Figure 8. Constructing the landslide shoring wall would be very similar to building a conventional shoring wall. However, since this option involves removing soil at the toe of the landslide, the anchor loads would be substantially larger than in Option 2. The large anchor loads may require vertical members consisting of drilled shafts rather than more conventional steel soldier piles. Once vertical members are completed, the existing reservoir slope would be excavated in stages while ground anchors are installed and tensioned. Ground anchors would prevent landslide movements under static conditions, but could be damaged by earthquake-induced landslide movements, as discussed in subsequent sections.

5.5 Reservoir 3 Option 4 – Shear Piles

The fourth option evaluated for Reservoir 3 would involve constructing a row of large-diameter, reinforced concrete shear piles that span the landslide shear zone. Section and plan views of Option 4 are shown on Figures 9A and 9B. Shear piles are heavily-reinforced drilled shafts designed to transfer landslide loads to stable material below the shear zone. Shear piles would have to be located upslope of the reservoir where landslide shearing is occurring directly on top of competent bedrock because the soil beneath the shear zone near the reservoir is not competent enough to resist loading from shear piles. Preliminary calculations indicate that shafts between 6 and 10 feet in diameter spaced approximately two shaft diameters center-to-center would be required to develop the required loads to stabilize the landslide. Stability analyses for Option 4 assume that the west slope of the reservoir would not be excavated. Shear piles would prevent landslide movements under static conditions once sufficient landslide movement occurs to mobilize capacity in the piles. Preliminary calculations indicate that shear piles are not likely to be heavily damaged by seismically-induced landslide movements.

5.6 Reservoir 4 Option 1 – Backfill Reservoir

The first remedial option for Reservoir 4 involves placing fill in the decommissioned reservoir. A plan and section of Option 1 are shown in Figure 10. Two fill geometries were evaluated as shown on the cross section. Fill option A would involve filling the existing reservoir to Elevation 230 feet. Fill option B would place additional fill on the slope west of the reservoir. Option 1 for Reservoir 4 could be combined with any of the four options described above for Reservoir 3.

5.7 Reservoir 4 Option 2 – Anchor Block Wall

The second option for Reservoir 4 involves constructing ground anchors on the slope near the base of the landslide to resist landslide loads. A plan and section of Option 2 are shown in Figure 11. Construction techniques and sequence would be similar to that described for ground anchors at Reservoir 3. Fill could be placed in Reservoir 4 in conjunction with ground anchors to lower the number of anchors required.

5.8 Reservoir 4 Option 3 – Landslide Shoring Wall

The landslide shoring wall option discussed above for Reservoir 3 is not recommended for Reservoir 4. Implementing the shoring wall option at Reservoir 4 would require large amounts of unnecessary excavation and involve removal of several large trees. For these reasons, the landslide shoring wall option was not evaluated for Reservoir 4.

5.9 Reservoir 4 Option 4 – Shear Piles

The final remedial option for Reservoir 4 involves constructing a row of large-diameter, reinforced concrete shear piles that span the landslide shear zone. A plan and section of Option 4 are shown in Figure 12. Shear piles would be located upslope of the reservoir where the shear zone is located directly over bedrock. Fill could be placed in Reservoir 4 in conjunction with shear piles to lower the number of piles required.

5.10 Stability Modeling

Stability modeling of the landslide was performed using SLOPE/W (GeoSlope 2007). SLOPE/W uses Spencer's limit-equilibrium method to determine the FS for a slope. Two landslide cross sections were developed for the landslide as shown in Figures 4 and 5. The shear zone location is based on slope inclinometer readings, exploratory borings, and descriptions of materials encountered in dewatering shafts. The shear zone roughly follows top of bedrock for the majority of the landslide length. Near the landslide toe, the shear zone diverges from bedrock and daylights in the western slope of Reservoirs 3 and 4. The location of the landslide toe is based on descriptions in Clarke (1904).

Groundwater. The position of the groundwater is an important variable in the stability analyses. For analysis purposes, there are two groundwater conditions that can be used to model the landslide. The 1893 (or "pre-tunnel") groundwater level is the condition that was present when the reservoirs were initially constructed. Pre-tunnel groundwater levels are shown in the logs of the original exploratory borings and dewatering shafts (Clarke 1904). However, groundwater observations from the dewatering shafts do not necessarily reflect the groundwater pressures acting on the landslide shear zone. In large landslide masses it is not uncommon to have perched groundwater within the landslide mass that does not influence the shear strength on the shear zone. In addition, extensive dewatering from the tunnel work has altered groundwater levels from the pre-tunnel condition. For these reasons, stability analyses should not be based on the pre-tunnel groundwater levels described in Clarke (1904).

Following reservoir construction and landslide reactivation, a system of dewatering tunnels was constructed near the shear zone. The tunnels lowered groundwater levels within the landslide mass. The post-tunnel groundwater is currently monitored with several piezometers installed specifically to measure pore-water pressures near the shear zone of the landslide. The piezometers indicate groundwater levels substantially lower than pre-tunnel levels. Monitoring of landslide movements during the last 30 years indicates that the landslide is close to a FS of 1.0; therefore, current groundwater levels were used to back analyze the landslide.

Back Analysis. Stability analyses were performed to back analyze the shear strength along the landslide shear zone. The landslide mass can be subdivided into three different blocks with different base inclinations (see Figures 4 and 5). There is an upper driving block with a relatively flat base, an intermediate block with a steeper base angle, and a lower block with a relatively flat base. This base geometry resulted in convergence problems for SLOPE/W. To overcome convergence issues, we back-analyzed each block separately for a FS of 1.0. We started by evaluating the lower landslide block assuming that the shear surface extended to the ground surface from the western limit of the lower block at an angle of 60 degrees to horizontal. The friction angle along the base of the block was varied to get a FS near 1.0; the friction angle on the lower block remained fixed for subsequent analyses. We then evaluated the middle block by calculating the stability of the middle and lower blocks as a unit. The friction angle of the middle block was varied to achieve a FS near 1.0. The final step was to evaluate the stability of all three blocks as a unit. The friction angle of the upper block was varied to get the calculated FS of the whole mass near 1.0. The back-calculated friction angles for each block are shown in Figures 6 to 12.

Mitigated FS. Key factors that affect the target FS for landslide improvement include the rate of landslide movement, the level of understanding of the landslide, the size of the landslide, and the consequences of failure. For very large landslides with a well-defined shear zone and a long history of movement data, like the Washington park landslide, it is possible to work toward a low target FS. By comparison, for slope stability problems (e.g. roadway cuts or fill slope developments) where the geometry and strength information is poorly defined, target FS are typically near 1.5. In the case of the Washington Park landslide, we can calculate fairly accurately the reduction in stabilizing force that caused the 1893 reactivation, and how much improvement resulted from the drainage tunnels. In addition, based on back-calculation of the failure zone, we also have a good understanding of the average shear strength along the failure zone. This allows us to use the back-calculated model to evaluate the relative improvement of various stabilization options.

Slope stability analyses indicate that the original excavation at the toe of the landslide decreased stability of the landslide by 5 to 7 percent. This excavation caused the landslide to move at a rate of approximately 15 inches per year. The dewatering tunnels slowed the rate of movement to approximately ½ inch per year, which suggests that the marginal stability of the pre-1893 landslide was restored. From this, one can conclude that the dewatering tunnels improved landslide stability approximately 5 to 7 percent. In our opinion, improving the stability an additional 5 to 7 percent would be a reasonable and realistic goal for mitigating the landslide. This would result in a target FS of 1.05 to 1.07. Another way to look at this level of improvement is that the target FS of 1.05 to 1.07 provides roughly twice the amount of stabilizing force compared to what originally caused the reactivation i.e. the 1893 excavation reduced stability 5 to 7 percent and the combined improvements of dewatering tunnels and proposed current remediation would increase the stability 10 to 14 percent. This rational for developing a target FS is termed the "original profile analysis" (Cornforth 2005). Landslide Technology has used this approach to successfully stabilize several medium to large landslides.

The results of the original profile analyses are summarized on Figures 6 and 10. For the cross-section through Reservoir 3, stability analyses indicate that the 1893 excavation decreased landslide stability by 4.7%. As previously stated, the dewatering tunnels brought the FS back to roughly 1.0. Applying an additional horizontal force of 140 kips per lineal foot to the toe of the landslide raises the FS by 4.8% to approximately 1.05. For the cross-section through Reservoir 4, the 1893 excavation decreased the FS by approximately 6.7%, and the tunnels brought the FS back near 1.0 An additional horizontal force of 190 kips per foot applied to the toe of the landslide raises the FS by 6.6% to approximately 1.07.

The analyses indicate that it is important for the drainage tunnels to continue functioning properly since they will provide approximately half the stabilizing force for the target FS.

Mitigation Modeling. Schematic cross sections of the mitigation measures are shown in Figures 6 through 12. Groundwater levels are assumed to remain unchanged except locally where new underdrains would be installed. For these cases, the groundwater was assumed to follow the base of the underdrain. Shear strengths of the materials were obtained from the back calculated model.

The compressible inclusion was modeled as a free face. In reality, the compressible inclusion will provide a small amount of resistance. The amount of the resistance will depend on the compressive strength of the EPS, which will be determined during final design if Option 1 is selected.

Ground anchors were modeled using the internal tieback model within SLOPE/W. To simplify the analysis input, the entire tieback group was modeled as a single tieback declined at 20 degrees below horizontal. Shear resistance of tieback tendons crossing the shear zone was neglected.

Shear piles were modeled with a horizontal point load at the shear zone. The portion of the landslide shear zone downslope of the shear piles was assumed to continue providing shear resistance to the landslide.

5.11 Seismic Deformation Analyses

There are no widely-accepted published procedures for estimating the displacements of large landslides with low static FS during seismic events. The Newmark (1965) analysis technique is a theoretical solution that treats the landslide mass as a rigid body and assumes movement occurs whenever the FS drops below 1.0. The calculated FS varies as the acceleration varies during the earthquake time history. Case histories of translational slides subjected to earthquake motions indicate that Newmark-type analyses predict larger movements than actually occur. Potential reasons for the over prediction include the fact that the rigid body assumption may not be valid for large landslides and that clay soil generally has a higher shear strength during rapid loading than during slow, monotonic loading.

Kulhawy and Mayne (1990) studied the influence of loading rate on the shear strength of clay soils, and concluded that shear strength increases by approximately 10% for each order of magnitude increase in loading rate. This study indicates that the shear strength of clay soils could be 70 to 85% higher during seismic shaking than during static loading. Newmark analyses completed using increased shear strengths predicted displacements that are much closer to those measured in documented case histories. For the current study, Newmark analyses were completed to estimate seismic displacements for seismic events with different return periods. For these analyses, the shear strength along the base of the landslide was increased 70% above the back calculated value. The pseudo-static coefficient was calculated for a FS of 1.0. The pseudo-static coefficient required to achieve a calculated FS of 1.0 is termed the yield acceleration, or k_{yield}. Figures 6 through 12 tabulate the yield acceleration for the landslide in the unmitigated and mitigated conditions. The figures also show the estimated displacements for ground motions with 475 and 2,475-year return periods.

The displacement estimates shown in Figures 6 through 12 indicate that the proposed mitigations do not substantially raise the yield acceleration from the unmitigated case. As such, the displacement estimates for the mitigated and unmitigated cases are similar, indicating that landslide mitigation would not significantly decrease potential seismic displacements. Displacement estimates for the 475-year ground motion are generally less than ½-inch for all mitigation measures. Displacement estimates for the 2,475-year ground motion are generally less than 3½ inches.

Based on inclinometer measurements and descriptions in Clarke (1904), the thickness of the shear zone at Washington Park varies from less than 6 inches to 2 feet. Any structural elements (i.e. ground anchors or shear piles) that span the shear zone would be subjected to shear displacements over the thickness of the shear zone. In our opinion, displacements on the order of 3 inches along the shear zone could damage a conventional ground anchor. If ground anchors are damaged by landslide movements, the landslide could resume moving at a slow creep rate. Three inches of shear displacement is close to the acceptable limit for shear piles, and will require detailed study and structural analysis during final design if selected as the preferred mitigation option.

Design Earthquake. We understand that PWB currently designs its facilities for ground motions with a 475-year return period. For critical infrastructure, the International Building Code (IBC) requires a minimum design ground motion of 475-years, but larger ground motions can be used for design at the Owner's option.

5.12 Implications of Geologic Landslide Interpretation

The Oregon Department of Geology and Minerals (DOGAMI) recently published maps showing the limits of historically active and interpreted ancient landslides for the Portland West Hills (Oregon Department of Geology and Minerals 2010). These maps are based on geologic interpretation of topographical features identified using recent LIDAR data. The Washington Park Landslide is mapped as an active landslide within a much larger ancient landslide. Based on a brief review of the maps, it is our opinion that there are other possible geologic interpretations for the area. Even if the "mega-slide" interpretation is correct, we are not aware of any movement of the larger landslide mass.

There are many structures that have been successfully built on large, dormant landslides (e.g. Bull Run Dam No. 2 and Bonneville Dam). If the PWB wants a better understanding of the mega-slide, two questions must be answered. The first question is whether the large landslide and the Washington Park landslide move at the same rate. The second question is whether the large landslide and the Washington Park landslide have coincident shear zones in the area of proposed mitigation measures. It is possible that the Washington Park landslide is an active lobe within the much larger mega-slide mass. This situation typically occurs when shear zones are not coincident. In this scenario, mitigation measures could be implemented for the smaller, active lobe provided they do not cross the shear zone of the larger landslide mass. If the Washington Park landslide and the mega-slide have coincident shear zones, mitigation measures for the Washington Park landslide mass may be overstressed by the larger landslide mass (provided it is active). There are various approaches to evaluate these issues if PWB elects to do so. Level of effort for a study could range from an office review of the LIDAR data to installing deep exploratory borings with instruments.