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Repair of Failing Spirit Lake Outlet Tunnel at Mount St. Helens

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ABSTRACT

The 18 May 1980 eruption of Mount St. Helens resulted in one of the largest debris avalanches recorded in history. The debris avalanche blocked the natural outlet of Spirit Lake. To prevent an uncontrolled and catastrophic lake break-out, the U.S. Army Corps of Engineers (USACE) constructed the Spirit Lake Outlet Tunnel from 1984 to 1985. Because Spirit Lake is located in the Mount St. Helens National Volcanic Monument, the project was transferred to the U.S. Forest Service (USFS) for ownership and management. During original tunnel construction, the most difficulty occurred within the 90-m-wide Julie and Kathy L. shear zone complex. In 1996, tunnel walls within this complex experienced significant convergence, which required major repair along a 30 m segment. During inspections in 2014 and 2015, a 10 m segment at the upstream end of the complex, which had experienced slow heave in the past, was observed to have experienced an increase in heave of approximately 0.6 m, which decreased the hydraulic capacity of the tunnel below acceptable limits. The USACE, in accordance with and with funding from the USFS, designed a repair based on the rib set-shotcrete support system that was used for the 1996 repair. In addition to removing and arresting the heave, the 2015 repair was sized to be compatible with a future potential rehabilitation involving stabilizing and re-grading the invert profile of the Julie and Kathy L. shear zone complex. The 2015 repair construction contract was awarded in September 2015 for \$3 million.

Keywords: Tunnel, heave, steel ribs, shotcrete, Mount St. Helens, Spirit Lake.

1. BACKGROUND

Mount St. Helens is located in southwest Washington, about 72 km northeast of the Portland-Vancouver metropolitan area (Figure 1) and is part of the Cascade Range that runs from British Columbia through Washington, Oregon, and into northern California. The range is made up of active volcanoes comprising a part of the Pacific “Ring of Fire”. Mount St. Helens erupted on May 18, 1980, resulting in a catastrophic landslide that released millions of tons of sediment into the upper North Toutle River drainage basin. A massive debris avalanche completely filled Spirit Lake, located about 6 km north of the mountain, blocking the lake’s outlet to the North Fork Toutle River and raising the lake level some 60 m (200 ft).

After the eruption, the U.S. Army Corps of Engineers (USACE) conducted investigations to determine the stability of the debris dam that blocked Spirit Lake. The debris dam consists mainly of debris avalanche deposit material, ranging from silt to boulder in size, overlain by rock fragments from the blast deposit. The uppermost material consists of a fine-grained ash and pumice deposit that is easily erodible.

In 1982, a governmental task force was formed to evaluate the hazard posed by the blockage of Spirit Lake. The group determined that the debris blockage could not safely pond water above elevation 1,059 m (3,475 ft) due to the character of the material and that Spirit Lake would reach elevation 1,059 m by March 1983. The lake could not be allowed to reach that elevation due to the potential for failure of the debris blockage from piping through the uppermost ash material and the resulting catastrophic flooding, widespread damage to downstream communities, and interruption of Columbia River navigation. Swift and Kresch (1983) provided inundation maps based on a hypothetical failure of the debris blockage.

An emergency pumping station was constructed to pump water from Spirit Lake over the debris blockage and into the North Fork Toutle River as an interim measure to maintain a safe lake level while a long-term solution was studied and determined. The pumping facility consisted of 20 pumps mounted on a barge near the shore of Spirit Lake. Water was pumped from the lake through 1,050 m (3,450 ft) of 1.5-m (5-ft) diameter steel pipe across the debris blockage to a stilling basin and, from there, to the North Fork Toutle River.

Once the lake level was stabilized through the interim pumping facility, a permanent safe lake level was determined (1,055 m or 3,460 ft), and alternatives for a permanent lake outlet were evaluated. Alternatives included 1) a buried conduit, 2) an open channel, 3) a tunnel, and 4) a permanent pumping facility. Various alignments for each alternative were evaluated, along with criteria related to location within the National Volcanic Monument, constructability, and cost. Because some of the alignments crossed the potentially unstable debris blockage, long-term stability, downstream impacts from erosion and sediment transport, water quality impacts, and effect on stability of the Spirit Lake and Coldwater Lake debris blockages were other important considerations. Each alternative was also evaluated for the ability to withstand impacts from future volcanic or seismic events due to the proximity of the mountain. Based on these considerations, a straight tunnel extending about 2,590 m (8,500 ft) from the west side of Spirit Lake through Harry's Ridge to South Coldwater Creek was selected as the preferred alternative. The Spirit Lake Outlet Tunnel was constructed by the USACE and turned over to the U.S. Forest Service as owners in 1985. Sager and Chambers (1986) and Sager and Budai (1989) describe original tunnel construction.

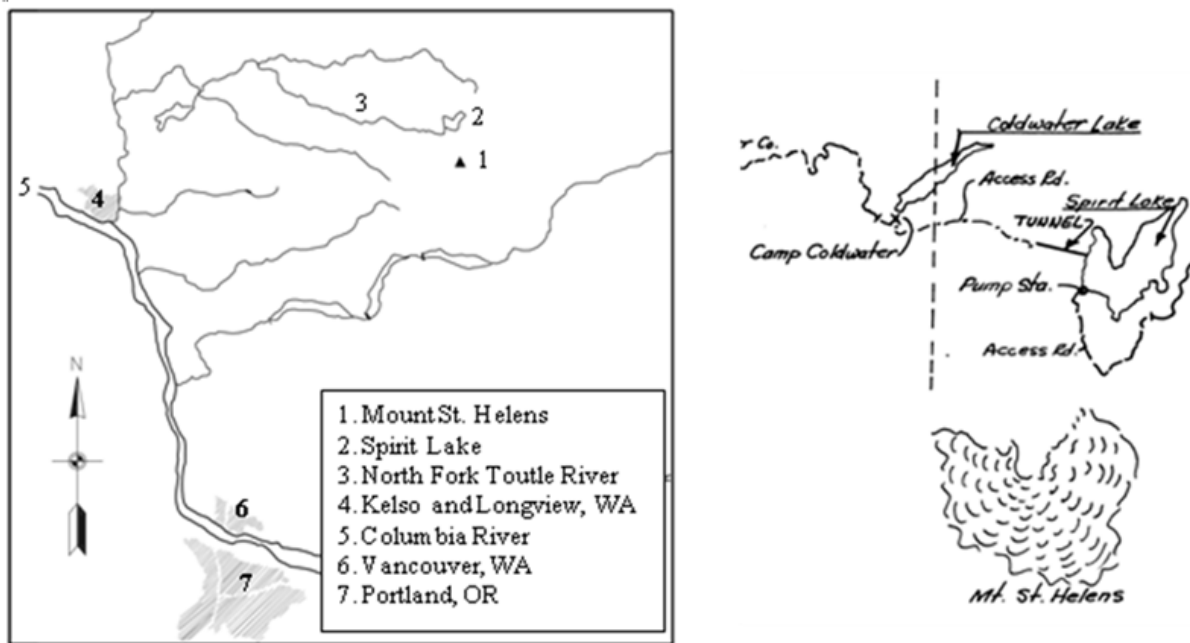


Figure 1. Map of region, left, and Spirit Lake vicinity, right. Spirit Lake map clipped from figure in USACE (1987)

2. TUNNEL DESIGN AND CONSTRUCTION

2.1. Hydrology and Hydraulic Design

The Design Memorandum for the project (USACE 1984) provides a full description of the hydrology and hydraulic design. Select relevant information is provided below.

Spirit Lake is the largest alpine lake affected by the May 1980 eruption of Mount St. Helens. Originally, the lake was approximately 56 m (185 ft) deep with a surface area of 5 km² (1,260 acres). The pre-eruption lake surface

fluctuated around elevation 975 m (3,200 ft) NGVD. During the eruption, Spirit Lake was displaced by debris from Mount St. Helens. The displaced lake came to rest at a pool elevation of 1,038 m (3,404 ft) NGVD, approximately 62 m (204 ft) higher than the original water surface. The minimum crest elevation of the debris dam forming Spirit Lake is approximately 1,079 m (3,540 ft) NGVD.

Spirit Lake was not only relocated, but the hydrologic characteristics of the watershed were altered. The pre-eruption 38.6 km² (14.9 mi²) commercial/recreational forest land watershed was changed to a 46.6 km² (18 mi²) basin denuded of vegetation. The increase in drainage is from the north slope of Mount St. Helens. Surface water from the southwest now flows from the rampart of the Mount St. Helens crater through a deeply carved chasm onto a flat area of avalanche debris. The areas to the east, north, and west of the lake are extremely steep and produce rapid runoff that flows directly into the lake.

The climate of the area is predominately a mid-latitude west coast marine type. Summers are comparatively dry and cool, and winters are cloudy, mild, and wet. During the wet season, rainfall is usually of light to moderate intensity and continuous over a period of time, rather than of the heavy short-duration type storms. However, rainfall of heavy intensity can be expected as more intense weather systems move inland. Mean annual rainfall precipitation over the Spirit Lake Basin ranges from 229 cm (90 in.) to over 305 cm (120 in.); the basin average is about 254 cm (100 in.). About half of the annual precipitation occurs between November and February.

Precipitation in the form of snow is a significant hydrologic factor in the Spirit Lake vicinity. The mean annual snowfall amount ranges from 236 cm (93 in.) to 1,250 cm (493 in.) with a basin mean of 719 cm (283 in.). Maximum observed snow depths at the pre-eruption Spirit Lake snow station ranged from 25 cm (10 in.) in early November to 416 cm (164 in.) in mid-to-late March. The snowpack can reach water content densities of 30 to 40 percent prior to the spring melt.

Flood frequency peak discharge data were determined for the Spirit Lake basin using a rainfall/runoff model of the basin based on the HEC-1 computer program. Precipitation-frequency data were determined per Miller et al. (1973). Losses used were minimal rates due to basin topography and lack of ground cover and soil mantle.

A Probable Maximum Flood (PMF) analysis assumed maximum flood-producing elements and conditions arranged in a sequence that will produce the most critical flood runoff. These meteorological and hydrological elements are probable maximum precipitation for the critical storm period, maximum basin snow cover, maximum temperatures during the design storm, minimum surface losses, and a unit hydrograph that will reflect the runoff for a storm the magnitude of the PMF. The peak discharge of the PMF derived for Spirit Lake is 1,220 m³/s (43,000 cfs). The total volume of the PMF, including base flow, is 60 million m³ (49,000 acre-feet).

To size the required outlet structure and tunnel capacity, the PMF was routed through Spirit Lake beginning at an elevation of 1,049 m (3,441 ft) NGVD and was preceded by a large antecedent flood (100-year event). The total volume of the two floods is 69 million m³ (56,000 acre-feet). Peak regulated outflow was 15.6 m³/s (550 cfs), and the maximum lake elevation during the flood event was 1,054 m (3,459 ft) NGVD.

Long-term operating criteria were adopted for hydraulic design guidance as follows: flow depth within the tunnel no greater than three-quarters of the tunnel height, Spirit Lake to be considered normal at pool elevation 1,049 m (3,440 ft) NGVD, the intake gate to remain fully open after initial drawdown, and pool fluctuations should be within 1.5 m (5 ft) above normal during most flood events. The maximum temporary safe lake elevation was determined to be 1,055 m (3,460 ft) NGVD. The project was designed to limit all possible hydrologic raises in the lake level to this elevation.

Hydraulic sizing of the tunnel was dictated by the selection of hydraulic parameters that allow flow within the tunnel to be at atmospheric (open channel) conditions. Concern was raised that pressurization of the tunnel without periodic air relief for negative pressures could result in collapse of the tunnel walls. A maximum flow capacity of 15.6 m³/s (550 cfs) was required for the tunnel and intake to pass flows without raising the lake level more than 1.5 m (5 ft) under normal conditions, or above elevation 1,055 m (3,460 ft) NGVD under PMF conditions. Design computations indicated that a tunnel diameter of 3.3 m (10.8 ft) would provide the required flow capacity while allowing for minor offsets during construction and variations in finished tunnel roughness.

2.2. Geology

The Spirit Lake tunnel is located in the western flank of the Cascade Range, a major north-south trending mountain range of volcanic origin. The tunnel penetrates through older volcanic sequences comprising Harry's Ridge. The geology of the tunnel was mapped in detail at a scale of 1:120 at the time of construction. A generalized geologic profile of the tunnel is shown in Figure 2. This description is taken from USACE (1987). General stratigraphy is Tertiary-age, predominantly volcanic tuffs overlain by predominantly basalt and andesite lava flows. Both units have been intruded by basaltic dikes. The rock sequence has been regionally deformed, sheared, and faulted, and it now dips about 30-40 degrees to the east. Strike of the flows and tuffs is nearly normal to the tunnel centerline.

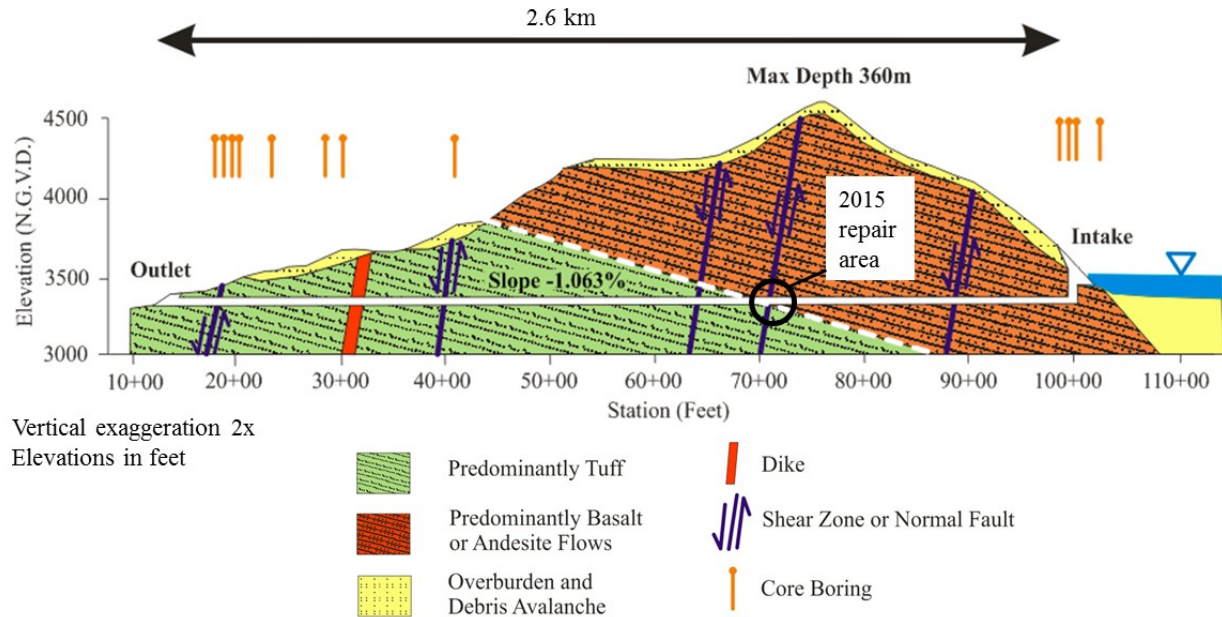


Figure 2. Geologic profile along tunnel (left is west, right is east)

The western two-thirds of the tunnel penetrates a sequence of tuffaceous rock types including fine-grained, lapilli, tuff breccias, localized layers of welded tuff, and minor lava flow sequences (USACE 1987). The tuffs are primarily a light-green, dense, fresh rock with angular, multi-colored fragments in the lapilli beds, with strengths of 34 MPa to 100 MPa (710 – 2,100 ksf). Interbedded lava flows range from less than 30 m (100 ft) to greater than 100 m (330 ft) thick, with strengths between 100 MPa and 250 MPa (2,100 – 5,200 ksf). Flow rock is generally comprised of multiple flows separated with flow breccia. The eastern one-third of the tunnel penetrates a sequence of predominantly basaltic and andesitic lava flows with minor amounts of interbedded volcanic tuffs. The geologic contact between the two units is composed of soft, decomposed tuffs.

Twelve shear zones and faults were encountered along the tunnel. The affected areas varied from 1 m (3 ft) to upwards of 20 m (66 ft). The primary condition of the shear zones is fractured rock in various stages of decomposition and some plastic to low plasticity fines. Within the shear zones are thin seams of clay gouge. Most of these shears are steeply dipping and cross the tunnel nearly normal to the centerline. The most serious shear zones were the Julie and Kathy L. shear zones, where two zones of about 15 and 20 m (49 and 66 ft) were encountered near the geologic contact and maximum ground cover. The shears were in ashy volcanic tuffs decomposed to weak rock and swelling clays. Spalling and squeezing ground occurred, and the grippers on the tunnel boring machine could not grip the rock. In addition, the soft clay caused significant deviation of the vertical alignment. Hand mining was required in some areas so that steel sets could be placed.

No large, sustained groundwater inflows were encountered along the length of the tunnel. Small inflows (19 l/min / 5 gpm or less) were occasionally encountered from rock fractures (USACE 1987).

2.3. Tunnel Construction and Support

Except for the downstream 70 m (225 ft) of the tunnel, which was excavated using drill-and-blast methods, the majority of the tunnel (2,500 m or 8,200 ft) was excavated using a tunnel boring machine (TBM). The TBM excavated the tunnel to a diameter of 3.4 m (11 ft). Three types of support systems were used depending on the rock conditions. The entire tunnel invert was lined with pre-cast or cast-in-place concrete. In “good rock” (55% of tunnel), no treatment was used on the walls and crown. In “fractured rock” (35% of tunnel), the walls and crown were covered with a minimum of 50 mm (2 in.) of steel fiber-reinforced shotcrete. In “sheared rock” (10% of tunnel), a rib set-shotcrete support system was used for the walls and crown (285 degrees of the tunnel circumference).

Rock loads for design were evaluated using the modified Terzaghi system. For the worst rock conditions, characterized by sheared rock and squeezing ground, a rock load of 410 kPa (8,570 psf) was used (USACE 1984). No groundwater pressure was assumed. Where needed, drains were installed 1.5 m (5 ft) into the crown during construction. The original design of the rib set-shotcrete system called for W150 mm x 37 kg/m (W6 in. x 25 lb/ft) rib sets with a steel yield strength of 248 MPa (36 ksi). A contractor proposal was accepted by the USACE to use W100x19 (W4x13) rib sets instead. The standard rib set spacing was to be 1.2 m (4 ft). The spacing could be reduced to as little as 0.3 m (1 ft) if squeezing ground conditions were encountered (USACE 1984).

In general, tunneling progress during construction was good. The problem area, where construction was difficult and slow, was the 90-m-wide (300-ft-wide) Julie and Kathy L. shear zone complex (station 72+50 to 75+50). Before encountering this complex, the tunnel advancement rate had been 1,178 m (5,834 ft) in 11 weeks. The TBM got stuck for two days near the beginning of the complex. It took about 4 weeks to advance the next 72 m (235 ft) (USACE 1987). Rib sets were placed as close as 0.6 m (2 ft) apart in two areas of the complex, including 20 m (66 ft) of sheared ashy tuff adjacent to (downstream of) the geologic contact with basalt. No rib sets were placed upstream of the geologic contact. Some pre-cast concrete invert sections were damaged and removed in the Julie and Kathy L. shear zone complex. Steel struts were installed instead, and the invert was covered with cast-in-place concrete. A significant flow of groundwater occurred into the tunnel when the permeable geologic contact was first encountered. Within one week, the trapped water had drained and only seeps remained.

The Spirit Lake Outlet Tunnel Foundation Report (USACE 1987) included a section describing conditions that could produce future problems. For the tunnel, “Slight possibility of ground adjustments in shear zones, especially the Julie – Kathy L. complex.” The conditions in the Julie and Kathy L. shear zone complex have indeed produced problems requiring repairs.

3. 1996 TUNNEL REPAIR

Routine inspections of the tunnel were conducted annually from the time the tunnel went into operation in 1985. These inspections sometimes resulted in minor patchwork repair contracts, with many of these repairs occurring repeatedly between stations 73+50 and 74+50. In October 1992, the inspection team found significant problems in this area of the tunnel, including large sections of shotcrete that had pulled away from the tunnel walls, uplifted and cracked precast invert segments, and sheared and buckled rib sets. The exposed areas were sealed with concrete to allow for continued operation of the tunnel while funding was secured and contract specifications were prepared to repair the tunnel.

A rib set-shotcrete support system was used for the repair. The repair support system was more robust than the original rib set-shotcrete support system. The loading conditions included the same rock load of 410 kPa (8,570 psf) that was used for the worst rock conditions in the original design, plus an assumed hydrostatic load of 203 kPa (4,243 psf), for a total design load of 613 kPa (12,813 psf). The design beam shape and yield strength were increased to W200x42 (W8x28) and 345 MPa (50 ksi), from the W100x19 (W4x13) and 248 MPa (36 ksi) used for the original rib sets. Thirty-four new rib sets were installed within the 30 m (100 ft) repair area. These were full-circle rib sets, not the 285-degree rib sets and invert struts used for original construction. The original rib sets were spaced at 0.6 m (2 ft) in one part of the area and 1.2 m (4 ft) in the other part. The repair involved excavating midway between the original rib sets to install the new rib sets, then covering them with steel fiber-reinforced silica

fume shotcrete above the springline and cast-in-place concrete below the springline. The original rib sets were left in place. The 1996 repair has performed well, with no signs of movement.

4. 2003 TUNNEL SURVEY AND HYDRAULIC ANALYSIS

Changes in invert slope in the vicinity of the Julie and Kathy L. shear zone complex have been a known issue since tunnel construction. In 2003, a survey was completed to establish a baseline for future monitoring and provide a basis for hydraulic modeling of the existing tunnel conditions. See Figure 3.

Survey data was used to develop HEC-RAS cross sections for hydraulic modeling. The hydraulic model consisted of steady state flow analysis allowing a mixed flow regime (supercritical and subcritical flow regimes possible), with simulated flow rates of 0.71 m³/s (25 cfs) to 18.4 m³/s (650 cfs). Sensitivity analyses performed on model boundary conditions and roughness coefficients showed that results in the area of interest were not significantly affected by changes in these parameters.

Analyzing model results indicated that the abrupt changes in the invert slope at stations 74+45 and 75+65 (see Figure 3) result in a hydraulic jump forming near station 77+25. The tunnel would not be able to pass the design flow of 15.6 m³/s (550 cfs) without violating the flow depth design criterion. Furthermore, USACE (1980) describes how slug flow can develop when the water surface exceeds 80-85% of the conduit height due to unaccounted-for air bulking, particularly with hydraulic jumps.

A HEC-RESSIM model was used to assess the increase in flood risk associated with restricting the Spirit Lake outflow. Flood risk is the risk of the lake elevation exceeding the maximum safe level during a flood event, which could result in debris blockage failure and lake break-out. The minimal increase in flood risk associated with a safe, lower outflow was deemed acceptable compared to the risk associated with pressurization and/or slug flow within the tunnel. A recommendation was made to restrict flow in the tunnel to 9.9 m³/s (350 cfs) by partially closing the intake gate to three-quarters open. Figure 3 shows the water surface profile for this flow.

5. RECENT TUNNEL DISTRESS

Significant tunnel invert heave was discovered during a routine inspection in October 2014 (See Figure 4). The heave is located at the upstream end of the Julie and Kathy L. shear zone complex, in the vicinity of the geologic contact between volcanic tuffs and lava flows and the high-angle Kathy L. shear zone, which required extensive steel supports during original construction. The heave location is about 30 m (100 ft) upstream of the 1996 repair area. The location is identified in Figure 2 as the 2015 repair area.

Figure 5 shows the geologic mapping from original construction and the approximate extent of heave. The heave began a long time ago due to high ground pressures, the weakness of the sheared tuff dipping beneath the tunnel, the adverse orientation of fractures in the basalt, and the absence of steel rib sets. For several years, the amount of heave remained steady at approximately 61 cm (24 in.). The distress was limited to the tunnel invert; the shotcrete lining on the walls and crown showed no cracking or distress. During the October 2014 inspection, 46 cm (18 in.) of additional heave was measured. Existing cracks in the cast-in-place concrete invert showed severe dilation. Again, the walls and crown showed no distress. A follow-up inspection in April 2015 revealed another 10 cm (4 in.) of heave. The tunnel diameter had now been reduced to approximately 2.1 m (6.8 ft). This segment of the tunnel was in an active state of heave failure.

The geometry in the 2003 HEC-RAS hydraulic model was updated with the 2015 physical measurements of the heave. Invert elevations were adjusted in the reach near tunnel station 75+75 to represent the measured reduction of tunnel cross sectional area. The results of the hydraulic modelling indicated that the tunnel would no longer pass 9.9 m³/s (350 cfs) without violating the flow depth design criterion. Flow within the tunnel would require further restriction to 7.1 m³/s (250 cfs) to operate within design criteria. This further reduction in project outflow would significantly increase the risk of unsafe lake levels, which was deemed unacceptable. A repair was needed.

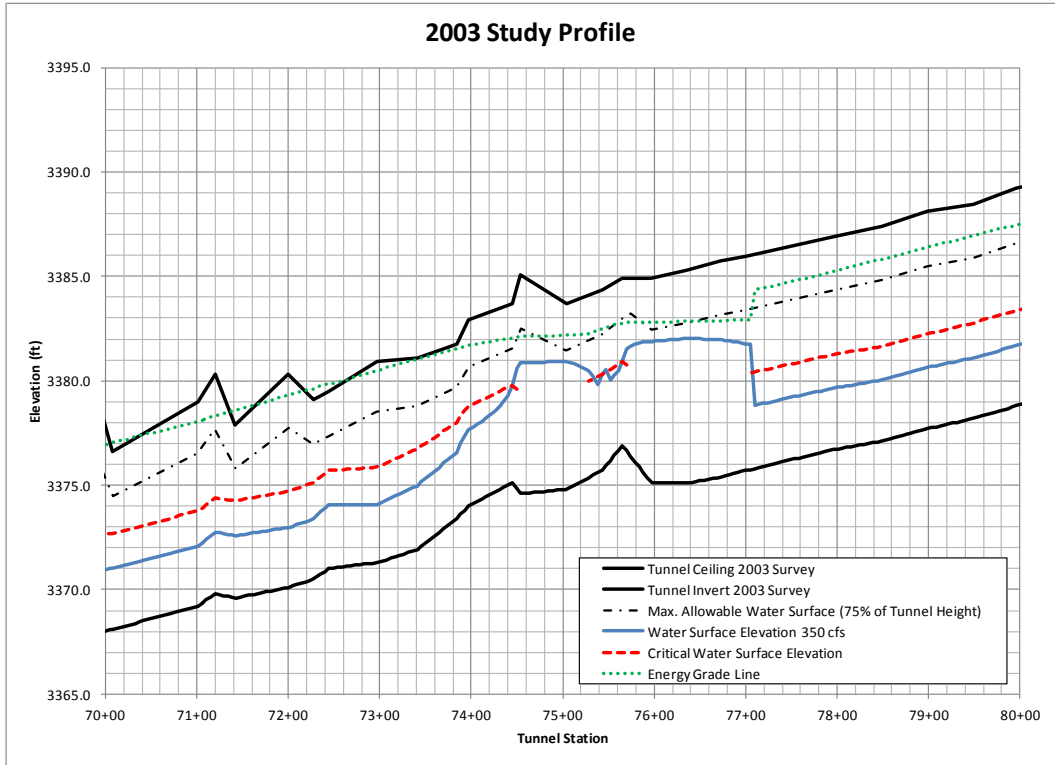


Figure 3. Water surface profile for flow of 9.9 m³/s (350 cfs) and 2003 geometry conditions



Figure 4. Photo of heave in tunnel

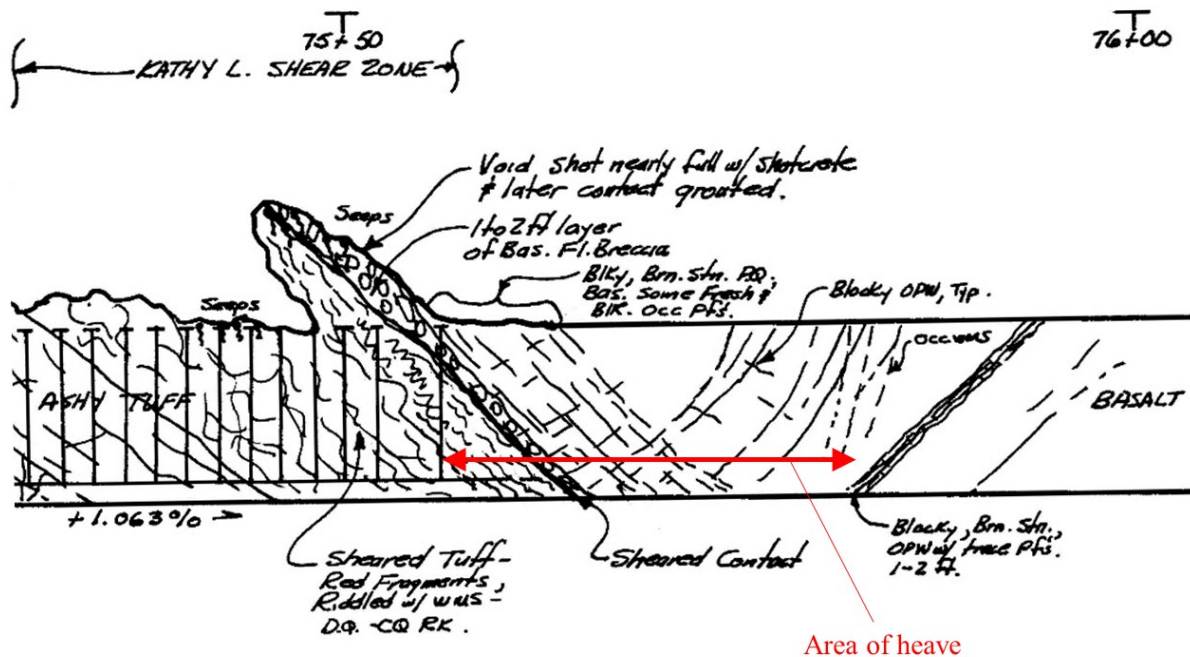


Figure 5. Geology in area of heave (from USACE 1987)

6. 2015 REPAIR DESIGN

The time period from the beginning of repair design to award of the construction contract, including development of plans and specifications and several reviews, was five months, from near the end of April to near the end of September, 2015. Given the continuing heave observed in April, it was considered critical to repair the tunnel before another full winter season of high flows. The design team spent a short amount of time considering various repair options and quickly settled on the rib set-shotcrete support system approach that had been successful for the 1996 repair.

The loading conditions in the area of the 1996 repair were more severe than the loading conditions in the area of the 2015 repair. In the 1996 repair area, the full tunnel circumference was subjected to squeezing ground conditions. In the 2015 repair area, only the invert was subjected to squeezing ground conditions from the sheared tuff dipping below the fractured basalt. The tunnel walls and crown showed no signs of distress. The 2015 repair design called for the installation of full-circle W200x42 (W8x28) rib sets (with 345 MPa/50 ksi steel) on 0.6 m (2 ft) centers. This was the same rib set design as used in 1996, except no rib sets were spaced at 1.2 m (4 ft). The 2015 repair design then called for the rib sets to be covered with a minimum of 50 mm of steel fiber-reinforced shotcrete.

Figure 6 shows both a profile and section of the 2015 repair design. The heave invert profile was measured in April 2015. The heaved material was assumed to consist of about 0.3 m (1 ft) of cracked concrete overlying blocky basalt and sheared tuff. During inspections, it was difficult to observe the invert conditions near the upstream end of the area due to the pool of water behind the crest of the heave profile. The plan was to assess the need to extend the repair into the “optional segment” shown in Figure 6 after the contractor established a cofferdam upstream of the area. At the downstream end of the repair, the exposed and deformed original rib set at station 75+53 was to be removed.

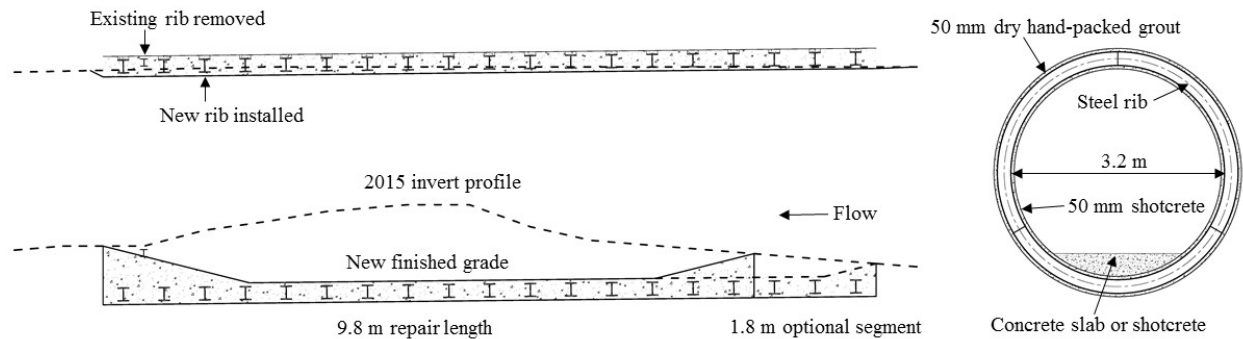


Figure 6. 2015 Repair design profile and section

For safety purposes, the contract required the excavation and support system installation to occur in 1.2 m (4 ft) increments, which covered two rib sets. The grout and shotcrete were required to attain 70% of the design compressive strength (27,600 kPa/4,000 psi at 28 days) before the next 1.2 m excavation increment could be made. The contract allowed for 1.5 m (5 ft) drains to be installed as needed in the walls and crown. The permeable geologic contact at the downstream end of the repair area was exposed to the open tunnel in small areas, and no flow was observed coming from the contact during inspections leading up to the repair.

The 2003 HEC-RAS hydraulic model was updated with the proposed repair geometry. Invert elevations were adjusted to represent the repair cross sectional area with a diameter of 3.1 m (10.2 ft). The results of the hydraulic modelling indicated that the tunnel would pass 11.3 m³/s (400 cfs) without violating the flow depth design criterion (see Figure 7). This flow capacity is greater than the 2003 restricted capacity of 9.9 m³/s (350 cfs). The additional flow capacity will allow for larger gate openings, which will result in lower Spirit Lake elevations. This will decrease the likelihood of exceeding the safe lake elevation. Furthermore, the lower lake elevations will result in less head on the gate, which will reduce the potential of higher flows within the tunnel. This will further decrease the risk of pressurization and/or slug flow within the tunnel.

The 3.2-m inside diameter of the new rib sets (tunnel inside diameter of 3.1 m after application of shotcrete) was chosen in order to achieve an invert profile compatible with a future potential rehabilitation involving stabilizing and re-grading the invert profile of the Julie and Kathy L. shear zone complex. The potential rehabilitation would create a constant tunnel cross-section and slope from station 73+41 to 76+36. The results of hydraulic modeling indicate that the rehabilitated tunnel would pass in excess of 15.6 m³/s (550 cfs) without violating the flow depth design criterion, restoring the capacity of the Spirit Lake tunnel system to its original design capacity. Note that this inside tunnel diameter of 3.1 m (10.2 ft) is slightly less than, but close to, the 3.3 m (10.8 ft) diameter from original design, which allowed for minor offsets during construction and variations in finished tunnel roughness.

7. 2015 REPAIR CONSTRUCTION

A contract was awarded to the prime contractor Catworks Construction and their tunnel subcontractor LRL Construction on 24 September 2015 for \$3 million. While the original intent was to perform construction during the fall of 2015, various delays pushed the beginning of construction inside the tunnel to January 2016. At the time this paper was submitted, in late December 2015, the steel rib sets had been fabricated, construction access to the tunnel outlet (downstream end) had been established, and the majority of construction submittals had been submitted and approved. The authors intend to describe the repair construction during the Symposium in June 2016.

The contractor did submit a variation to the original design. Instead of fully blocking the rib sets with hand-packed grout and using shotcrete as lagging, the contractor proposed a steel lagging and grouting design. Steel channel lagging would be placed between the rib sets and the volume between the excavated ground and the steel ribs/lagging would be filled with non-shrink grout. The ribs and lagging would then be covered with shotcrete (with synthetic as opposed to steel fibers). This contractor proposal was accepted.

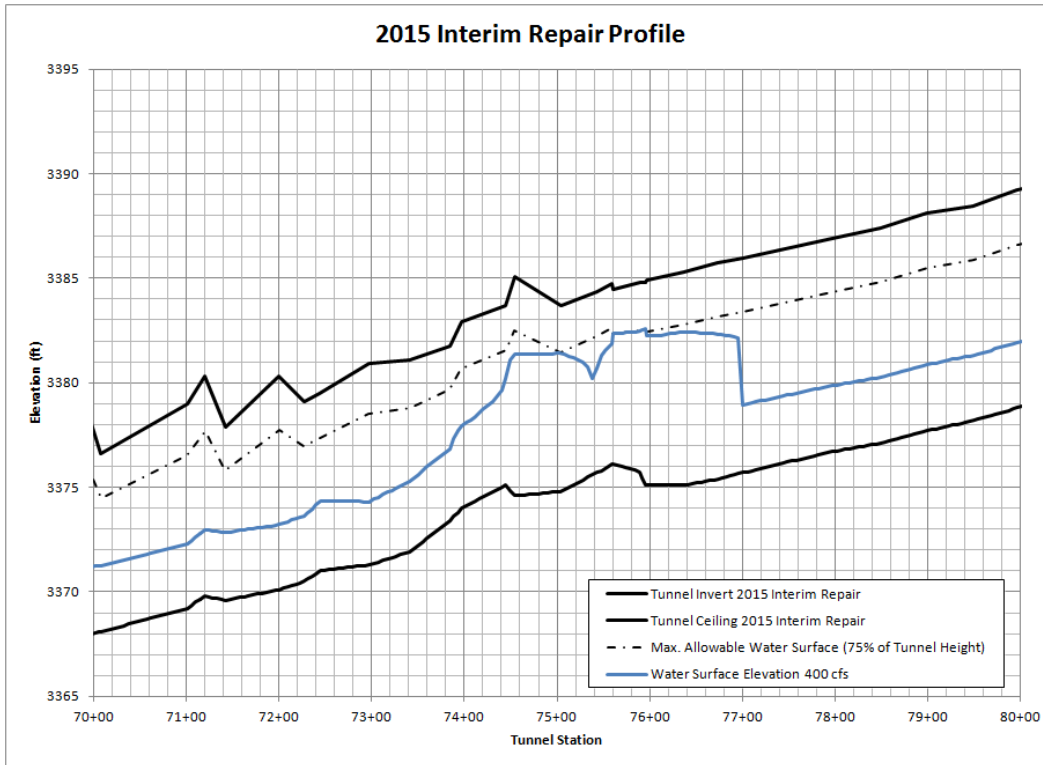


Figure 7. Water surface profile for flow of $11.3 \text{ m}^3/\text{s}$ (400 cfs) and 2015 repair geometry conditions

8. CONCLUSIONS

A rib set-shotcrete support system has been successfully used for support in the tunnel in areas of weak ground (shear zones and the tuff/basalt geologic contact). Within the Julie and Kathy L. shear zone complex, which includes soft and squeezing ground conditions, repair designs in 1996 and 2015 used full-circle W200x42 (W8x28) rib sets, with 345 MPa (50 ksi) steel, on 0.6 m (2 ft) centers. The 2015 contractor-proposed design used steel lagging and grout instead of shotcrete for lagging. Given the positive long-term stability performance of the 1996 repair and the successful implementation of the 2015 repair, this rib set design should be one alternative to consider for the future potential rehabilitation of the entire Julie and Kathy L. shear zone complex.

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