11400	$2,500,000	$50,000	2%
MacMillan	4550	$1,000,000	$30,000	3%
Helgesen	2275	$600,000	$100,000	17%
Towers	800	$250,000	$70,000	28%
Noon's	2275	$500,000	$50,000	10%
Scott Creek	4550	$1,000,000	$0	0%
TOTAL	49550	$11,350,000	$600,000	5%

RECOMMENDATIONS

New reservoirs can be constructed to provide a high level of seismic resistance for minimal cost if seismic resistance is considered at the concept stage. We recommend that some or all of the following measure be incorporated to increase the seismic resistance of concrete water reservoirs.

- Use "push-over" type analysis to determine failure modes
- Provide a load path to the foundation and detail components for ductility
- Provide adequate flexibility in piping connections at structure interface
- Restrain internal piping for seismic forces
- Provide adequate freeboard or design for uplift on roof.
- Provide lateral restraint or lateral load resisting system for roofs
- Construct walls and floor slabs monolithically with foundations

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APPENDIX

The following procedure was used to calculate the uplift impact force on the underside of the reservoir roof slab. The sloshing wave impact force on the underside of a rigid roof is a function of the momentum of the liquid impacting the roof. Kurihara et.al. (1992) and Kobayashi (1980) have developed equations for the impact pressures based on the elevating velocity of the sloshing wave.

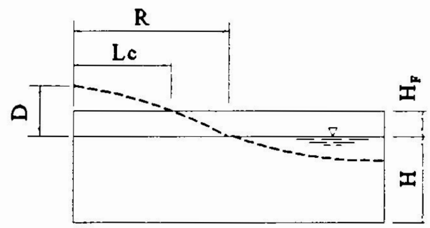

Figure A-1

The elevating velocity, v, in m/sec is calculated from equation (1) assuming sinusoidal sloshing motion. The maximum wave height D in metres and the fundamental sloshing period, T in seconds is calculated from a Housner analysis or other equivalent method assuming the tank has no roof. H_F is the available freeboard height from the surface of the fluid to the underside of the roof in metres. The contact length of the wave, L_C with the underside of the roof in metres is calculated by equation (2). R is the radius of the reservoir in metres.

(1) $$v := D \cdot \frac{2 \cdot \pi}{T} \cdot \cos\left[\sin^{-1} \cdot \left(\frac{H_F}{D}\right)\right]$$

(2) $$L_C := \frac{2 \cdot R}{\pi} \cdot \cos^{-1}\left(\frac{H_F}{D_{MAX}}\right)$$

The impact pressure of the fluid on the underside of a flat rigid roof, P in kPa is calculated by equation (3). H is the fluid height in metres, γ is the fluid density in kN/cubic metre, g is the gravitation constant in m/sec^2, and v is the elevating velocity in m/sec. The pressure on the underside of the roof is maximum at the wall and decreases to zero at a distance L_C from the wall.

(3) $$P := 6.63 \cdot \frac{\gamma}{g} \cdot (H + H_F) \cdot \frac{R}{H_F} \cdot \frac{(0.2 \cdot R + H + H_F)}{(0.4 \cdot R + H + H_F)^2} \cdot v^2 + 0.35 \cdot \frac{\gamma}{g} \cdot R \cdot \frac{(H + H_F)}{(0.4 \cdot R + H + H_F)} \cdot \frac{2 \cdot \pi}{T} \cdot v$$

The pressure on the underside of the roof produces an equal pressure on the wall adjacent to the roof which is a maximum at the roof-wall intersection and decreases to near zero at the bottom of the tank (Amano et. al. 1989).

Kobyashi's method uses a equation (4) to calculate pressure on the underside of a flat roof. The impact pressure is a function only of the elevating velocity. This method produces pressures which are generally less than Kurihara's method.

(4) $$P := 0.221 \cdot \frac{\gamma}{g} \cdot (v \cdot 100)^{1.6}$$