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SEISMIC ANALYSIS OF A PARTIALLY-BURIED DRINKING WATER RESERVOIR

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ABSTRACT

This paper describes the details of the seismic analyses undertaken to retrofit the Kersland drinking water reservoir in Vancouver, British Columbia, Canada. The reservoir has a storage capacity of about 67 million litres, measures approximately 100 m by 150 m, and is partially buried. The walls are supported by above-ground soil berms. Seismic upgrading of the reservoir required an assessment of the loads imposed on the perimeter walls of the reservoir due to the design seismic event. The problem of soil-structure and structure-fluid interaction during seismic loading is complex, and could significantly increase the lateral forces on the reservoir wall. Seismic response analysis of the soil-structure system was carried out using the finite element program FLUSH. Modeling the effects of water under earthquake loading was included as convective and impulsive forces using a series of lumped masses attached to selected structural beam element nodes and a horizontal spring. The bending moments and shear forces in the reservoir wall, and seismic earth pressures exerted by the soil on the wall were obtained for detailed structural analyses. The results of FLUSH analyses were compared with the available closed-form solutions.

INTRODUCTION

The magnitude and distribution of lateral earth pressure exerted on a retaining wall during an earthquake are complex. The closed-form solutions available in the literature are not conclusive, particularly with regards to the magnitude and the distribution of the lateral loads (Richards and Elms, 1979, Matsuzawa et. al., 1985, Stadler et. al., 1995, and Richards et. al. 1999). In the case of a reservoir, the problem becomes even more complicated as there is soil-structure and fluid-structure interaction during seismic loading.

Most of the available closed-from solutions for seismic lateral earth pressures are based on the Mononobe-Okabe (M-O) method. This method considers the inertia forces due to an earthquake on the Coulomb wedge and provides a seismic earth pressure coefficient. However, the method does not provide information on the distribution of earth pressure and the anticipated magnitude of movement/rotation of the wall. Richard and Elms (R-E) extended the M-O method to include effects of wall movement away from the backfill. Different researchers have suggested different distributions varying from triangular to inverted triangular shapes.

Matsuzawa et al. (1985) suggest that the dynamic water pressures should be calculated using the Westergaard's approximate solution. This may provide an estimate of the impulsive hydrodynamic forces caused by the portion of water accelerating with the tank but would not provide the convective force caused by the portion of water sloshing in the tank. Adding the hydrodynamic pressures to the seismic lateral earth pressures computed from the M-O or R-E method may not give realistic loads on the retaining wall, as movements of the structure and water could be out of phase.

BACKGROUND OF THE RESERVOIR

The Kesland Reservoir is one of the 18 drinking water reservoirs in the Greater Vancouver area and is located in the Queen Elizabeth Park. It consists of two separate but structurally connected reinforced concrete tanks providing a total water storage capacity of about 67 million litres. The first reservoir unit was constructed in 1954 and measures approximately 46 m by 100 m. The second unit was added during 1958-59 and measures approximately 100 m by 150 m (see Fig. 1). Both reservoir units have been constructed by partially excavating the upper surficial clay/silt deposits and dense glacial till to about 4 m depth below existing ground surface.

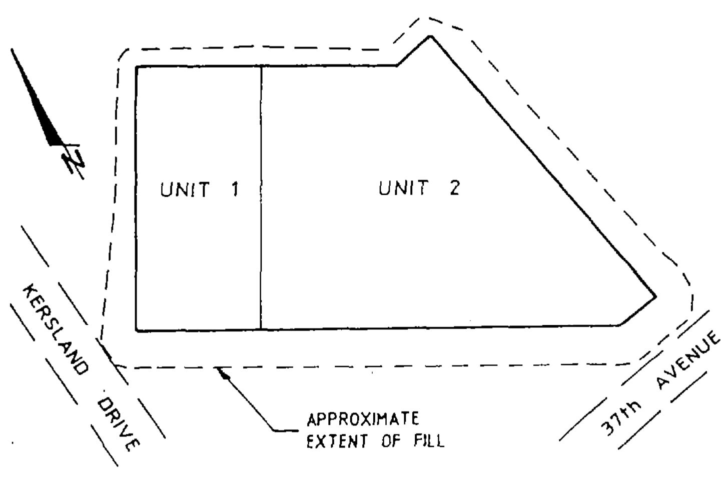

Fig. 1. Site Plan – Kersland Reservoir.

The outer perimeter walls of the reservoir are supported on strip footings located about 2.5 m above the reservoir floor, making the central portion of each unit deeper than the perimeter areas. The reservoir roof is supported on a series of single unbraced vertical columns, which are supported on shallow spread footings within the sloping and flat bases of the reservoir. The reservoir roof is structurally separated from the perimeter wall system.

The natural ground surface at the site generally slopes down from north to south and the reservoir walls in the western and southern areas are supported by earth berms on the outside. The earth berms typical slope at about 3H:1V. Although the available construction reports indicate that backfill has been placed for lateral stability of the outer perimeter walls, no information was available on the nature or level of compaction of backfill. It was inferred that the excavated soil was used as backfill, with little or no compaction effort.

FIELD INVESTIGATION

As the nature and extent of the backfill outside the wall were uncertain, a field investigation comprising mapping using Ground Penetrating Radar (GPR) along 13 lines, 4 auger holes, 6 dynamic cone penetration tests and 6 Becker penetration tests was carried out. The results of the investigation indicated the following:

- The footings of the perimeter walls appear to be founded on the competent till-like soils.
- The wall backfill consists of native material excavated for construction of the reservoir, and has been placed with little or no compaction effort.
- The relative density of the backfill is variable from loose to medium dense with possible very loose pockets.
- The groundwater table is likely to be located about 2 m to 2.5 m below perimeter wall footings.

EARTHQUAKE GROUND MOTION PARAMETERS

The National Building Code of Canada (NBCC-1995) and the British Columbia Building Code (BCBC-1998) defines that the reservoir site is located within Seismic Zone 4 (on a scale of 0 to 6), which is one of the higher seismic loading designations in Canada. The site-specific peak firm-ground accelerations obtained from Pacific Geoscience Centre (PGC) in British Columbia for 100-, 200- and 475-year return period earthquakes were 0.09 g, 0.13 g and 0.21g, respectively. However as per the project brief, and to be consistent with previous studies of two other reservoirs in the Vancouver area, a peak horizontal firm-ground acceleration of 0.19 g was selected in the analysis of the seismic response.

The selection of acceleration time histories that correspond to the design earthquake is a topic of extensive discussion (for example, refer to Somerville, 1998). In the current geotechnical practice, the input time histories are selected by one of the following procedures:

1. Selecting several (typically more than 3) acceleration time histories recorded from past earthquakes that correspond to the fault mechanisms, geological conditions and earthquake magnitudes that are applicable to the site, and uniformly scaling the records so that the peak acceleration of each selected record corresponds to the site required site-specific peak acceleration.
2. Developing a site-specific firm-ground acceleration design response spectrum and obtaining a series of acceleration time histories with their spectra matching the design spectrum closely. This is generally carried out by modifying actual earthquake records in the frequency domain using specialized computer software. However, this approach has the disadvantage of not retaining the phase characteristics of the time history that is being modified.
3. Generating site-specific acceleration time history by numerical simulation using seismological models. This would require detailed geological aspects of the earthquake source, fault mechanism, path, site conditions, basin response effects etc.

In this study the first approach was adopted. Sy et al. (1991) indicate that the ground motions considered in the seismic risk analysis for the Vancouver area using Cornell-McGuire method and the NBCC seismic model, give an normalized acceleration to velocity ratio (i.e. "a/v" ratio) of unity. The dominant contributions in the NBCC model come from earthquakes of Magnitudes 6.3 to 7.3 at epicentral distances of about 30 km to 70 km and a depth of about 20 km.

Sy et al. (1991) have selected a suite of 22 records to meet the above criteria. Among these 22 records, 3 records (Record Numbers 29, 48 and 64) were selected for this study. The reasons for selecting the particular records are as follows:

- The acceleration spectrum of Record No. 29 is similar to the average spectrum computed from the 22 records.
- Records Nos. 48 and 64 have the highest responses in the short and long term period ranges respectively. This would permit an evaluation of the impact of short and long term period motions on the structural response.

The details of the selected records are presented in Table 1.

Table 1. Details of the Selected Earthquake Records

Record No.	29	48	64
Station	900 South Freemont Street	4867 Sunset Boulevard	Titograd Seismic Station
EQ Event	San Fernando 1971	San Fernando 1971	Yugoslavia 1979
Magnitude	6.4	6.4	7.0
Epicentral Distance (km)	42	35	49
PGA (g)	0.11	0.16	0.03
Peak Velocity (m/s)	0.11	0.16	0.04

The acceleration response spectra of the 3 selected records are shown in Fig. 2, together with the mean spectrum computed from the suite of 22 records.

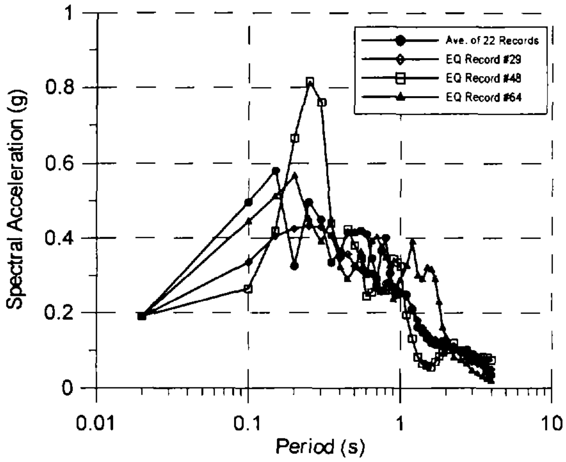

Fig. 2. Acceleration Response Spectra of the Selected Earthquake Time Histories.

ANALYSIS METHOD

Seismic ground response analyses were carried out using the two-dimensional finite element program FLUSH. The analyses considered both soil-structure and fluid-structure interaction effects. The main objective of the analyses was to evaluate the lateral soil pressures, bending moments and shear forces on the perimeter walls and their foundations, which were considered to be critical structural components of the reservoir.

The program FLUSH models the non-linear and hysteric behavior of soil using equivalent linear visco-elastic stress strain relations with strain-dependent moduli and damping. Equivalence is achieved by an iterative procedure such that the moduli and damping values used are compatible with the computed strains.

The structural components of the reservoir, the wall, footings and the concrete liner were modeled using linear bending elements (beam elements) assuming linear-elastic stress-strain properties. The structural components comprised a collection of interconnected beam elements that may be subjected to axial forces, shear forces, and bending moments. The beam elements were attached to the soil elements, where appropriate, to simulate the soil-structure interaction effects. In the analytical procedure followed in FLUSH, compatibility between the soil and beam elements are satisfied at all points along the soil-structure interface.

The hydrodynamic effects (impulsive and convective sloshing forces) resulting from the water contained in the reservoir were modeled using a series of equivalent lumped masses and a spring attached to selected nodes of the beam elements in the FLUSH model. The magnitude and distribution of these lumped masses and the spring constant were provided by the structural consultant and were based on the guidelines provided in the Design Code TID-7024 "Nuclear Reactors and Earthquakes". The computed model parameters are dependent on the geometry of the tank and the characteristics of the fluid contained.

The response of the interior columns and the roof of the reservoir were uncoupled from the response of the outer walls and their footings since the roof is not structurally connected to the perimeter walls. Therefore, the roof and interior columns were not included in the FLUSH model.

From a seismic loading point of view, the southwest and northeast walls, where sloping ground conditions exist, were considered to be more critical. Typical cross sections of both of these walls were analyzed. However, only the details of the analyses of the southwest wall are presented in this paper. The following cases were considered for the southwest wall:

- Case 1. Reservoir with the design high water level.
- Case 2. Reservoir with the design low water level.

The FLUSH model developed for the southwest wall is shown in Fig. 3.

The lumped masses and spring attached to the nodes of the beam elements to model hydrodynamic effects of the water contained in the reservoir are shown in Fig. 4. These lumped masses were considered only for Case 1 since for the low water scenario, the actual water level in the reservoir is slightly below the base of the footing of the perimeter wall.

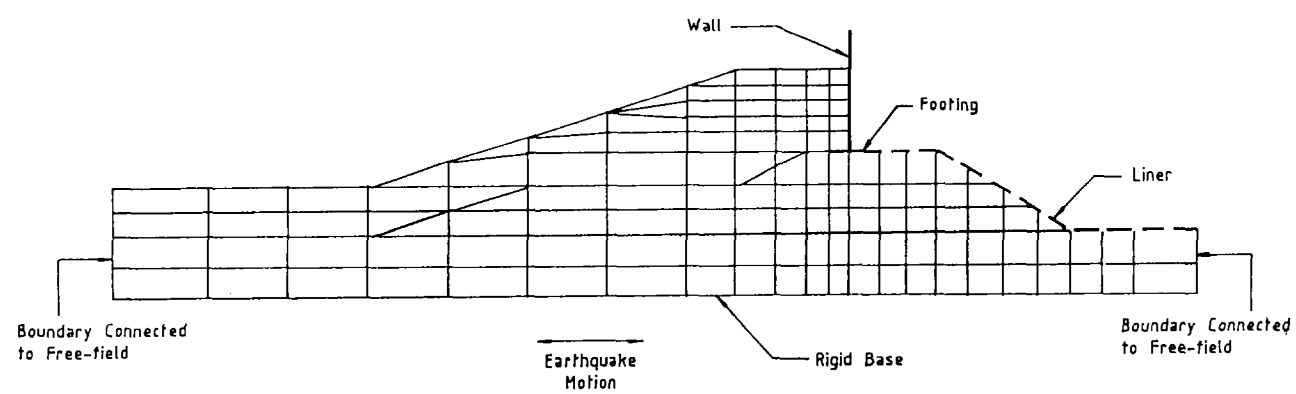

Fig. 3. FLUSH Model – Southwest Wall.

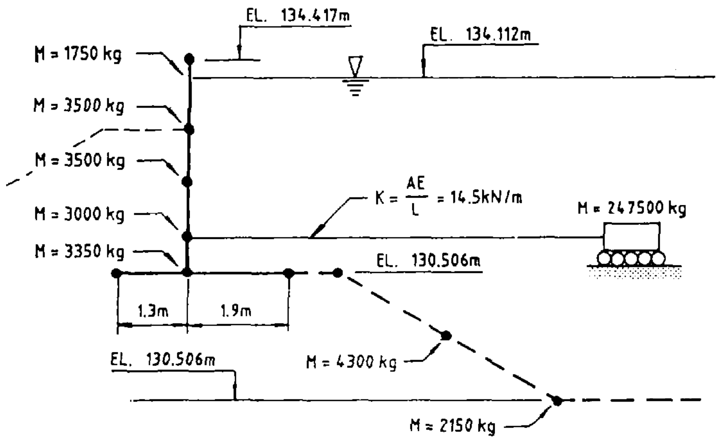

Fig. 4. Modeling of Hydrodynamic Effects of the Water in the Reservoir.

Soil Parameters

The key soil parameters required for the FLUSH analysis are:

- Small strain shear moduli, G_{max}, for the soil zones;
- Variation in shear modulus ratio, G/G_{max} as a function of shear strain for each soil type; and
- Variation in the fraction critical damping as a function of shear strain for each soil type.

The G_{max} values were estimated using the correlation proposed by Seed et al. (1986), which is given below:

$$G_{max} = 440(N_1)_{60}^{1/3} P_a \left(\sigma'_m / P_a\right)^{1/2} \quad (1)$$

where $(N_1)_{60}$ is the Standard Penetration Test (SPT) blowcount normalized to a confining stress of 96 kPa (1 t/ft^2) and corrected to an energy level of 60% theoretical free fall energy of the hammer, σ'_m is the mean normal effective stress and P_a is the atmospheric pressure in the desired units (i.e. P_a = 101 kPa). The above correlation has been developed for cohesionless soils. Although the silt content in the soils is somewhat higher (30 to 40% passing US #200 sieve), the material behavior is expected to be essentially similar to that of a cohesionless soil and therefore the above correlation was considered to be applicable.

The $(N_1)_{60}$ values obtained from the dynamic cone penetration tests and the Becker Penetration test results were used in Equation (1) to obtain the G_{max} values. An average $(N_1)_{60}$ of 12 blows/0.3m was considered representative for the loose to medium dense backfill, whereas 75 blows/0.3 m was considered representative for the very dense glacial till like soil.

The variations in shear modulus ratio and the fraction of critical damping adopted for the materials were similar to those adopted by Idriss (1990) and are shown in Fig. 5.

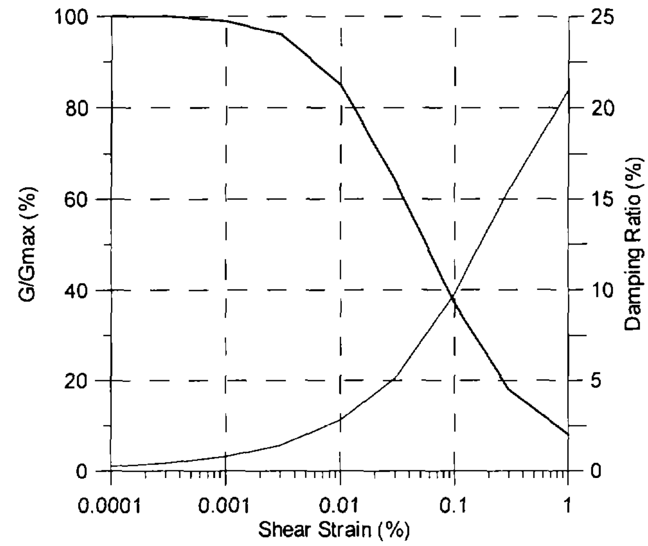

Fig. 5. Modulus Reduction and Damping Variation with Shear Strain.

Structural Parameters

The main structural parameters required for the FLUSH model are the cross sectional area, A, moment of inertia, I, and the elastic modulus for the structural elements, which are presented below:

	A (m^2/m)	I (m^4/m)
Wall	0.381	5.16×10^{-3}
Footing	0.508	1.09×10^{-2}

The Young's modulus of concrete was assumed to be 27000 MPa.

RESULTS OF THE ANALYSES

The results presented herein are due to the applied seismic motions only. The effects of static loading should be added to these results to assess the combined effect of both static and seismic loads.

It should be also noted that only the maximum values of accelerations, lateral stresses, bending moments and shear forces are presented herein. The range of values presented for a selected variable corresponds to the results from different earthquake records.

The peak horizontal accelerations computed along the reservoir perimeter wall are shown in Fig. 6.

The predicted maximum lateral earth pressures on the outside of the perimeter wall are shown in Fig. 7. It should be noted that the height of the earth berm outside the Southwest wall is about 2.7 m above the base of the wall.

The predicted maximum bending moments and shear forces in the reservoir wall are shown in Figs. 8 and 9.

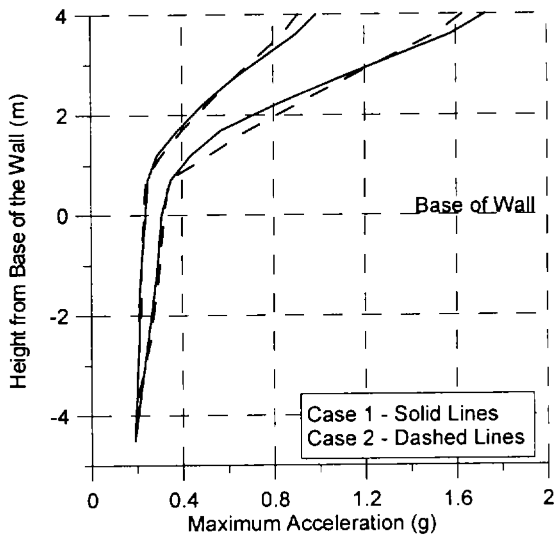

Fig. 6. Variation of Maximum Accelerations Along the Reservoir Wall.

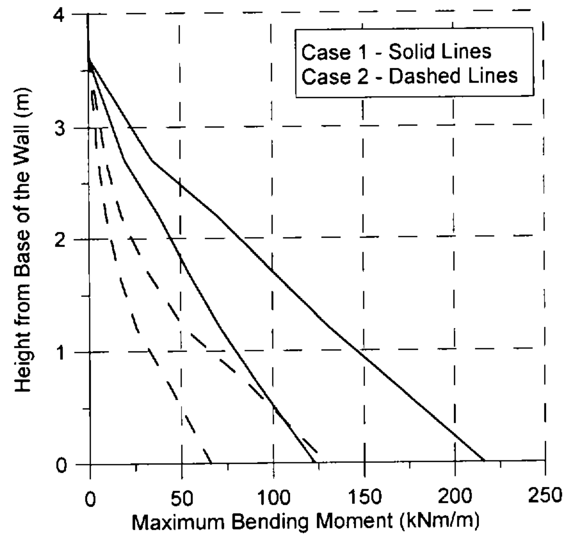

Fig. 8. Variation of Maximum Bending Moment in the Reservoir Wall.

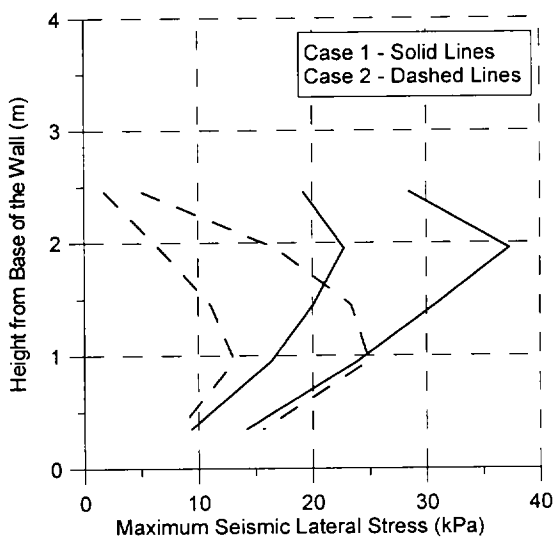

Fig. 7. Variation of Maximum Seismic Lateral Earth Pressure in the Reservoir Wall.

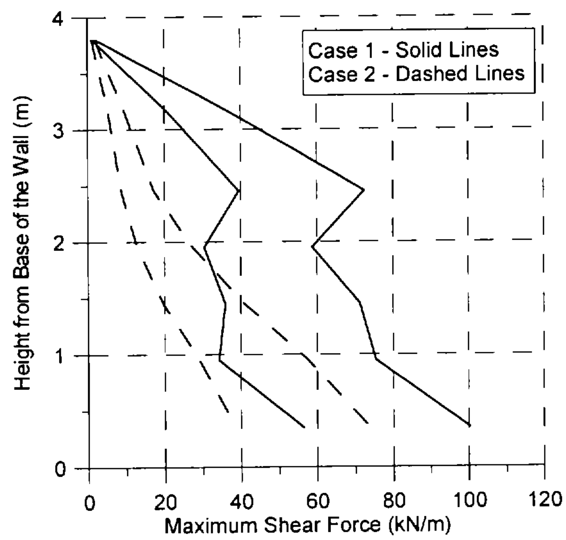

Fig. 9. Variation of the Maximum Shear Force in the Reservoir Wall.

DISCUSSION

The results of the analyses indicate that the largest bending moments and shear forces occur at the base of the wall and for the high design water level in the reservoir. For the case of high design water level (Case 1), the following results were predicted from the analyses:

- The resultant force due to seismic lateral earth pressures varied from 45 to 70 kN/meter length of the wall.
- The point of application of the resultant force was at a height of about 0.57H, where H is the height of the embankment above the base of the wall.
- The maximum bending moment at the base of the retaining wall predicted by FLUSH (Fig. 8) varied from 125 to 215 kNm/metre length of wall. However, the bending moment due only to the seismic lateral earth pressure acting on the wall (calculated using the resultant force and point of action) was in the range of 70 to 110 kNm/m. The remainder of the bending moment was due to the inertia forces acting on the 380 mm thick wall and the hydrodynamic effects.

As Case 1 includes both soil-structure and fluid-structure interaction effects, comparison of the results for this case was not possible with the available closed-form solutions. However, it was possible to compare the results for Case 2, which does not include the hydrodynamic effects, with the results of closed-form solutions. Recognizing that the wall may move several millimeters during seismic loading, the results of the finite element analyses were compared with those of M-O and R-E methods. For Case 2, the following results were predicted from the FLUSH analyses:

- The resultant force due to seismic lateral earth pressure varied from 20 to 45 kN/metre length of the wall.
- The point of action of the resultant force varied from about 0.42H to 0.44H.
- The bending moment at the base of the retaining wall varied from 65 to 130 kNm/metre length of the wall. The bending moment due to the seismic lateral earth pressure acting on the wall only (calculated using the resultant force and point of action) was in the range of 30 to 60 kNm/m. The remainder of the bending moment was due to the inertia forces acting on the 380 mm thick wall.

The ground response analyses indicated that the soil backfill in the vicinity of the wall would experience an average horizontal acceleration of about 0.5 g. Using the R-E method, for an outward wall movement of about 5-mm and an average acceleration of 0.5 g in the Coulomb wedge, a seismic lateral earth pressure (excluding static loading effects) coefficient of 0.6 was computed. This resulted in a seismic loading-induced lateral force of about 45 kN/m, which is in agreement with the upper-bound force derived from the lateral earth pressures computed in the FLUSH analysis. The corresponding dynamic lateral earth pressure coefficient computed from Wood's (1973) solution is close to 0.9, which is considerably higher than the coefficient estimated from the results of FLUSH analyses.

If results of ground response analyses were not available, based on empirical data, the horizontal ground acceleration would have been estimated to be close to be about 0.27 g (allowing for a modest amplification of 30%). For a non-yielding wall, or a wall that is fixed against movement, with a 50% increase in the acceleration coefficient as recommended in ATC-6 for bridge abutments, this would result in a seismic earth pressure coefficient (excluding static effects) of about 0.44. The resulting lateral seismic pressure is close to 30 kN/m, which is closer to the lower-bound force derived from the earth pressures computed from the FLUSH analyses.

The distribution of the seismic lateral earth pressure and the point of application of the resultant lateral thrust predicted by the results of FLUSH analyses agreed well with those shown in Richards et al. (1999) for walls "fixed" against movement.

SUMMARY AND CONCLUSION

The seismic response of a reservoir retaining wall, where the soil-structure and fluid-structure interaction could occur, require more detailed analysis, if the magnitude and distribution of lateral pressures on the walls are to be computed accurately.

For the case of a reservoir retaining wall, the hydrodynamic effects of the fluid contained within the reservoir significantly increases the lateral seismic loads on the retaining wall.

The results of the finite element analyses carried out using FLUSH, where no hydrodynamic effects were considered, generally agree with the range predicted by the available closed-form solutions. When the effects of inertia forces on the wall stem are included, the bending moments at the base of the wall can be significantly larger (by a factor of 2) than the bending moments computed using only the lateral soil pressures acting on the wall.

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