APPENDIX M: REMEDIATION OPTIONS (LANDSLIDE TECHNOLOGY, 2004)

LANDSLIDE STABILITY EVALUATION

A slope stability evaluation of the landslide at the drilling transect for boreholes LT-1, LT-2, and LT-3 was performed by Landslide Technology (Landslide Technology, 2004) using data available in 2002 and spring of 2003, including (1) borehole data, (2) depth of sliding and groundwater data from instrumentation, (3) geologic reconnaissance of the site, and (4) topographic map. The results of the stability analysis were used in evaluating potential slide treatment options, which are discussed in the section below entitled Remediation Option Analysis. Samuel R. Christie and Dr. Stephen E. Dickenson of Oregon State University reexamined the stability analysis of Landslide Technology (2004); their results are summarized in Appendix N and generally agree with the Landslide Technology analysis for the cross section through the boreholes. They obtained similar results for cross sections north and south of the boreholes.

The stability and remediation analyses from Landslide Technology (2004) are for convenience of the reader reproduced below unchanged from the original report, except for a quotation from an unpublished letter from Landslide Technology in response to review comments. The quotation is in regard to the effect on slide stability of water-filled fissures or cracks in the landslide.

Back Analysis

The stability analyses (Landslide Technology, 2004) were performed on cross section A-A', Figure M1. This section was selected because it is nearly parallel to the direction of slide movement and passes through the

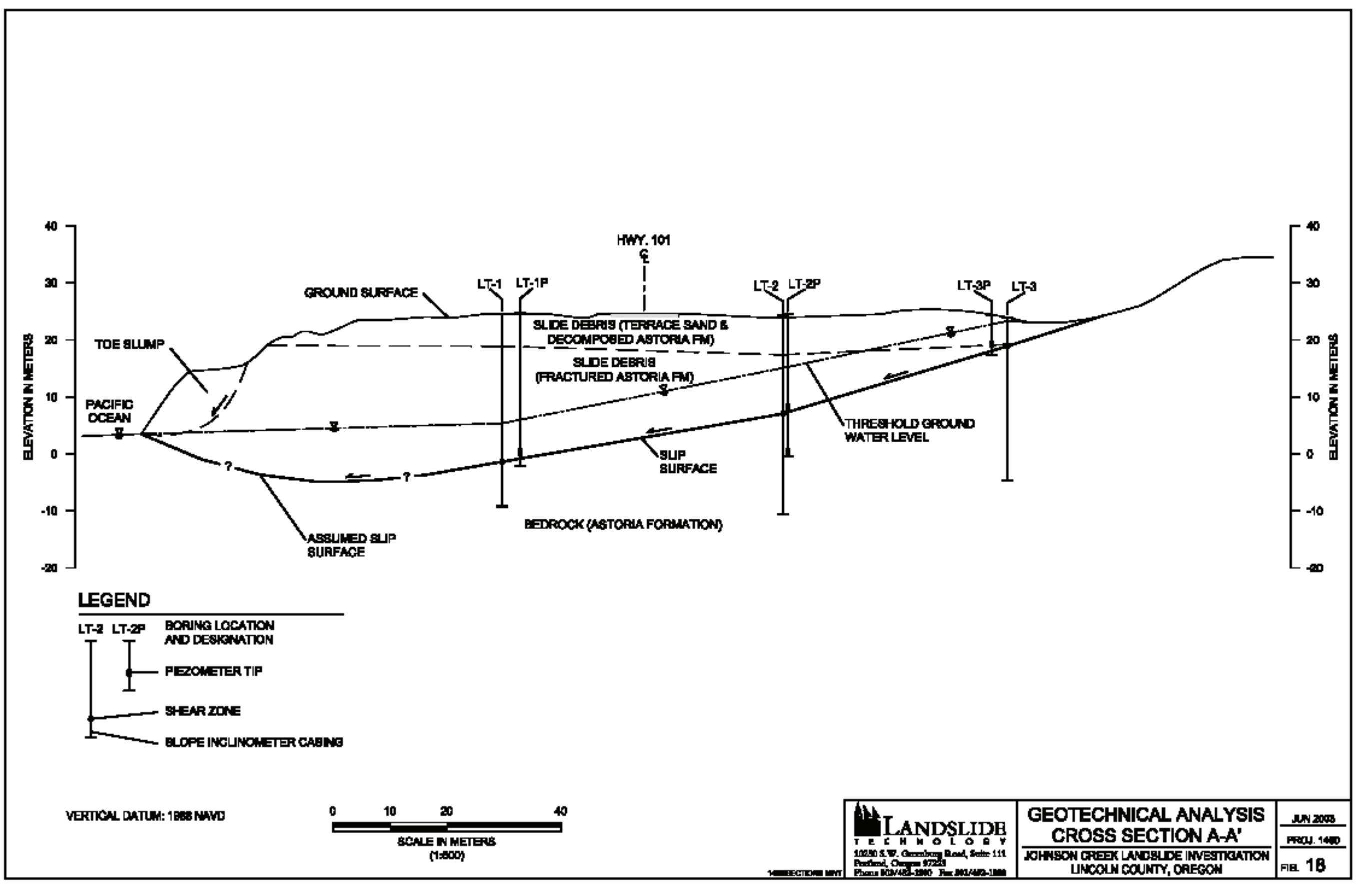

Figure M1. Generalized cross section used by Landslide Technology (2004) for stability analysis. Note that the locations of LT-3 and LT-3p are reversed from actual locations. This minor error should not materially affect the analysis. Location of the cross section is essentially the same as A-A' in Figure 6 of the main text.

three sets of instrumented borings. Analyses were performed using Spencer's method in the computer program XSTABL. Soil parameters used for this study are discussed in more detail in the following sections.

The analyses were performed by back-calculating the required strength (angle of shearing resistance, ϕ') along the shear zone for incipient failure conditions (i.e., for a factor of safety equal to 1.0). The improvements to the factor of safety (FOS) were then checked for various treatment options using the back-calculated ϕ_r'.

Shear zone. The location of the shear zone is estimated based on the known depth of movement in inclinometers LT-1, LT-2, and LT-3, the location of cracks observed upslope from the instrumentation, interpreted topography, and observations from the test pit at the slide toe. The analyzed slip surface is shown in Figure M1.

Groundwater levels. Groundwater levels used in the back analysis stability evaluation are based on piezometer measurements when a threshold level of 10.0 m (32.8 ft) of head on the slide plane was reached in LT-2p. The depth of the groundwater measured below the ground surface at this time for LT-1p, LT-2p, and LT-3p was 19.2 m (elev. 5.4 m), 8.6 m (elev. 15.7 m) and 0.7 m (elev. 23.3 m), respectively. This groundwater level was kept constant throughout the back analysis and is shown in Figure M1.

Material parameters. Strength and density parameters of the soil and rock used in the analyses were estimated based on moisture content, material classification, and our experience with similar materials. Residual ring shear testing of the Astoria Formation material found in the shear zone resulted in an effective residual friction angle of $\phi_r' = 13.1$ degrees. The strength and density parameters of the soil and rock used in the analysis are summarized in Table M1.

Table M1. Summary of material strength and density parameters.

Material	Unit Weight kN/m^3 (pcf)	Cohesion Intercept, c′ Pa (psf)	Angle of Shearing Resistance, ϕ' (degrees)
Terrace sand and decomposed Astoria Formation	18.1 (115)	0	32
Astoria Formation	21.2 (135)	0	6.5*
Rock fill	18.1 (115)	0	42

*Back calculated value from the geologic cross section shown in Figure M1.

Analysis results. The back-calculated residual strength (ϕ_r') value for the slip surface analyzed in cross section A-A' (Figure M1) was determined to be 6.5 degrees. This single digit value is comparable with similar slides in the Astoria Formation and other large translational landslides in tuffaceous sediments and decomposed volcanic rocks, all of which have been investigated by Landslide Technology. The difference between the back analyzed ϕ' value and the value obtained from the ring shear testing (13.1 degrees) may be attributed to the fact that the sample tested may not be representative of the entire failure surface. The back-calculated ϕ_r' value is an average value for the model.

Sensitivity Analysis

A parametric investigation was performed to evaluate the sensitivity of landslide stability to the following parameters: precipitation, groundwater levels, erosion, and beach sand level. Specific parameters were varied as discussed in the following sections.

Precipitation and groundwater. An evaluation of the sensitivity of slide movement to precipitation and groundwater level was performed. As discussed in section 5.3 [of Landslide Technology (2004)], a rainfall event which measures 55 to 60 mm of rainfall in a 24-hr period is likely to trigger landslide movement. Peak rainfall events cause groundwater to rise above threshold levels, further destabilizing the landslide. With the available piezometer data, groundwater levels for a "severe storm" were modeled by raising the highest measured levels in piezometers LT-1p, LT-2p, and LT-3p by 1.5 m (but not above the ground surface). Groundwater levels used for the theoretical "severe storm" analysis are elevation 9.0 m, 19.0 m, and 24.1 m at piezometer locations LT-1p, LT-2p, and LT-3p, respectively. The results indicate that a rise in groundwater level of 1.5 m above the back-analyzed level would decrease the FOS of the slide mass by seven percent.

During the winter months groundwater levels appear to stay at reasonably stable levels, except during moderate to severe rainfall events. These "normal winter" levels were measured at average elevations of 5.0 m, 14.6 m, and 21.4 m in piezometers LT-1p, LT-2p, and LT-3p, respectively. By varying only the groundwater level in the slide the results of the analysis indicate that decreasing the groundwater level to the theoretical "normal winter" results in an increase in the FOS of the slide on the order of two percent higher than the back analysis.

Water-filled cracks. Landslide Technology (2004) did not discuss the effect of water-filled cracks, but reviewers of the 2004 report did ask about this issue. Here is the response from Landslide Technology in their September 4, 2003, unpublished letter:

"Regarding the potential effect of water-filled tension cracks, Boring LT-3 is located near the head of the slide and any cracks downhill from LT-3, with or without water, would be modeled as an internal force in the stability analyses of the overall landslide and would have very minor effect on the friction angle (only the added weight of water in the tension crack). A water-filled crack uphill from LT-3 might have an effect on the back-calculated friction angle, and we tested this to see any difference. We placed an 18-ft high water filled crack east of LT-3, and the factor of safety against sliding increased slightly, and the resulting phi residual dropped from 6.5 to 6.45 degrees. We interpret that this difference is due to the removal of a small portion of the landslide's driving wedge."

Erosion and beach sand movement. To evaluate the effect of ocean surf on the stability of the slide, both erosion of the cliff face at the toe of the slide and the seasonal deposition and removal of sand due to surf action were analyzed.

To evaluate the sensitivity of the slide to erosion of the bluff at the beach, stability analyses were performed and compared to the back-analysis results. The models were developed by offsetting the entire face of the bluff (up to an approximate elevation of 14.6 m) 0.3 m (1 ft), 1.5 m (5 ft), and 3.0 m (10 ft) to the east, respectively (Figure M2). To isolate the effect of the erosion, the geometry of the shear zone at the toe remained unchanged from the back analysis. To keep the groundwater conditions constant through the analyses, groundwater levels for the 3.0-m erosion study were used. The only difference between this groundwater level and that used in the back analysis is a slight lowering of the water level west of LT-1P due to a change in the inflection point of the groundwater surface at the beach as a result of the changing location of the cliff face.

An additional study was performed to isolate and evaluate the effect of seasonal deposition and removal of sand from the beach relative to the stability of the slide. The model for this analysis consisted of adding approximately one meter of sand to the beach area, which isolated the effect of the sand by limiting variations to the model (i.e., the failure surface). For this analysis the groundwater level remained unchanged from the back analysis model. The geometry of the shear zone was modified only by extending the toe outward to the new ground surface.

Summary of sensitivity analysis. A parametric study has been performed to evaluate the sensitivity of the slide to three major parameters: (1) precipitation and groundwater, (2) erosion, and (3) the seasonal deposition and removal of sand on the beach. The back analysis model was used as the reference, and for each

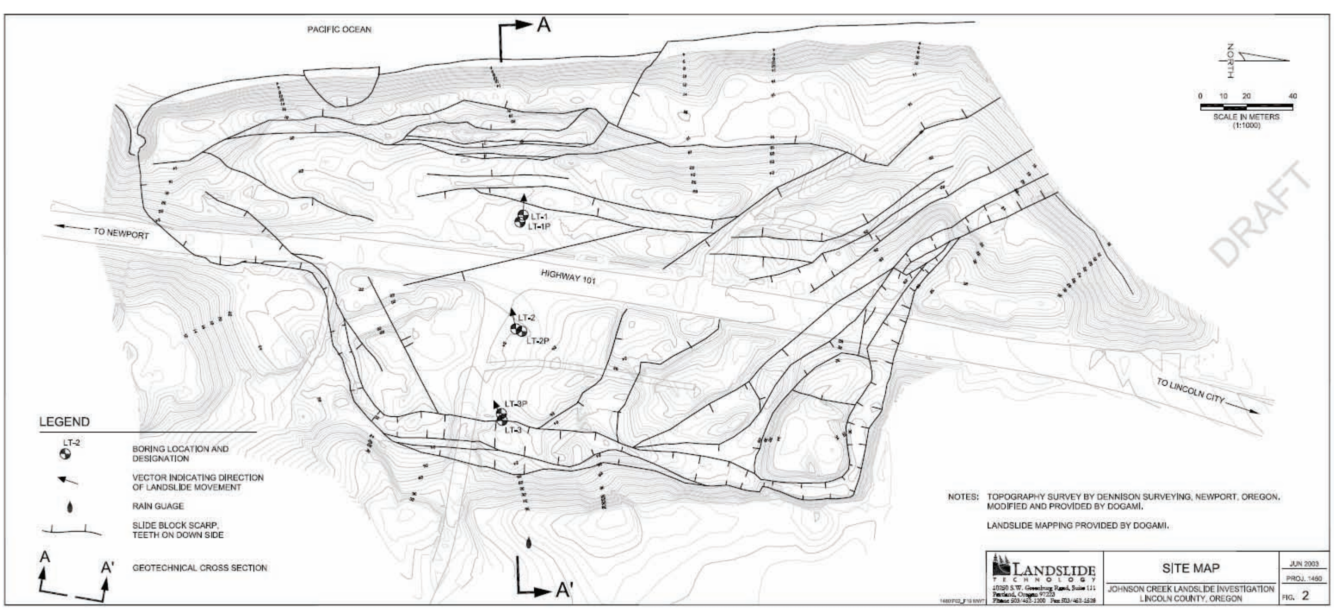

Figure M2. Site map. Slide block boundaries (black lines) are from Landslide Technology (2004).

parameter incremental changes were made to determine the resulting percent change in FOS. A summary of the analyses is provided in Table M2.

Table M2. Summary of sensitivity analyses.

Parameter	Change in Factor of Safety from Back-Analysis (− = Decrease / + = Increase)
Groundwater	
"normal" 2003 winter level	+2.0 %
"severe storm"	−7.2 %
Erosion of cliff face	
0.5 m (1 ft) of erosion	– 0.8 %
1.5 m (5 ft) of erosion	– 3.6 %
3.0 m (10 ft) of erosion	– 6.8 %
Seasonal deposition/removal of sand	
1.0 m (3 ft) removal	– 0.3 %
1.0 m (3 ft) deposition	+ 0.3 %

CONCEPTUAL REMEDIATION OPTIONS

Landslide Technology (2004) evaluated several remedial options to increase landslide stability and minimize ground movement affecting the roadway; for convenience, their analysis is reproduced below. These options include (1) unloading near the headscarp, (2) toe buttress, (3) horizontal drains, (4) tied-back shear pile wall, and (5) maintenance. Each remediation option was designed to improve the factor of safety by at least 10 percent (FOS=1.10) during the "severe storm" event.

A brief discussion of each option is presented, along with advantages and disadvantages. The cost estimate for each option is based on general and specialized construction costs, plus a 25 percent contingency to provide for the uncertainties of conceptual level design. The cost estimates do not include costs for environmental issues (e.g., permitting), final design, preparation of plans and specifications, contractor procurement, or construction control.

The northern and southern limits were estimated based on topographic interpretations and headscarp cracks observed in the highway and along the approximate northern and southern limits of the slide area. For the purpose of estimating costs of the treatment options, the slide is assumed to be 360 m (1180 ft) north-south along the beach.

Option 1 – Unload Upper Slide

This option entails unloading the head of the slide by excavating material east of the highway and installing two French drains along the east side of the excavation. The excavation would extend approximately 160 m (525 ft) north from the access road crossing the headscarp. The approximate limits of the excavation are shown in Figure M3. The elevation of the excavation floor would be approximately 18 m (59 ft) (Figure M4).

French drains would minimize ponding during and after construction. A connector drain would be constructed to tie the two drains together at the southern end of the excavation, and a drainline would outlet into the drainage swale south of the slide and east of the highway, as shown in Figure M3.

This option provides a theoretical improvement in the factor of safety of 20 percent using back-analyzed groundwater levels, and a 12 percent improvement using the "severe storm" event.

Advantages:
- Relatively low construction cost
- No environmental impact to the beach area
- Good access for construction
- Simple construction techniques
- Minor long-term maintenance required
- Highway alignment not affected

Disadvantages:
- Provides no protection against continued toe erosion, which could eventually reactivate slide movement even with unloading implemented
- Short-term environmental impacts
- Requires disposal of excavated material
- Relocation of utilities
- Potential ponding in the excavation area

Conceptual Construction Cost: $0.9 million

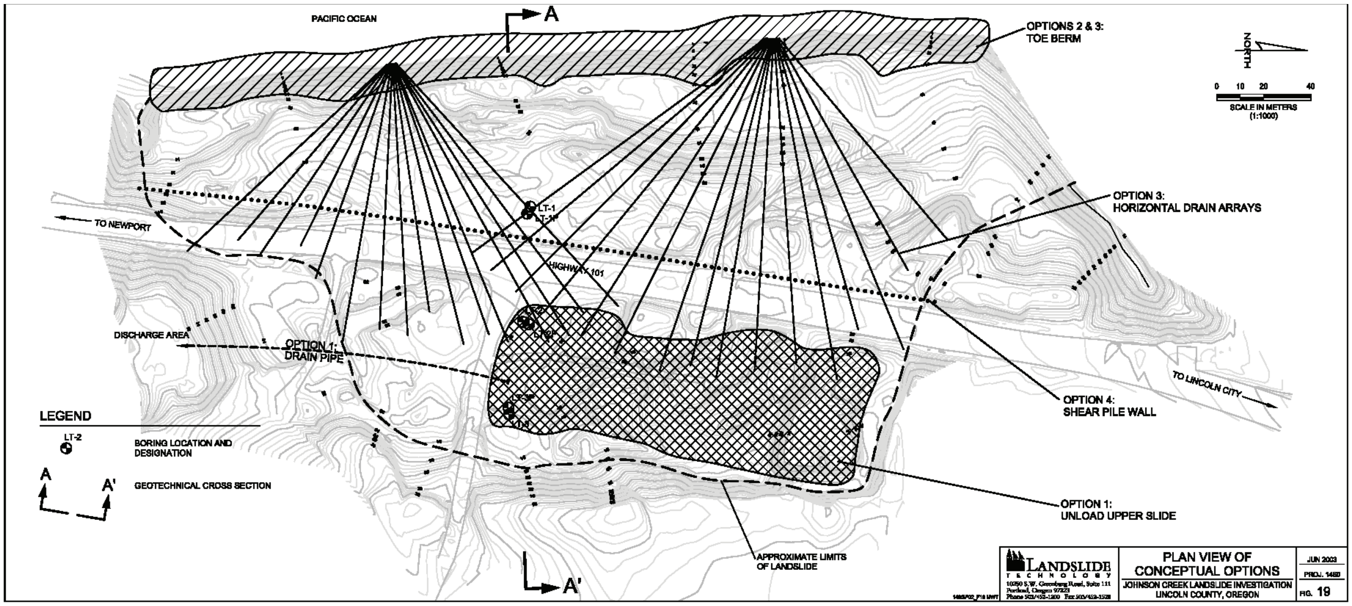

Figure M3. All remediation alternatives summarized by Landslide Technology (2004).

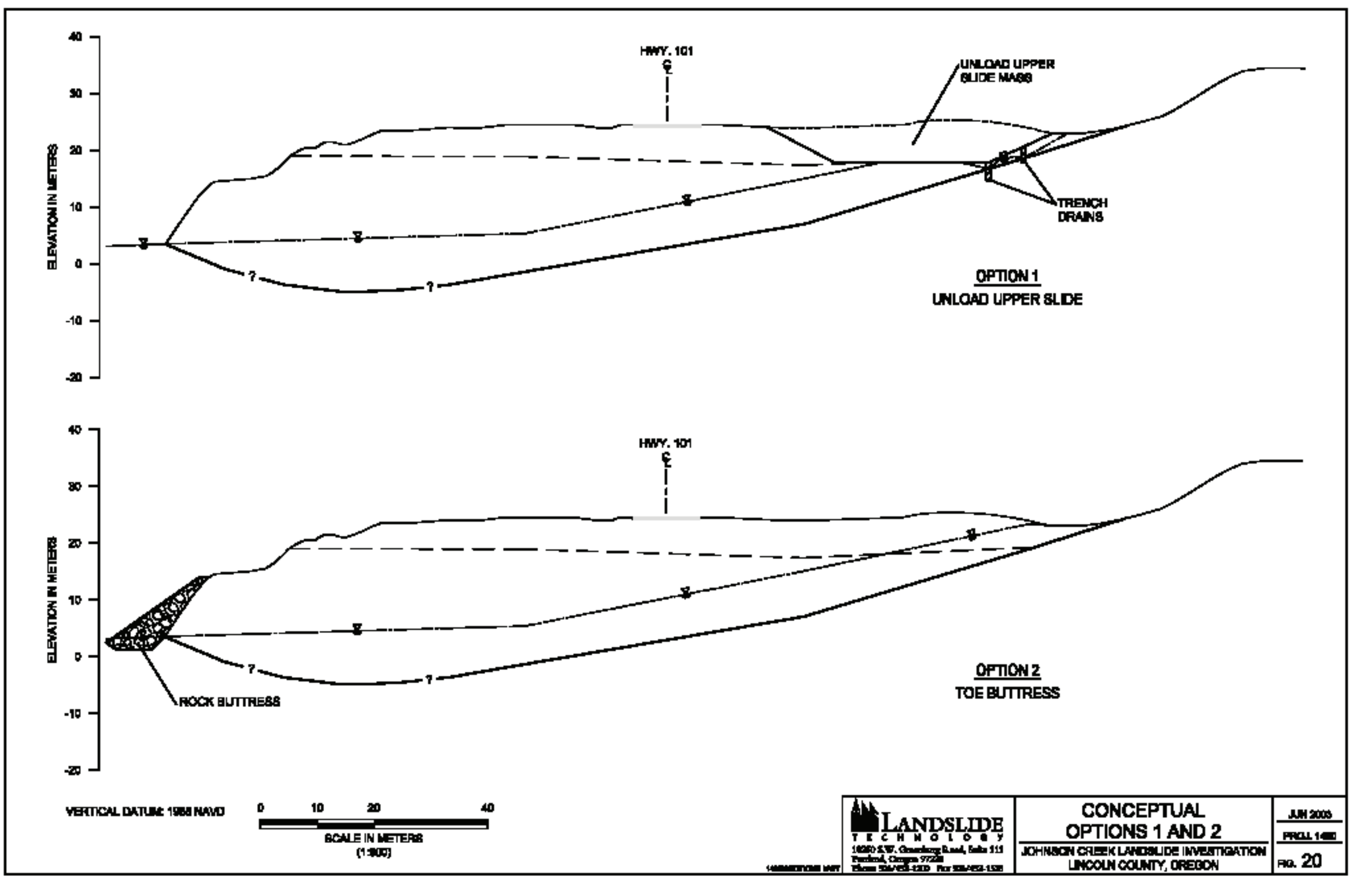

Figure M4. Remediation by unloading the head of the slide and buttressing the slide (taken from Landslide Technology, 2004).

Option 2 – Toe Buttress

This option would involve building a buttress on the beach along the toe of the slide as shown in Figures M3 and M4. The buttress would consist of rockfill with a key extending approximately 2 m (6 ft) below the beach, and riprap facing for erosion protection. The buttress would be 11 m high (36 ft), extend approximately 8 m (26 ft) onto the beach from the bluff, and have a 1V:1.5H slope face with the level top extending approximately 2 m (6 ft) out from the existing slope face.

Construction would consist of excavating the key trench in sections, placing a geotextile fabric and then rockfill materials in lifts. The construction of the key trench would occur in 15-m (50-ft) sections to prevent slide instability during construction. Once the length of key was fully constructed, rockfill and riprap would be placed in lifts along the length of the slide to the finished height.

This option provides a theoretical improvement in the factor of safety of 19 percent using back-analyzed groundwater levels, and a 12 percent improvement using the "severe storm" event.

Advantages:

- High degree of confidence in stability improvement
- Relatively low construction cost
- Limits rate of bluff erosion
- Simple construction techniques
- Minimal long-term maintenance required
- Highway alignment not affected

Disadvantages:

- High environmental impact (construction on beach)
- Limited access to site

Conceptual Construction Cost: $1.1 million

Option 3 – Horizontal Drains

This option would consist of installing horizontal drains through the slide mass from the toe of the slope (Figures M3 and M5). The drains would consist of slotted PVC pipe installed laterally into the slope face with a specialized drill rig. The horizontal drains would attempt to reduce the groundwater level during normal conditions and prevent the buildup of groundwater pressure during extreme rainstorm events.

Based on the stability analyses, improvement in the FOS from horizontal drains is about 1% during the "severe storm" event. Also, the rotational failures at the toe of the larger slide are likely to shear the horizontal drains rendering them less effective or inoperable, which could also worsen the stability of the rotational failures.

Other options would be necessary to provide additional stability to the overall slide, such as a toe buttress. A riprap toe buttress could minimize erosion of the bluff and could provide stability to the rotational toe failures.

Based on the 1% improvement in FOS during the "severe storm" and the potential for rotational failures at the slide toe, this option is not recommended for the Johnson Creek landslide. Nevertheless, to provide comparison to other options, a conceptual design might include two drain arrays as shown in Figure M3. The cost estimate includes a total of 36 horizontal drains in two arrays for a total constructed length of 4,270 m (14,000 ft).

Advantages:

- Relatively low construction cost
- Simple construction techniques
- Highway alignment not affected
- Low long-term environmental impact
- Minor long-term maintenance

Disadvantages:

- Stability improvement is low
- Limited design life of the drains with erosion and slide movement
- Limited access to site

Conceptual Construction Cost: $0.5 million

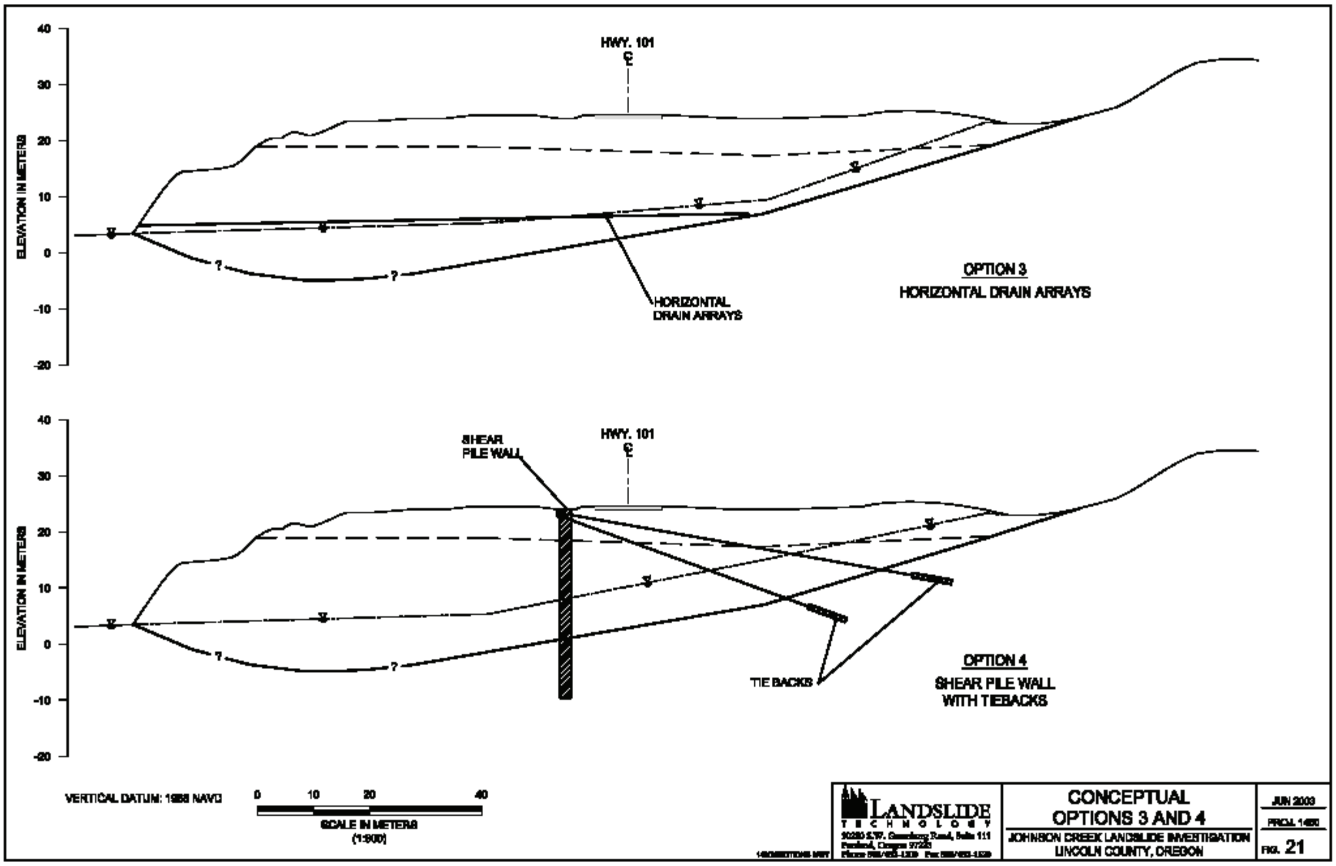

Figure M5. Remediation by horizontal drains and shear pile wall with tiebacks (taken from Landslide Technology, 2004).

Option 4 – Tied-Back Shear Pile Wall

This option consists of constructing a row of large diameter, heavily reinforced concrete piles with tieback anchors to resist slide movement, installed just west of the highway as shown in Figures M3 and M5. Conceptual design consists of a 342-m-long (1122 ft) wall of 1.4-m (4 ft) diameter and 36 m (120 ft) deep piles with a spacing of 3.0 m (10 ft). A continuous, structural capping beam would be constructed at the top of the shear piles. Two rows of tiebacks would be installed through the capping beam (Figure M5). The tiebacks would decrease pile deflection and movements and would result in less passive contact pressures in the sandstone below the shear zone. The wall and anchors could be covered and the site restored to a natural condition. This conceptual design provides a factor of safety of 1.3 during the "severe storm" event.

Advantages:

- High degree of confidence in stability improvement
- Low environmental impact (no construction on beach)
- Minimal long-term maintenance
- Highway alignment not affected

Disadvantages:

- Expensive
- Specialized construction technique
- Construction could impact highway traffic
- Lower slide area may continue to move due to continued bluff erosion

Conceptual Construction Cost: $11 to 14 million

Option 5 – Road Maintenance

This option would consist of continued maintenance of the road. This option requires that the slide area continue to be inspected on a weekly basis and on a daily basis during large storm events, and then quickly repaired when significant movements occur. ODOT records indicate that yearly costs for maintenance have been approximately $15,000 per year prior to the late 1970s, and $20,000 per year more recently.

Advantages:

- Inexpensive
- Low environmental impact

Disadvantages

- No effective stabilization
- Landslide will continue to move
- Continued risk to property and life safety
- Requires continual inspection and emergency repair as necessary

Cost: $20,000 a year for basic maintenance (~$400,000 for 20 years)

Summary of Remediation Options

Remediation options that were evaluated for Johnson Creek landslide include unloading, buttressing, draining, a tied-back shear pile wall, and maintenance. A summary of the construction options is provided in Table M3.

Unloading, buttressing, and a tied-back shear pile wall are effective methods to remediate this landslide. Considering the large size of this landslide, unloading and buttressing are relatively low cost options. With stabilization and cost consideration, buttressing would be a preferential option; however, it has a significant environmental impact. A shear pile wall is extremely expensive primarily due to the depth of sliding. Draining groundwater from the landslide through horizontal drains would be ineffective. Groundwater levels within the slide mass are relatively low, and high groundwater levels following precipitation events rapidly drop or naturally drain from the fractured slide mass. Based on the conceptual costs for the construction of these remediation options, annual maintenance becomes a reasonable option.

Table M3. Remediation option comparison.

	Remediation Option				
	1 Unload Upper Slide	**2 Toe Buttress**	**3 Horizontal Drains**	**4 Tied-Back Shear Pile Wall**	**5 Road Maintainance**
Effectiveness	moderate	high	low	high	low
Constructibility	good	good	moderate	difficult	not applicable
Engineering	simple	moderate	moderate	difficult	simple
Environmental long-term impact	low	high	low	low	low
Maintenance long-term	low	low	moderate	low	high
Construction costs ($ million)	0.9	1.1	0.5	11–14	0.4 (20 yrs)

REFERENCE

Landslide Technology, 2004, Geotechnical investigation, Johnson Creek landslide, Lincoln County, Oregon: Oregon Department of Geology and Mineral Industries Open-File Report O-04-05, 115 p.