PRELIMINARY GEOTECHNICAL EVALUATION WASHINGTON PARK RESERVOIR IMPROVEMENTS

Portland, Oregon

REPORT
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Black & Veatch

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EXECUTIVE SUMMARY

The Portland Water Bureau (PWB) is evaluating options to replace water supply Reservoirs 3 and 4 with a buried reservoir at the location of the existing Reservoir 3. The existing reservoirs have experienced distress from a large, upslope landslide since their construction in the 1890's. The primary purpose of this study is to evaluate options to mitigate landslide impacts on new infrastructure and to estimate landslide deformations that could occur during a seismic event.

Four mitigation options were evaluated for the landslide. These options include: 1) isolate new structures from potential future landslide movements with a compressible inclusion and backfill Reservoir 4; 2) stabilize the landslide with an anchor block wall; 3) stabilize the landslide with a vertical shoring wall; and 4) stabilize the landslide with large-diameter shear piles. Options 2 through 4 involve structurally stabilizing the landslide using high capacity ground anchors or heavily reinforced piles. The structural options could also be combined with backfilling of Reservoir 4.

Landslide models were developed to perform limit-equilibrium slope stability analyses. Back analyses of shear strength along the failure zone were performed on existing landslide conditions assuming the current factor of safety (FS) is close to 1.0. Subsequent analyses were performed to evaluate the benefit from backfilling at the toe of the landslide and the forces required to achieve a target FS improvement for the structural solutions. The results range from an improvement in FS to 1.03 for Option 1, to FS = 1.05 to 1.07 for the structural options. Stability analyses assume that existing drainage tunnels installed in the 1890's and early 1900's will continue to function and work with proposed mitigation measures to achieve calculated stability improvements.

Seismic studies were completed to estimate ground motions likely to be experienced at the site for earthquakes with return periods of 475 years and 2,475 years. Newmark-type analyses show that displacements for the 475-year and 2,475-year seismic events would generally be less than ¼ inch and 4 inches, respectively. All mitigation options would be able to tolerate the estimated displacements during a 475-year event. However, Options 2 and 3, which involve the use of ground anchors, could be severely damaged from seismic landslide displacements during a 2,475-year event. It is our understanding that the City currently designs their water infrastructure for a 475-year event.

Comparative construction cost estimates for the four options range from \$23M to \$73M. These figures include the costs for landslide mitigation and other geotechnical aspects of construction such as excavation shoring and reservoir foundations, but do not include costs for the reservoir, piping, and appurtenant structures. The cost estimates include approximately \$9M for deep foundations to support a new buried reservoir, which would be required irrespective of landslide mitigation. Option 1 is the least costly option but also achieves the lowest improved FS. The structural options are progressively more expensive but provide a higher degree of stabilization. Based on discussions with PWB and the planning team, Option 1 would satisfy reservoir design and performance criteria with an acceptable level of risk. Option 1 at Reservoir 3 would accommodate potential continued landslide movement without loading the new reservoir. Option 1 (with fill Option B) at Reservoir 4 would change the aesthetics of the site, but would likely stop landslide movements.

1. INTRODUCTION

1.1 General

The Portland Water Bureau (PWB) is evaluating options to replace the open reservoirs at Washington Park. The current preferred alternative involves constructing a 15 million gallon buried reservoir in the current Reservoir 3 location and decommissioning Reservoir 4. This report summarizes a preliminary geotechnical evaluation of conceptual options to stabilize the landslide above Reservoirs 3 and 4, and presents general construction and foundation recommendations for the proposed new reservoir.

1.2 Site and Project Background

PWB operates two open reservoirs located in Washington Park approximately 2 miles west of downtown Portland (see Figure 1). The reservoirs were constructed between October 1893 and September 1894. Reservoir 3 has a capacity of approximately 16½ MG with a surface water level near elevation 299.5 feet. Reservoir 4 has a capacity of approximately 18 MG with a surface water level near elevation 229.5 feet. Construction of the reservoirs reactivated a large, ancient landslide upslope of the work (see Figure 2). The landslide is approximately 1,700 feet long and 1,100 feet wide at the base of the slope. Since construction, both reservoirs have experienced damage due to continuing landslide movements. To slow landslide movements, the City constructed a network of drainage tunnels in the early 1900's. The tunnels were successful in slowing, but not stopping, landslide movement. Regular monitoring over the past 20 years has shown that the landslide continues to move at roughly ⅛-inch per year. Quarterly monitoring of slope inclinometers has also shown that the landslide tends to move more during the wet season than during the dry part of the year. A detailed description of movement history and prior landslide studies is included in a Geotechnical Data Report prepared under a companion study (Cornforth, 2010).

1.3 Scope of Work

The scope of work for this task order is to evaluate conceptual options and estimate costs to stabilize the ancient landslide at Reservoirs 3 and 4. This information would be used to decide if construction on the existing site is feasible and what design considerations would need to be included in the project development. The conceptual mitigation options would be refined during future work phases.

Our scope of work includes the following tasks:

- Develop geologic models of the landslide above Reservoirs 3 and 4
- Perform stability analyses for current conditions
- Perform site-specific seismic studies
- Calculate estimated earthquake-induced landslide movements and loadings
- Evaluate potential for earthquake-induced liquefaction at the site
- Evaluate four options to stabilize the landslide and/or mitigate landslide movements

- Develop comparative construction costs for mitigation options
- Develop preliminary geotechnical design recommendations for new reservoir structures and general site development
- Calculate lateral landslide loading on proposed structures
- Summarize preliminary evaluation and geotechnical recommendations in a Preliminary Geotechnical Evaluation Report

2. PROJECT DESCRIPTION

2.1 General

In response to recent Federal water quality regulations, PWB is planning to replace open water reservoirs at Washington Park with buried storage. New facilities and piping would be located at the toe of a large, slow-moving landslide. The project to replace the open reservoirs would include constructing mitigation measures to stabilize the landslide or constructing new facilities that could accommodate future landslide movements.

2.2 Companion Studies

Several engineering and geotechnical studies have been performed for the open reservoirs and landslide at Washington Park. In fall 2010, Cornforth Consultants completed nine new exploratory borings, installed new landslide instrumentation, and performed down-hole geophysical testing at the site in a companion study to work for this report. Our recent Geotechnical Data Report (Cornforth 2010) presents a summary of the geologic setting, subsurface conditions, and previous studies completed at the site. The report also summarizes recent explorations and instrumentation work.

2.3 Objectives of Study

The primary objectives of the current study are to evaluate conceptual mitigation options for the landslide impacting Reservoirs 3 and 4, and estimate construction costs for the mitigations. Another important objective is to estimate the displacement of the landslide during seismic events of different return periods. Ancillary objectives include developing standard geotechnical design recommendations for foundations and retaining structures associated with new reservoir infrastructure. The study and recommendations included in this report are not intended to be used for construction. Concepts included would be refined during final design at a later time.

3. LABORATORY TESTING

3.1 General

Laboratory testing was completed on selected soil samples retrieved from recent subsurface explorations to aid in the evaluation of seismic site response. The tests included six Atterberg limits and six grain size analyses.

3.2 Atterberg Limits

Liquid and plastic limits (Atterberg limits) were determined for seven samples. Test procedures complied with ASTM D-4318. Results of the Atterberg Limits tests are summarized in the following table and on Figure 3.

Table 3-1: Index Test Results

Boring	Sample No.	Depth (feet)	Natural Moisture (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Percent Passing No. 200
CC-2	S-4	12	26	NP	NP	NP	95.3
CC-4	S-4	10	41	56	36	20	72.0
CC-7	S-5	12.5	27	40	22	18	95.5
CC-8	S-1	2.5	32	36	25	11	97.9
CC-8	S-7	20	33	52	22	30	99.7
CC-9	S-7	20	28	40	21	19	92.2

3.3 Grain Size Analyses

Grain size analyses were performed on the six samples listed in the table above to determine the amount of fine-grained (finer than No. 200 sieve) material in the soil. Tests were conducted by determining the dry weight of the total sample, washing the sample over the No. 200 sieve, and then determining the dry weight of the material retained. The results of the grain-size analyses are shown in Table 3-1.

4. SEISMIC STUDIES

Abbreviations and symbols used in this section:

CSZ – Cascadia Subduction Zone
GMPE – Ground Motion Prediction Equation
M – Earthquake Magnitude
NGA – Next Generation Attenuation
NSHM – National Seismic Hazards Maps
PEER – Pacific Earthquake Engineering Research Center
PGA – Peak Ground Acceleration
R – Radius from Earthquake Source
SF – Scaling Factor
USGS – United States Geological Survey

4.1 General

Seismic studies were completed to estimate landslide displacements that could be expected to occur during a large earthquake. Ground motions were calculated for potential seismic sources at varying return periods. Response spectra were developed for each seismic source and return period to assist with selection of input rock motion acceleration time histories. For each input acceleration time history, a one-dimensional site response analysis was completed to develop ground motion response along the landslide shear zone. Predicted ground motions were then used to estimate displacements for the landslide using Newmark analysis techniques.

4.2 Evaluation of Seismic Hazards

Seismic hazard analyses determine the frequency and sources of ground motions from future earthquakes. USGS recently published 2008 National Seismic Hazards Maps (NSHM), with updates in 2008 and 2010 (Petersen, et al. 2008, USGS 2008a, USGS 2008b, USGS 2010). The 2008 update (Petersen, et al. 2008) includes: (i) new recurrence distribution and ground motion models for Cascadia Subduction Zone (CSZ) interface earthquakes; (ii) a new source zone around Portland and the coast to account for potential of large, deep (intraslab) earthquakes; and (iii) three new Next Generation Attenuation (NGA) ground motion equations for crustal faults in the Western U.S.

Analysis of the seismic hazard at Washington Park utilized the 2008 USGS Interactive Deaggregation (USGS 2008c, USGS 2009) to evaluate probabilistic hazard levels and source contributions for various return periods. Table 4-1 shows the results of the probabilistic seismic hazard deaggregation for 144-year, 475-year and 2,475-year return periods. More detailed results of the deaggregation are shown in tabular form in Appendix A. The 0.0 second spectral acceleration values on rock (Site Class B) are 0.09g, 0.20g and 0.44g for 144-year, 475-year and 2,475-year return periods, respectively (the 0.0 second spectral acceleration is equivalent to peak ground acceleration (PGA) at the ground surface).

Table 4-1 indicates that the random crustal (shallow gridded earthquakes) and CSZ Interface are the predominate contributors to the seismic hazard at the site, each accounting for approximately

30 percent of the total hazard. Additional significant contributors are the Grant Butte Fault, the Portland Hills Fault and the CSZ intraslab source.

Appendix A includes histogram representations of the deaggregation results for 0.0 second, 0.2 second and 1.0 second spectral acceleration, respectively, for return periods of 144-years, 475-years and 2,475-years.

Table 4-1: Results of Probabilistic Seismic Hazard Deaggregation

		Percent Contributions from Principal Sources at PGA					
			OR-WA Faults			Cascadia Subduction Zone	
Return Period	PGA (g)	Random Crustal (gridded)	Grant Butte	PHF Char[1]	PHF-GR Mag[2]	Interface	Intraslab
144-year	0.09	36.4	3.7	<2	<2	29.8	25.4
475-year	0.20	28.0	4.0	<2	4.1	34.3	23.5
2,475-year	0.44	23.0	<2	6.7	15.3	33.3	17.8

[1]PHF Char = Portland Hills Fault - Characteristic Earthquake Model

[2]PHF-GR Mag = Portland Hills Fault – Gutenberg-Richter Magnitude-Frequency Model

[3]Based on 2008 USGS Interactive Deaggregation with 2009 Update

4.3 Seismic Sources

The probabilistic seismic hazard deaggregation (Table 4-1) indicates five sources are predominant hazards at Washington Park. These include the random crustal (shallow gridded) earthquake, the Portland Hills Fault, the Grant Butte Fault, and the CSZ interface and intraslab sources. Appropriate earthquake magnitude and distance pairs were developed for these sources as shown in Table 4-2.

For the random crustal (shallow gridded) source, various combinations of magnitude (M6 to 6.5) and distance (10-15 kilometers) have been used in practice in western Oregon. We selected a magnitude 6.5 with a distance of 10 km, which is a conservative combination for the random crustal earthquake in view of the deaggregation histograms (Appendix A). The Grant Butte Fault source, with smaller earthquake magnitude (M) and larger radius (R) from the earthquake source, than both the Portland Hills Fault and Random Crustal sources, was not considered further in the analysis.

Recurrence intervals for both the interface and intraslab subduction earthquakes are estimated at 500 years (Petersen et al. 2008). Maximum credible earthquake magnitudes for these two sources are a M9.2 megathrust event for the interface zone, and a M7.2 earthquake for the intraslab zone (Petersen et al. 2008).

Table 4-2: Magnitude/Distance Pairs for Principal Earthquake Sources

Source	Source to Site Distance	Estimated Moment Magnitude
Random Crustal	10 km	6.5
Grant Butte Fault	12 km	6.2
Portland Hills Fault	1.5 km	7.0
CSZ Interface	88 km	8.5 to 9.2
CSZ Intraslab	53 km	7.2

[1]Based on 2008 USGS Interactive Deaggregation with 2009 Update

4.4 Ground Motion Models and Target Response Spectra

Ground Motion Prediction Equations. Acceleration response spectra were developed deterministically using Ground Motion Prediction Equations (GMPE) for rock sites. As discussed in later sections, the rock site ground motions were incorporated into site response analyses to determine appropriate ground motions within the soil above bedrock.

USGS has adopted the NGA models developed under the leadership of the Pacific Earthquake Engineering Research Center (PEER) to derive spectral accelerations for crustal earthquake sources. These models include: Abrahamson-Silva 2008, Boore-Atkinson 2008, Campbell-Bozorgnia 2008, Chiou-Youngs 2008, and Idriss 2007. The five NGA ground motion models were used to derive response spectra (5% damping ratio) for the random crustal and Portland Hills Fault sources. For this analysis the crustal source GMPE's were equally weighted (20% each).

USGS adopted the following three ground motion models for the CSZ interface sources in the 2008 Update of NSHM (Petersen et al. 2008), with weights as indicated:

- Youngs et al. (1997) – 0.25
- Atkinson and Boore (2003, global model) – 0.25
- Zhao et al. (2006) – 0.5

Attenuation relationships developed by Youngs et al. (1997), Atkinson and Boore (2003), and Zhao et al. (2006) with the weighting factors as shown above were used to derive the target response spectrum (5% damping ratio) for the CSZ interface earthquake sources.

For the CSZ intraslab earthquake source, the USGS adopted the following two ground motion models in the 2008 Update of NSHM (Petersen et al. 2008), with weights as indicated:

- Youngs et al. (1997) – 0.5
- Atkinson and Boore (2003, global model) – 0.5

Attenuation relationships developed by Youngs et al. (1997) and Atkinson and Boore (2003), with the weighting factors shown above, were used to derive the target response spectrum (5% damping ratio) for the M=7.2 and R=53 km intraslab earthquake.

Return Period and Response Spectra. Deaggregating the seismic hazard into individual sources allows determination of target response spectra and individual acceleration time histories. Developing these response spectra also requires consideration of the recurrence interval (i.e. how often an earthquake occurs) of the source event as well as the return period used for design. As an example, the CSZ source has a documented average recurrence interval of approximately 500 years (typically range of recurrence interval is 350 to 700 years); therefore, for a probabilistic return period of 500-years, the model indicates that a median value for the CSZ source would be appropriate. This information is reflected in the epsilon term, ε, used by USGS to indicate the number of standard deviations above or below the median.

Selection of an appropriate ε for the GMPE is often not explicit. Traditionally it has been either 0 (median or 50th percentile) or 1 (median plus one standard deviation or 84th Percentile). The selection of ε should consider how conservatively the M and R values were selected.

To assist in the selection of ε, the response spectra for magnitude, M, and radius, R, in Table 4-2 were compared with the uniform hazard response spectra for return periods of 144-years, 475-years and 2,475-years. The ε values were selected such that the calculated response spectra matched fairly well with the uniform hazard response spectra for the corresponding return period. Generally, the selected ε values were close to the values provided in the interactive deaggregations (Appendix A) for the site. Response spectra for the individual sources considered ε values ranging from -3 to 1. Selected values of ε for the return periods of interest are shown in Table 4-3.

Table 4-3: Selected ε-values Based On Uniform Hazard Response Spectra Comparison

	144-yr Return Period		475-yr Return Period		2,475-yr Return Period	
Source	ε	PGA	ε	PGA	ε	PGA
Random Crustal	-1	0.11	0	0.20	1	0.36
Portland Hills Fault	-3	0.10	-1.5	0.24	-0.5	0.41
CSZ Interface	-1	0.08	0	0.16	1	0.32
CSZ Intraslab	-1	0.11	0	0.21	1	0.42

[1]PGA based on GMPE's

Based on selected ε values in Table 4-3 above, the GMPE's were used to develop target horizontal response spectra using 5% damping for a rock site. The response spectra for these sources, when considered together, closely match the uniform hazard response spectra for the given return period. The random crustal, Portland Hills Fault and CSZ intraslab closely match the uniform hazard response spectra in the short period range (PGA to about 0.5 sec), and the CSZ interface and intraslab closely match the ground motions in the longer period range (about 1 second).

The random crustal and Portland Hills Fault were compared to determine if they could be analyzed together. The Portland Hills Fault yielded the highest ground motions, longer duration, and closer source-to-site distance, and therefore was selected to represent ground motions for the crustal fault sources.

4.5 Ground Motion Time History Selection

Ground motion time histories were selected for landslide deformation analyses. Selected ground motion records were chosen such that the earthquake (magnitude), site (geology and site-to-source distance), and response (spectral acceleration) characteristics closely match those of the target response spectra. The time history search used the response spectra for the 2,475-year return period level motions as the target. Time histories for other return periods (144-year and 475-year) used the same time histories, scaled appropriately to match the respective response spectra.

We searched available recorded ground motion records and response spectra using a number of cataloged sources including: the PEER Strong Motion Database; the PEER/NGA Database; and the Consortium of Organizations for Strong Motion Observation System (COSMOS) Virtual Data Center. Further details on searches for the individual sources are discussed in the following sections.

Ground motion time-histories reflect the nature and characteristics of the associated fault type, magnitude of the event, and the source-to-site distance. For instance, the design Subduction zone event is expected to have larger magnitude, lower PGA, and longer duration (longer than 1 minute), than the design crustal event. The time histories used for site response analyses reflect the level of shaking and duration. Since landslide deformation analyses are based on the acceleration time histories, the estimated displacements reflect the magnitude, source-to-site distance, and duration.

Portland Hills Fault Source. For the Portland Hills Fault source, the PEER Ground Motion Database (http://peer.berkeley.edu/peer_ground_motion_database/site) for shallow crustal earthquakes was searched for time histories. Since many records are available, we did not attempt to modify time histories for the 2,475-year return period. The crustal ground motions that met the Portland Hills Fault target magnitude, distance, and rock site criteria were further analyzed by comparing their response spectra (at 5% damping) with the target spectra. Three acceleration time histories were selected that closely matched the target response spectra, with particular emphasis in the period range from 0.1 to 0.5 seconds. Selected ground motions are shown in Table 4-4. Since the NGA relationships are based on the geometric mean of the two horizontal components of the time history, we compared the geometric mean spectra for each ground motion record with the target response spectra (from NGA relationships). Subsequently, the horizontal component (of the orthogonal pair) with the greatest PGA was selected for use in the deformation analyses.

Table 4-4: Selected Ground Motions for Portland Hills Fault Earthquake Event

Earthquake	Station	Magnitude	Distance (km)	PGA[1] (g)	475-yr SF[2]	144-yr SF[2]
Loma Prieta 10/18/1989	Saratoga – Aloha Ave	6.9	8.5	0.38	0.5	0.25
Chi-Chi, Taiwan 9/25/1999	TCU078	6.2	7.6	0.47	0.6	0.2
Northridge 1/17/1994	LA - UCLA Grounds	6.7	22.5	0.45	0.6	0.2

[1]PGA shown is for orthogonal pair with maximum PGA. For 2,475-year return, scaling factor is 1.0.
[2]SF is scaling factor applied to modify motions for various return periods.

CSZ Interface Earthquake. Subduction zone ground motions were selected from earthquake records for two subduction zone events (the 1985 Michoacan, Mexico and the 1985 Valapraiso Chile earthquakes). A third motion, which is a synthetic time history developed for Bull Run Dam No. 2 was included in the set of ground motion records. The earthquake response spectra (at 5% damping) were compared to the target response spectrum. The response spectra with similar characteristics were selected and scaled to match spectral accelerations for the period range of 0.1 to 0.5 seconds. Table 4-5 shows parameters of the selected earthquake ground motions.

Table 4-5: Selected Ground Motions for Interface Subduction Zone Earthquake Event

Earthquake	Station	Magnitude	Distance (km)	PGA	2,475-yr SF[1]	475-yr SF[1]	144-yr SF[1]
Valparaiso 3/3/1985	Llayllay, Chile	7.8	93	0.35g	0.8	0.4	0.2
Michoacan 9/19/1985	La Union, Mexico	8.1	83.9	0.17g	1.4	0.7	0.4
CSZ Interface Synthetic	-	8.5	174.0	0.12g	2.4	1.1	0.6

[1]SF is scaling factor applied to modify motions for various return periods.

CSZ Intraslab Earthquake. Subduction zone ground motions were selected from earthquake records for two intraslab zone events (the 2001 El Salvador and the 1997 Michoacan, Mexico earthquakes), and a synthetic time history developed for the City of Portland groundwater pump station. The earthquake response spectra (at 5% damping) were compared to the target response spectrum. The response spectra with similar characteristics were selected and scaled to match spectral accelerations for the period range of 0.1 to 0.5 seconds. Table 4-6 shows parameters of the selected earthquake ground motions.

Table 4-6: Selected Ground Motions for Intraslab Subduction Zone Earthquake Event

Earthquake	Station	Magnitude	Distance (km)	PGA	2,475-yr SF[1]	475-yr SF[1]	144-yr SF[1]
El Salvador 1/13/2001	Observatorio	7.6	91	0.38g	1.0	0.5	0.2
Michoacan 1/11/1997	Caleta de Campos	7.1	37	0.40g	1.0	0.5	0.2
CSZ Intraslab Synthetic	-	7.5	65	0.30g	1.3	0.65	0.3

[1]SF is scaling factor applied to modify motions for various return periods.

4.6 Site Response Analysis

A geologic profile of the landslide was generated using existing subsurface information and data from recent explorations. Initially, profiles were generated at two locations within the landslide mass (FH-1 and CC-5) where shear wave velocity profiles were measured. Preliminary analyses indicated that there was negligible difference in estimated deformations for the two locations; therefore, the profile at FH-1 was used because its location is considered more representative of the entire landslide mass (located away from the landslide toe in the translation portion of the slide). The profile in CC-5 would be more representative of the ground motions likely to felt by the new reservoir structure. The geologic profile and shear wave velocity data from boring FH-1 were incorporated into a generalized soil profile for site response analyses. The soil profile and engineering parameters used for site response analyses are summarized in Table 4-7 below.

Table 4-7: Soil Profile for 1-D Site Response Analyses

Depth (feet)	Soil Description	Unit Weight (lb/ft^3)	Shear Wave Velocity (ft/sec)
0 to 45	Medium stiff to stiff, silty clay to clayey silt	120	750 to 950
45 to 75	Stiff to hard, sandy, clayey silt with gravel	110	1,160 to 1,830
75 to 95	Gravel sized rock fragments in silty sand	125	2,200 to 2,500
95 to 135	Fractured basalt	135	3,000 to 3,200
135 +	Basalt bedrock	140	3,500

Using the generalized subsurface profile, the seismic response of the overburden soil to the input bedrock ground motions was calculated using the program SHAKE2000 (Ordonez 2009). The horizontal equivalent acceleration (HEA) time series along the base of the landslide mass was used to represent the seismic coefficient time series of the landslide mass. The HEA/g is the ratio of shear stress to total vertical stress at the landslide surface as a function of time for a given earthquake input ground motion. The seismic coefficient time series were used to estimate deformation using the Newmark method, as discussed in Section 5.11.

5. LANDSLIDE MITIGATION STUDIES

5.1 General

Four mitigation options were evaluated to stabilize the landslide above Reservoirs 3 and 4. The first option for each reservoir involves making modest improvements to the stability of the landslide and designing a new Reservoir 3 structure to accommodate future landslide movements. The remaining options for each reservoir involve implementing structural mitigation measures to achieve a target factor of safety (FS) for slope stability. The following sections describe the different mitigation options and details of the slope stability analyses and deformation estimates.

5.2 Reservoir 3 Option 1 – Accommodate Slide Movements

The first option for Reservoir 3 involves constructing a new reservoir slightly east of the existing reservoir site and isolating it from future landslide movements with a compressible inclusion. A plan and section of Option 1 are shown in Figure 6. Temporary excavation shoring walls would be required on all but the west side of the new reservoir. During reservoir construction, a compressible inclusion would be placed on the west wall of the new structure. Imported granular material would be used to backfill around the reservoir. A reflecting pool would be constructed to maintain the aesthetics of the site.

The compressible inclusion shown in Figure 6 would accommodate future landslide movements without applying additional load to the new reservoir. A compressible inclusion is an engineered thickness of low strength material designed to crush at a specific load. Expanded polystyrene (EPS) is commonly used for this type of application. The strength and stiffness of the EPS would be specified to be slightly higher than earth pressures exerted by reservoir backfill. Future landslide movement (static or seismic) would exert lateral pressures greater than the compressive strength of the EPS. EPS deforms plastically to strains on the order of 90 to 95 percent at any load above its compressive strength. Therefore, the compressive inclusion may absorb landslide movements while applying a constant load to the structure.

5.3 Reservoir 3 Option 2 – Anchor Block Wall

A second option for Reservoir 3 would involve installing ground anchors on the west slope of the existing reservoir to resist landslide loads. A plan and section of Option 2 are shown in Figure 7. A drill bench would be excavated on the western reservoir slope to allow ground anchors to be installed using conventional equipment and techniques. Concrete bearing pads would be placed or cast in place against the excavated slope. Ground anchors would then be drilled through the bearing pads and bonded into stable ground below the landslide shear zone. Ground anchors would prevent landslide movements under static conditions. Seismically-induced landslide movements could damage the anchors. This topic is discussed in further detail in subsequent sections.

5.4 Reservoir 3 Option 3 – Landslide Shoring Wall

A third option evaluated for Reservoir 3 would involve constructing a landslide shoring wall along the western edge of the existing reservoir. A plan and section of Option 3 are shown in