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PERFORMANCE-BASED SEISMIC EVALUATION OF CONCRETE RESERVOIR STRUCTURES

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SUMMARY

To mitigate a risk of water supply disruption in a post-earthquake situation in Vancouver, a major urban center located in the seismically most active region of Canada, the Greater Vancouver Water District (GVWD) has initiated seismic assessment and subsequent retrofitting of critical water reservoirs in the Greater Vancouver area. The reservoirs are typically twenty to eighty years old and are generally of similar construction consisting of a reinforced concrete flat slab roof structure supported by the columns independent of perimeter concrete cantilever walls. At the time of the original construction, flat slab structures were designed to sustain gravity load effects only and, therefore, lateral load resisting capacity of these structures is found to be rather limited. The paper discusses seismic evaluation of a concrete reservoir, in particular the flat slab roof structure, using the methodology prescribed in the FEMA 356 document *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*. Nonlinear static ("pushover") analysis has been performed using the SAP2000 software. Various performance levels, ranging from the Collapse Prevention to Immediate Occupancy, have been considered in the evaluation. The performance acceptance criteria include deformation demands for flexure in beam and column elements, and shear force demand vs. capacity for punching shear in the flat slab. The results include the estimated lateral drift values at various structural performance levels and the corresponding predicted damage scenarios for key structural elements.

INTRODUCTION

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 most seismically active region of Canada. To mitigate a risk of water supply disruption in a post-earthquake situation, the Greater Vancouver Water District (GVWD) initiated in 1992 seismic assessment and subsequent retrofitting of critical water storage facilities (reservoirs) in the Greater Vancouver area. GVWD supplies water to approximately two million people, about half the population of British Columbia. The water distribution network includes 22 balancing reservoirs, critical for providing network storage and capacity during peak day demand. Four

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reservoirs upgraded to date were originally constructed between 1928 and 1980. Basic information related to the four GVWD-owned reservoirs upgraded to 2002 is summarized in Table 1 (three additional reservoirs have either been upgraded, reconstructed, or are in the process of being upgraded since 2002).

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	1974/75	36 ML	1998
Cape Horn	1980	44 ML	2002

Several retrofitting schemes have been used for upgrading the deficient reservoir roof 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. The first two schemes represent conventional seismic upgrade solutions, and were used successfully for upgrading the Vancouver Heights and the Kersland reservoirs, as discussed by Sherstobitoff [1]. The third, less conventional option, entails the installation of seismic dampers to achieve a substantial increase in modal damping ratio from the original level of 2 to 5% to over 20% and thereby reduce the lateral drift response and the overall seismic demand; this scheme was used for the retrofit of the Central Park and Cape Horn reservoirs, as discussed by Nikolic-Brzev [2]. Comprehensive seismic studies were performed to evaluate the seismic safety of each reservoir and to identify the most appropriate retrofit scheme once a retrofit was deemed necessary. This paper focuses on the seismic evaluation of the Cape Horn reservoir roof structure using nonlinear static analysis and demonstrates the use of performance-based criteria in predicting structural damage at different design earthquake levels. A detailed description of the analysis is provided in Sandwell [3]. The seismic evaluation described in this paper has been performed using the nonlinear static analysis according to the FEMA 356 document *Prestandard and Commentary for the Seismic Rehabilitation of Buildings* [4].

DESCRIPTION OF THE EXISTING STRUCTURE

The Cape Horn Water Reservoir is located in Coquitlam, British Columbia. Construction of this 44 million litre reservoir took place in 1980. The reservoir consists of a basin excavated into existing soil, lined with concrete with a flat central area and sloping sides, and cantilever concrete walls at the top of the slopes. The reservoir roof structure consists of four separate units, each separated from one another by 50 mm wide expansion joints. The two northern roof panels are approximately 50.3 m by 35.6 m in plan. The two southern roof panels are approximately 50.3 m by 43.0 m in plan. The roof is a two-way concrete flat slab 230 mm thick and is supported by 540 mm square columns on a typical grid spacing of 7.32 m in each direction. The majority of the columns (108 in number) are 6.25 m high. The shorter columns (46 in number) on the sloped portion of the slab on grade are 4.80 m high. The flat slab is thickened in the region over the columns with 2.45 m square by 100 mm thick drop panels and 610 mm high tapered column capitals. The slab is structurally independent of the walls and is supported vertically on the top of the perimeter walls by a continuous neoprene rubber pad. The plan view of the reservoir roof and a typical vertical elevation are shown in Figure 1.

The perimeter walls are generally 2.45 m high on the north, east, and west sides and 3.70 m high on the south side. The vertical cantilever walls supported by strip footings were constructed in 12.2 m sections, with waterstops and sealant at the vertical contraction joints.

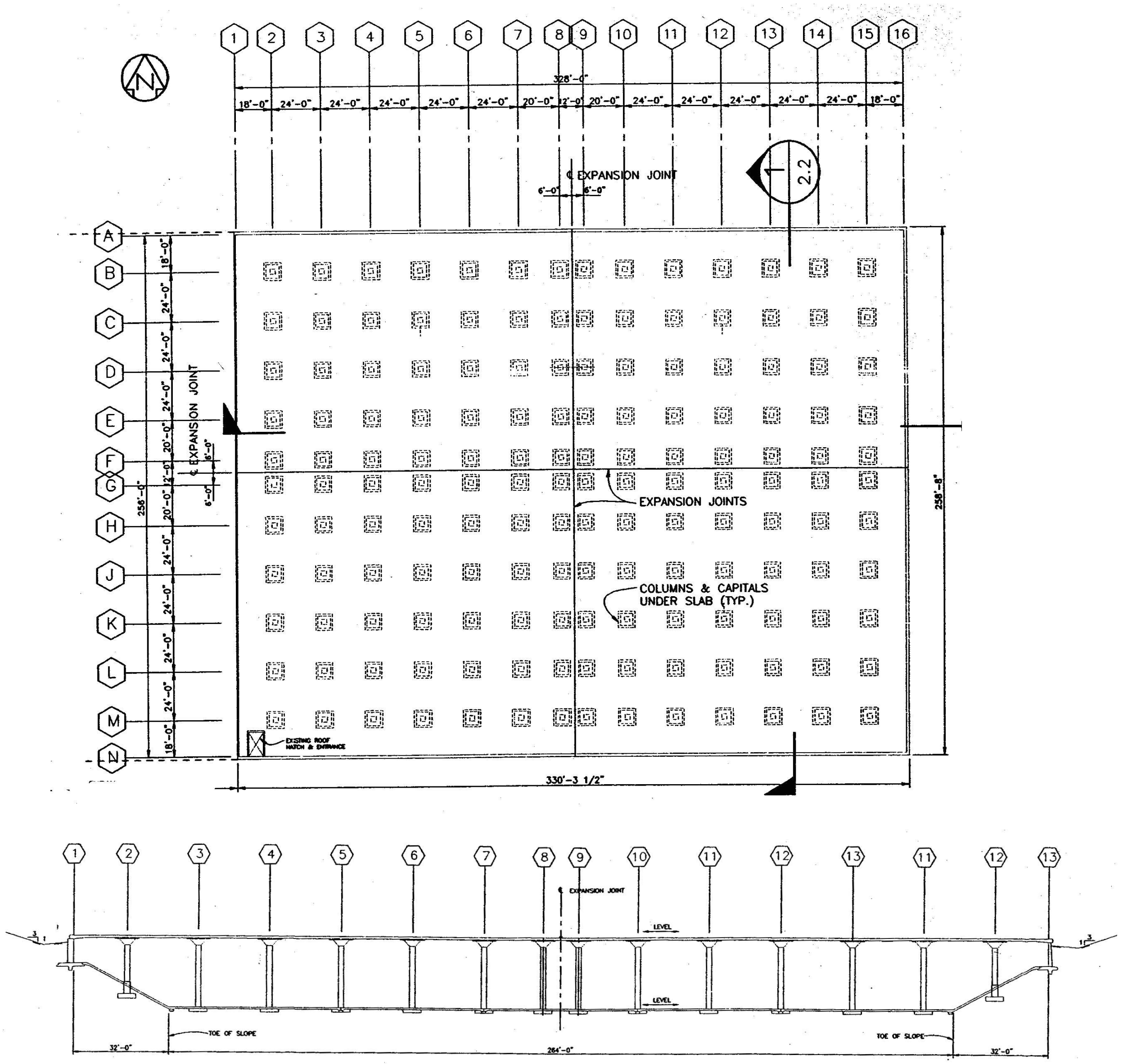

Figure 1: The reservoir roof plan and a vertical elevation .

SEISMIC CRITERIA

According to the 1995 National Building Code of Canada (NBCC) [5], the reservoir is located in Seismic Zone 4 of Canada. GVWD previously conducted a deterministic seismic risk study for a location close to the Cape Horn reservoir site to estimate Peak Ground Acceleration (PGA) levels corresponding to design earthquakes. The design earthquake levels and the corresponding seismic performance criteria for these projects are summarized in Table 2.

Table 2: Design Earthquake Levels and the Seismic Performance Criteria

Earthq. level (Return period)	PGA	Seismic performance criteria
SLE (100 years)	0.07*g*	Reservoir exhibits elastic response with no damage.
OBE (475 years)	0.20*g*	Reservoir remains operational but may experience cracking and moderate leakage that may be repaired, when convenient, within a year following the event.
MCE	0.50*g*	Reservoir may experience extensive damage, however, no sudden, catastrophic release of water occurs from the containment structure.

SLE = Service Level Earthquake
OBE = Operating Basis Earthquake
MCE = Maximum Credible Earthquake - an M 6.5 event occurring at a distance of approximately 10 km from the site, with an estimated firm ground PGA level of 0.5 g.

As a result of the seismic risk study, a set of response acceleration spectrum curves corresponding to the mean confidence level was developed. A response spectrum curve corresponding to the OBE level earthquake is shown in Figure 2.

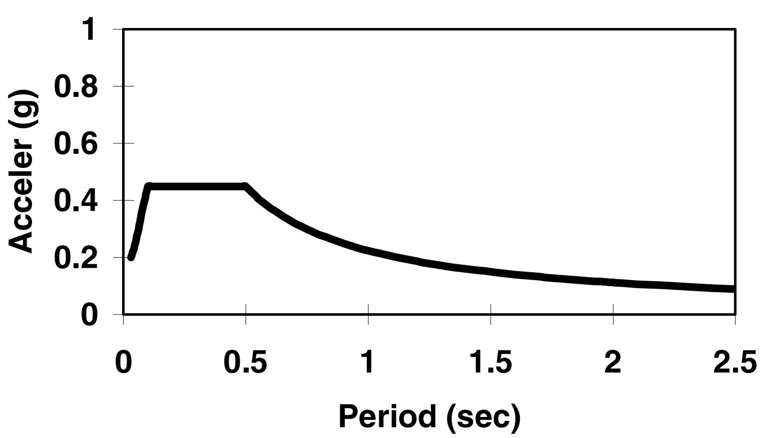

Figure 2: Response spectrum curve corresponding to the OBE level (PGA=0.2 g), mean confidence level.

SEISMIC FEATURES

The Cape Horn reservoir was constructed in 1980 and it is expected that the design was carried out according to the 1975 National Building Code of Canada (NBCC). The roof structure consists of a flat slab supported by columns, column capitals, and drop panels. The columns have rebar ties at a vertical spacing of 200 mm (8 inches) for the top 1.8 m (6 feet) and 300 mm (12 inches) for the remainder of the column. As the tie spacing does not decrease near the base of the columns like it does at the top, it has been understood that the original design assumed that the small spread footings at the base of the columns would not provide any fixity in rotation (i.e. column bases are pinned). Seismic design requirements of the Canadian Concrete Code [6] prescribe that the minimum bottom steel in flat slabs must be continuous over the columns to ensure the ductile structural performance in an earthquake. In the Cape Horn reservoir roof slab there is some overlap of the bottom reinforcing steel over the column according to the original construction drawings, however it is inadequate per current code requirements for ductile performance. Details of roof slab and column reinforcement are shown in Figure 3.

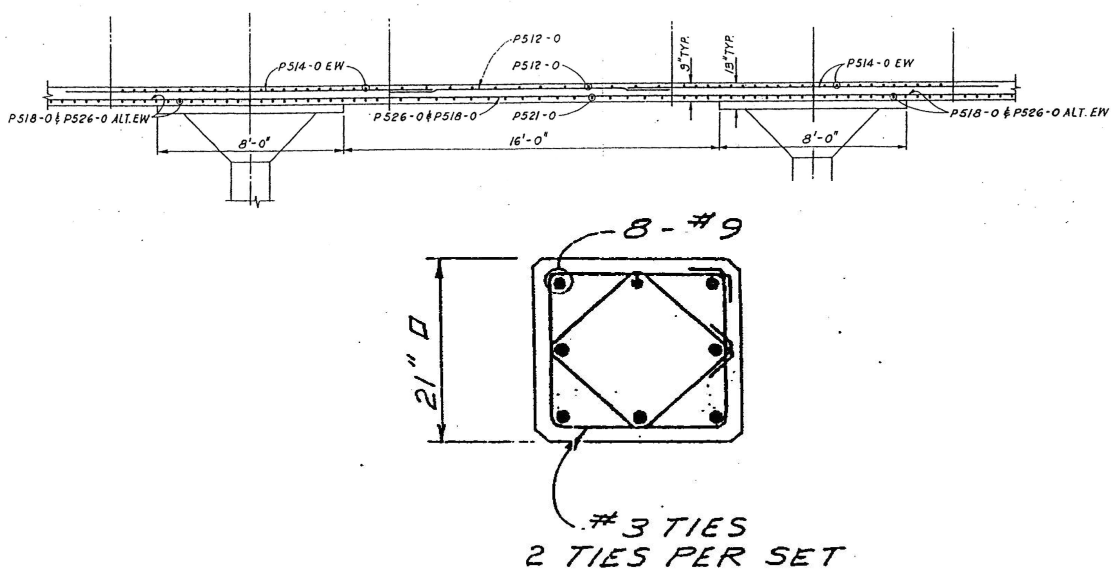

Figure 3: Details of the slab and column reinforcement.

SEISMIC EVALUATION OF THE ROOF STRUCTURE

Initially, equivalent static analysis and response spectrum dynamic analyses according to the 1995 NBCC were performed to evaluate lateral deformability and load-bearing capacity of the existing reservoir roof structure. As these analyses revealed that the structure is borderline to meet the GVWD seismic performance criteria for the OBE level earthquake (see Table 2), nonlinear static (pushover) analysis according to the FEMA 356 [4] was subsequently carried out to verify the results of the linear analysis and confirm the adequacy of the existing structure.

LINEAR STATIC ANALYSIS

The equivalent static method according to 1995 NBCC [5] was used to perform the seismic analysis of the existing structure. The following parameters were considered in the analysis: **v** = 0.2 (velocity factor), **F** = 1.0 (foundation factor, corresponding to firm soil conditions), and **I** = 1.0 (importance factor). According to the CSA [6], value of **R** = 1.5 has been recommended for two-way slab systems without beams. **R** denotes the force modification factor, which reflects the expected ductility capability of a structure; for example, **R** = 1 relates to elastic response. Based on the original construction drawings, detailing of columns and in particular tie spacing, complies with the requirements for moment frames with nominal ductility (**R** = 2) according to (CSA 1994).

Several 2-D structural models were developed to evaluate lateral deformation capacity of the existing roof structure using the equivalent frame analysis as per the CSA [ ]. Typical interior and perimeter frames were identified for the analysis. The main difference between these frames is the column length and the base support conditions. The "interior" columns are 6.25 m long whereas the "perimeter" columns are shorter (4.8 m long) and fixed at the base. A number of parameters affecting the seismic response of the roof structure were varied in the analysis, including column base support conditions (pinned/fixed/variable soil spring stiffness) and moduli of inertia values for the slab and columns (cracked/uncracked). For the cracked structure, values of gross modulus of inertia for the columns and the slab were reduced by 30% and 60% respectively, as recommended by the CSA [6]. Combinations of various structural parameters mentioned above resulted in nine different seismic load cases. Out of these, the three most relevant cases corresponding to the OBE (I=1.0) earthquake level are described in Table 3.

Table 3: Summary of Seismic Load Cases (OBE, I=1.0)

Seismic Load Case	Period (sec)	Seismic Base Shear Force (V)
Case 1: 100% NBCC code value (R=1.5)	0.28	0.23W
Case 2: Cracked concrete slab and columns at hinge locations only, all column bases supported by stiffest soil springs (R=1.5)	0.51	0.17W
Case 3: Same as Case 2, except R=2.0	0.51	0.13W

The demand/capacity (D/C) ratios were determined for critical load-bearing structural elements: roof slab and columns, as shown in Table 4. The analysis has revealed a general inadequacy of these elements to sustain the effects of a design level earthquake. The analysis has shown that the negative flexural capacity of the slab at the transition from 330 mm (13 inches) thick drop panel to 230 mm (9 inches) thick roof slab represents a "weak link" in the system. It should be noted, however, that the results for Case 3 analysis performed using the **R** value of 2.0 show that the existing structure is "almost adequate" for the OBE (I=1.0) earthquake. A further analysis was deemed appropriate, and therefore a nonlinear static analysis was performed to obtain a more precise prediction of the expected seismic performance for the existing reservoir roof.

Table 4: Summary of D/C Ratios at Critical Locations for Various Cases

Structural Component	Case 1	Case 2	Case 3
Exterior column (bending)	1.7	1.2	0.9
Interior column (bending)	1.3	1.0	0.7
Perimeter slab – negative bending at the outside edge of the drop panel	1.9	1.4	1.1
Interior slab – negative bending at the outside edge of the drop panel	1.2	0.9	0.7

The elastic analysis determined that the acceptable lateral displacement of the reservoir roof should be below 35 mm to ensure the D/C ratios for all elements to be less than 1.

NONLINEAR STATIC (PUSHOVER) ANALYSIS

Background

Nonlinear static analysis, commonly known as "pushover" analysis, is a technique by which a computer model of the structure is subjected to lateral load of a predefined pattern. The intensity of the lateral load is slowly increased and the sequence of cracks, yielding, plastic hinge formations, and failure of various structural components as a function of increasing lateral load is recorded. The analysis continues until either a predefined target lateral displacement is exceeded or the structure collapses. Results of a preliminary pushover analysis for three of the GVWD reservoirs outlined in Table 1 of this paper are discussed by Nikolic-Brzev [7].

Pushover analysis of the reservoir roof structure was used to determine lateral deformation capacity (ductility) of the flat slab structure at different seismic performance levels and the corresponding force modification factor (R). The current Canadian Concrete Code [6] recommends an R value of 1.5 to be used for two-way slab systems without beams (per Cl. 21.9.1.). However, FEMA 356 [4] recommends that the force modification factor (m) value of 2 or higher be used for flat slab structures subjected to low gravity loads, which was the case of the Cape Horn reservoir. For a simple one-storey structure of the reservoir roof under consideration, the terms R and m are considered to be essentially equivalent (note that this may not be true for more complex or multi-storey structures).

Seismic Performance Levels

The basic criterion followed in the seismic performance evaluation of the reservoir roof structure is Life Safety (LS) performance, which corresponds to the code-prescribed performance level for "ordinary" structures subjected to a code-level design earthquake (OBE, I=1). Besides the LS performance level, two other seismic performance levels, namely Immediate Occupancy (IO) (corresponding to "post-disaster" facilities as per the 1995 NBCC) and the Collapse Prevention (CP) level were also considered in this analysis. It should be noted that the correlation between the FEMA 356 and the 1995 NBCC performance levels has been derived based on the comparison of lateral drift limits prescribed in those two documents. The 1995 NBCC prescribes the 2% and 1% drift limits for ordinary and post-disaster structures respectively. Based on this recommendation and a similar FEMA 356 recommendation related to concrete frame structures (Table C1-3 of FEMA 356), it can be concluded that the FEMA 356 Life Safety performance level corresponds to the NBCC category of "ordinary structures" (I=1.0), whereas the Immediate Occupancy (IO) level as per FEMA 356 corresponds to the NBCC category of "post-disaster" structures (I=1.5). A summary of seismic performance levels as per FEMA 356 and 1995 NBCC is provided in Table 5.

Table 5: Seismic Performance Levels (based on Tables C1-2 and C1-3, FEMA 356)

	Structural Performance Levels		
FEMA 356	Immediate Occupancy (IO)	Life Safety (LS)	Collapse Prevention (CP)
Corresponding 1995 NBCC performance level	OBE, I=1.5 Post-disaster structures	OBE, I=1 Ordinary structures	Not addressed.
Overall Damage	Light	Moderate	Severe
Lateral Drift Limit	1% transient; negligible permanent	2% transient; 1% permanent	4% transient or permanent

The 1995 NBCC does not offer an explicit description of damage conditions corresponding to various seismic performance levels (e.g. ordinary structures or post-disaster structures). However, FEMA 356 provides a description of damage conditions for various structural systems at IO, LS, and CP performance levels (Tables C1-3 and C1-4, FEMA 356). Description of the expected damage condition of the reservoir roof at various FEMA 356 seismic performance levels has been summarized in Table 6. For the purposes of this analysis and evaluation, the LS performance is considered the upper bound of acceptable damage for the roof structure per the GVWD performance criteria at the OBE earthquake (described in Table 2), whereas the CP performance is seen to be similar to the GVWD performance criteria at the MCE earthquake.

Table 6: Expected Damage Condition for the Reservoir Roof Structure at various FEMA 356 Seismic Performance Levels

	Structural Performance Levels		
FEMA 356	**Immediate Occupancy (IO)**	**Life Safety (LS)**	**Collapse Prevention (CP)**
Comparable GVWD criteria	Approximately SLE, no damage	OBE, reservoir remains operational, moderate leakage	MCE, extensive damage, no catastrophic release of water
Correspondin g 1995 NBCC performance level	OBE (I=1.5) Post-disaster structures	OBE (I=1) Ordinary structures	Not addressed.
Slabs	Some visible cracking near the slab-column connections (at drop panel locations)	More extensive flexural cracking near the slab-column connections (at the drop panels); some minor spalling at the bottom of the slabs in a few locations.	Extensive flexural damage in the slabs; spalling expected at the bottom of the slab in many locations; Collapse not expected;
Columns	Minor cracking in the columns, probably not visible to the unaided eye	Flexural cracking visible near the tops of some of the columns (plastic hinge regions below the capital).	Extensive flexural cracking expected at the tops of the columns (plastic hinge regions); spalling of concrete cover expected in some columns. Chances of a brittle shear failure in some columns due to inadequate ties (plastic hinges); would result in local drop-down of roof slab – extent undefined. Except for the local shear failures in a few columns, no global collapse expected;
Repair	Minor	Feasible at a reasonable cost	Repair might be possible depending on amount of permanent drift; however replacement might be economically more feasible.

Target Displacement

According to FEMA 356 [4], "target" displacement represents the maximum spectral lateral displacement a structure is expected to experience when subjected to the design earthquake. The GVRD site-specific response spectrum curve (OBE, I=1, mean confidence level) shown in Figure 2 was used for the calculation of the target displacement. To account for the effects of accidental torsion expected in the perimeter portions of the reservoir roof structure, the target displacement value obtained using the FEMA 356 procedure has been increased by 25%; this increase is based on the results of the 3-D response spectrum analysis, as discussed by Sandwell [3]. To evaluate the performance of the roof structure at earthquake levels higher than the OBE (I=1.0), the structural performance has also been evaluated at the "1.5 x target displacement" level. It should be noted that this increased target displacement level actually corresponds to a design earthquake with an intensity increased by 50% (corresponding to the OBE, I=1.5 design earthquake).

It should be noted that target displacement values corresponding to a typical interior frame were used in this evaluation, as the lower stiffness and larger deformation of the interior frames govern the overall performance. Target displacement values range from 76 mm (corresponding to 100% target displacement without torsional effects) to 143 mm (corresponding to 150% target displacement with the torsional effects considered).

Performance Acceptance Criteria

Once the target displacement has been determined, the accumulated forces and deformations at this displacement have been used to evaluate the performance of structural elements (slabs and columns) as follows:

1. For flexure in slab and column elements (i.e. "deformation-controlled" actions), the deformation demands have been compared with the maximum permissible plastic hinge rotation values for IO, LS, and CP performance levels prescribed by FEMA 356 [4], see Table 7. These values are specific for the detailing and reinforcing of the Cape Horn reservoir.
2. For "force-controlled" actions such as punching shear in the slab, the shear stress capacity ($\mathbf{v_c}$) is compared with the force demand ($\mathbf{v_f}$) at the target displacement level.

The Analysis Model – Key Features

The pushover analysis of two frames in the E-W direction was carried out using the SAP2000 software (NL-PUSH module), see CSI [8]. Out of the two frames considered, one is a typical interior frame, whereas the other one is a "perimeter" frame. The "effective beam width" model has been used for the pushover analysis, wherein columns and slabs are represented by frame elements that are rigidly interconnected at the slab-column joints. Pinned column base support conditions have been assumed both for interior and perimeter columns. Note that the expansion joint at the roof midspan has been deleted and modeled as a pinned connection in the analysis.

Gravity loads were applied as initial conditions in the pushover analysis and maintained throughout the analysis, with the following load combination: 1xDL + 0.25xLL, where DL denotes the dead load and LL denotes the snow load (25% design snow per NBCC). The load pattern used in the analysis was in the form of a uniform horizontal acceleration, wherein the force in each joint is proportional to the mass tributary to that joint.

The following effective stiffness properties for slab and column elements have been used in the analysis:

1. Columns – stiffness: $0.7E_cI_g$ (cracked stiffness, full height), where I_g denotes the moment of inertia for the column gross section and E_c is the modulus of elasticity of concrete. It should be noted that the column strength has been determined based on actual cross sectional dimensions (per CSA [6], Cl. N21.2.2.1 and also FEMA356 [4], Table 6-5).
2. Slab - stiffness: $0.165\ E_cI_g$, where I_g denotes the moment of inertia for entire transverse slab of width equal to the span L. The stiffness determined in this manner corresponds to the fully cracked slab width. The value of 0.165 has been determined as a product of the following two factors: effective slab width factor (α) equal to 0.5 (a generally acceptable value), which is multiplied by a value of 0.33 (so-called β factor) to account for cracking effects (Dovich [9], Wight [10]).
3. Slab - strength: $0.5E_cI_g$; the value of $0.5E_cI_g$ corresponds to the effective uncracked slab width of 0.5L. Note that only the reinforcement provided within the effective width contributes to the flexural

capacity of a slab section. This is a conservative estimate, as it is expected that at the higher seismic loads the slab reinforcement would yield over the full transverse width L.

The following concrete and steel material properties have been used in the analysis:

1. Steel yield strength (f_y) of 400 MPa and concrete compressive strength (f_c') of 40 MPa
2. Yield moment (M_y) has been determined using ϕ_c=1 and ϕ_s=1, where ϕ_c is resistance factor for concrete and ϕ_s is the resistance factor for steel reinforcement
3. Probable moment resistance (M_u) has been determined assuming ϕ_c=1 and ϕ_s=1.25

Plastic hinge properties for slab and column elements in concrete flat slab structures have been adopted as recommended by FEMA 356. Plastic hinge properties represent nonlinear load-deformation relations that define the behaviour of concrete sections under monotonically increasing lateral deformation. According to the FEMA 356 [4], the nonlinear relations have been approximated by line segments. Plastic hinge properties for slab and column sections have been summarized in the FEMA 356 Tables 6-14 and 6-8, respectively. Typical plastic hinge properties for slab and column elements are summarized in Table 7 and Figure 4.

Table 7: A Summary of Modeling Parameters and the Acceptance Criteria – Slabs and Columns [4]

	Modeling Parameters			Acceptance Criteria – Plastic Rotations		
	a (rad)	b (rad)	c	IO	LS	CP
Slabs	0.02	0.05	$0.2M_y$	0.01	0.015	0.02
Columns	0.02	0.03	$0.2M_y$	0.005	0.015	0.02

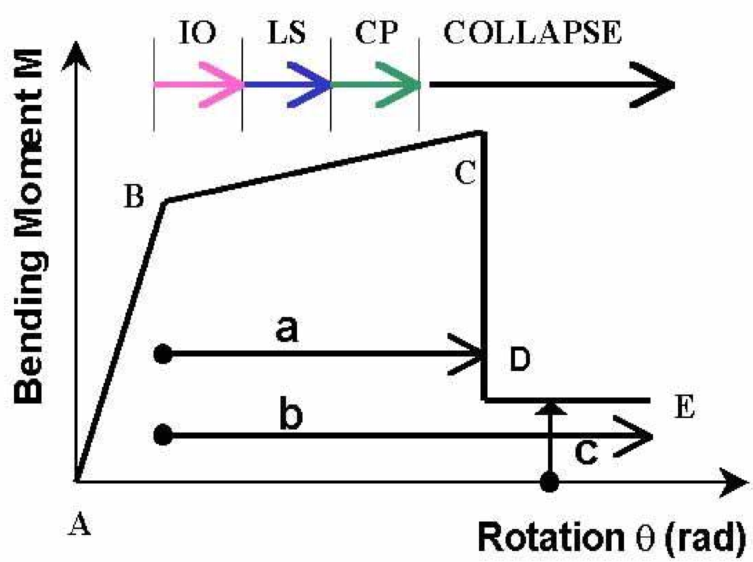

Figure 4: Plastic hinge model per FEMA 356 (IO= immediate occupancy, LS= life safety, CP= collapse prevention).

Punching Shear Capacity

Most reinforced concrete flat slab failures in the past earthquakes can be attributed to excessive vertical shear stresses due to gravity loads and unbalanced moments induced by seismic effects. The results of experimental studies (see Moehle [11]) revealed that gravity load level in flat slab structures is one of the key parameters determining the lateral behavior of column-slab connections and that the ductility capacity of flat slab structures is significantly higher for the structures subjected to low gravity loads. The FEMA 356 document recognizes the importance of gravity load level in flat slab structures: it allows for higher ductility capacity for flat slab structures with gravity/shear strength ratio of less than 0.2 by assigning the **m** value of 2.0 to 3.0, and prescribes non-ductile seismic design for structures with gravity shear/punching shear strength ratio of over 0.4 by assigning the **m** value of 1.0 corresponding to elastic response.

It should be noted that the structure under consideration has been subjected to a rather low gravity load level, mainly consisting of dead load (self-weight); the roof area is mainly used as the ground for the public tennis courts, corresponding to a rather low occupancy load. Snow load governs over the occupancy load in the live load category.

Shear stresses at the critical slab sections and the corresponding punching shear resistance have been determined according to CSA [6]. The key features of the shear stress analysis are summarized below:

- Two critical sections, one at the capital and other one at the drop panel location, have been considered in the analysis.
- The factored shear stress ($\mathbf{v_f}$) has been determined using the load combination: 1xDL+0.25xLL
- The factored shear stress resistance ($\mathbf{v_c}$) has been determined as per A23.3-94 Cl.13.4.4.
- Concrete material properties used in the analysis were: f_c'= 40 MPa and ϕ_c=0.8

Initially, the $\mathbf{v_f/v_c}$ ratio has been determined considering only gravity load effects, and this value was used to select the modeling parameters for slab hinge properties (per FEMA 356). The results of this analysis are as follows:

1. For the critical section at the drop panel: $\mathbf{v_f/v_c}$= 0.17 ($\mathbf{v_c}$ = 1.0 MPa)
2. For the critical section at the capital: $\mathbf{v_f/v_c}$= 0.07 ($\mathbf{v_c}$ = 1.4 MPa)

The above analysis has revealed that the higher shear stresses would develop in the critical section at the drop panel location. However, it should be also noted that the shear stress ratio at both locations is less than 0.2; according to FEMA 356 [4], such a low value of $\mathbf{v_f/v_c}$ ratio indicates a larger ductility capacity of flat slabs (corresponding to **m** value in the range of 2 to 3, depending on the seismic performance level).

The shear stress demand ($\mathbf{v_f}$) at the critical sections was determined at the various stages of pushover analysis, corresponding to the increasing displacement levels. The demand was compared to the factored shear stress resistance ($\mathbf{v_c}$). The results have shown that the $\mathbf{v_f/v_c}$ value is less than 0.4 even at the ultimate stages of the pushover analysis, when some of slab hinges have reached the CP performance level (this is true both for the interior and perimeter frames).

The Results

The results of the pushover analysis have shown that at the OBE (I=1.0) design earthquake level corresponding to the "1 x target" displacement, the structure is expected to perform at the Life Safety (LS) level with the exception of two columns showing the CP performance that would need to be retrofitted. However, at the OBE (I=1.5) earthquake level corresponding to the "1.5 x target" displacement, the structure is not expected to meet the requirements for the Life Safety (LS) performance. By and large, slab elements are expected to perform at the Collapse Prevention (CP) level, and two columns located at the midspan area of perimeter frames are expected to collapse, while retaining a limited gravity load-bearing capacity. Retrofit would be possible, however uncertain in localized areas of column collapse. Diagrams showing deformed shape of a typical interior frame at OBE (I=1) and OBE (I=1.5) earthquake level and the location of plastic hinges are presented in Figure 5.

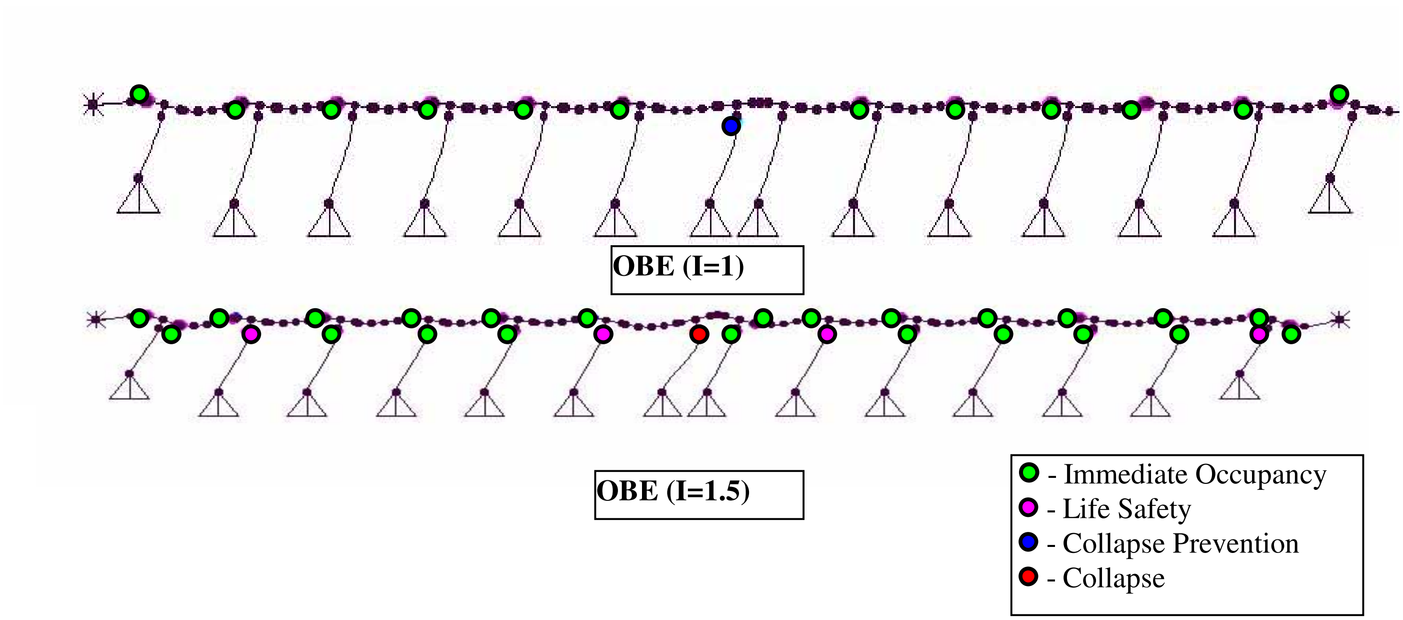

Figure 5: Deformed shape of a typical interior frame at the OBE(I=1) and OBE(I=1.5) earthquake levels showing plastic hinge locations and seismic performance levels.

A pushover curve showing base shear force versus the lateral displacement for an interior frame is presented in Figure 6.

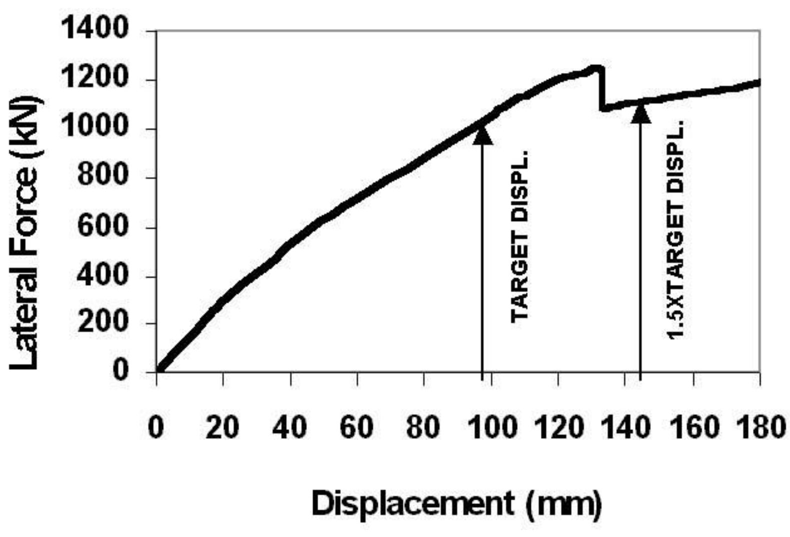

Figure 6: A pushover curve illustrating base shear force vs. lateral displacement for an interior frame structure.

The results of the analysis for a typical interior frame are summarized in Table 8. Note that the interpretation of damage conditions for slabs and columns has been developed jointly with the project peer reviewers Wight [10] and Anderson [12].

Table 8: Interior Frame Pushover Analysis - Summary of the Results

	Seismic Evaluation Levels	
	Target Displacement	1.5x Target Displacement
Design Earthq. Levels	OBE (I=1.0)	OBE (I=1.5)
General Damage	Slabs: IO Columns: CP (only 2 - one each direction of motion); others elastic	Slabs: LS Columns: Local collapse (2 columns); IO (all other columns)
Slabs	All hinges: IO performance IO performance: Some visible cracking near the slab-column connections at drop panel locations is expected	Most hinges: IO performance 8 hinges (at drop panel locations): LS performance LS performance: More extensive flexural cracking in the slabs near the slab-column connections (at the drop panels); some minor spalling at the bottom of the slab in a few locations.
Columns	Only 2 columns deformed in inelastic range (plastic hinge formed): CP performance; all other columns have remained elastic CP performance: Extensive flexural cracking and spalling of concrete cover is expected at the top of the column (plastic hinge region) with a chance of a brittle shear failure in the column due to inadequate ties.	2 columns (at the midspan): Collapse; other columns: IO performance IO performance: Minor cracking in the columns, probably not visible to the unaided eye Collapse: column would lose its lateral load-resisting capacity, and would retain a limited gravity load-bearing capacity; portions of the slab adjacent to the column might get damaged due to a loss of the support.
Repair	Feasible at a reasonable cost (at the 2 column locations there are chances of local “drop-down” slab collapse)	Possible, but uncertain in local areas of column collapse; likely to be costly in local areas.

CONCLUSIONS

The nonlinear static (pushover) analysis was used in the Cape Horn reservoir seismic study to obtain a more precise insight into the expected seismic performance of the roof structure beyond that obtained by the linear static and response spectrum analyses. The results of the pushover analysis have confirmed the locations within the structure that would need to be repaired after an earthquake. The analysis has also shown that the damage to the reservoir roof would be repairable after the OBE (I=1) and OBE (I=1.5) earthquake, however some areas of the reservoir would need an extensive repair including a replacement at some locations after an OBE (I=1.5) earthquake. Damage to the reservoir at the MCE earthquake would be very extensive and beyond repair. After considering the results of the seismic study, GVWD decided to undertake a retrofit of the reservoir roof using viscous damper devices. The retrofitted reservoir roof is expected to withstand the MCE earthquake without a significant damage.

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