Gorge 2nd Tunnel

Tunnel Diameter Optimization, Layout, and Water Transient Study Report

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Executive Summary

The Gorge 2nd Tunnel (G2T) project is proposed to bore a second power tunnel between Gorge Dam and the Gorge Powerhouse to increase efficiency. The second power tunnel will reduce head loss, raise the head pressure at the turbines, increase the torque on the generators, and produce more power for any given flow.

This report details the evaluations performed to develop tunnel alignment, connection configurations, and other project features that support construction cost estimating and hydraulic modeling. The results of the hydraulic modeling and outcomes of the June 9, 2009 Decision Workshop have also been incorporated in the report.

The G2T will be approximately 11,000 feet long and parallel the existing power tunnel. Excavated tunnel sizes of 14, 16, and 18 feet were initially considered. Larger excavated tunnel diameters of 20 and 22 feet were considered after the draft issue of this report. Connections into the existing power tunnel were selected in a June 9, 2009 Decision Workshop: the upstream connection will be at the base of the existing intake structure ramp, a location that maximizes hydraulic efficiency; and the downstream connection will be located just upstream of the bifurcation (described in the report as “Link 3”) that feeds all of the powerhouse units. This location was selected based on the most favorable hydraulic conditions over a range of operating scenarios.

Tunnel boring machine (TBM) methods are most appropriate for the tunnel construction, with use of drill-and-blast methods for construction of the starter tunnel and connection adits. The rock consists of Skagit Gneiss, a mineralogically banded, metamorphosed igneous rock of granitic composition. The rock is anticipated to have medium strength and abrasivity and is considered favorable for tunneling. The majority or the rock is anticipated to be strong and durable enough to remain unlined after construction.

Materials excavated from the tunnel will transported offsite and used as fill for a wildlife restoration project in an abandoned quarry. The estimated volume of material to be disposed of is between 115,000 and 280,000 cubic yards (bulked), given the range of proposed tunnel diameters.

Staging areas evaluations included powerhouse bridge capacity analysis and construction power requirements. Capacity analyses indicated that the TBM will need to be transported in smaller elements than typical so that the Gorge Powerhouse Bridge will not be overloaded. Construction power evaluations indicated that a power drop of approximately 6,500 kVA is required.

Initial baseline hydraulic modeling was performed for the current tunnel configuration to evaluate friction factors, with a subsequent model created to incorporate the addition of G2T, which simulates total system flows between 1,000 cubic feet per second (cfs) and 7,700 cfs. This stated system flow of 7,700 cfs represents the upper limit for all four (4) turbines combined at the Gorge Powerhouse, as stated in the June 2008 Planning Document for the project. However, during the Preliminary Design Phase, SCL identified that the station flow would be restricted by the 1995 Water Rights agreement—this flow was initially thought to be limited to a maximum flow of 7,440 cfs. This figure has since been confirmed to actually be 7,500 cfs. During high water events SCL routes flows greater than 7,500 cfs through the powerhouse to prevent spill over the dam.

From a hydraulic operation perspective, the G2T will not provide a power output of 207 MW at a station flow of 7,440 cfs. The maximum predicted power output at 7,440 cfs is around 199 MW at a forebay elevation of 874 ft and would require a 33-foot-diameter tunnel, after which the power output fails to

increase despite increasing the tunnel diameter. The results indicate that 202 MW can be achieved at 7,700 cfs with a 22-foot-diameter tunnel. The predicted power output for a 22-foot-diameter tunnel and a station flow rate of 7,440 cfs is approximately 196 MW. The predicted head loss with a new second tunnel was compared with head loss predictions for the existing water conveyance to determine the head loss reduction for a range of second tunnel diameters. The key results of the modeling include:

- The existing surge tank is sufficient to manage transient pressures;
- Head loss reduction for a station flow of 7,440 cfs is approximately 43.5 feet for a 22-foot-diameter tunnel and 25.4 feet for a 14-foot-diameter tunnel; and
- Head loss reduction for a station flow of 7,700 cfs is approximately 47.3 feet for a 22-foot-diameter tunnel and 27.3 feet for a 14-foot-diameter tunnel.

Additional modeling towards the end of the preliminary engineering phase was carried out to determine the maximum hydraulic capacity for a 22-foot-diameter new tunnel and the Link 3 connection. The output from this modeling indicated that a station flow of 7,500 cfs is achievable with a forebay elevation of only 817 feet (considered to be extremely low). Considering a normal forebay elevation of 874 feet, the calculated hydraulic capacity (station outflow) increases slightly to 8,220 cfs from 8,112 cfs which is considered to be the current hydraulic capacity based on data from a high water event in 2006.

During Final Design it is recommended that the downstream connection detail be examined in greater detail in order to further refine the connection angles, minimize associated friction losses, and provide additional input data for final hydraulic modeling of the preferred tunnel diameter and configuration.

Construction cost estimates were prepared for 14-, 16-, 18-, 20- and 22-foot-diameter tunnels, yielding a range from about $47.0 to $56.7 million—a 20 percent difference between the smallest and largest. In addition to cost considerations, several other factors influence the selection of tunnel diameter, including the results of the energy analysis, and final lining assumptions (head loss will increase if greater lengths of tunnel require a lining). A larger tunnel would provide a margin of safety for maintaining the desired hydraulic performance.

A 20- to 22-foot-diameter G2T will optimize hydraulic performance and is considered capable of delivering the required power output enhancement.

The energy and financial analyses for the recommended tunnel diameter of 22 feet gave approximately 56,000 MWh/year of additional energy, with a corresponding levelized annual value for this energy of $2.7 million per year—using a natural gas price of $6.00/MmBtu. The Renewable Energy Credit benefit associated with the Gorge 2nd Tunnel Project could increase this value by a further 65 percent to approximately $4.1 million per year.

1 Introduction

The Gorge 2nd Tunnel (G2T) Project proposes to bore a second power tunnel through rock between the Gorge Dam and the Gorge Powerhouse to increase the efficiency of the powerhouse. This will reduce frictional head loss of the water during tunnel conveyance, raise the head pressure at the turbines, increase the torque on the generators, and produce more power for any given flow. The new energy that the G2T produces will count towards SCL's commitment to renewable energy and will reduce the amount of energy produced by fossil fuels that the utility currently has to buy to meet its customers' needs.

1.1 Project Background

The Gorge Powerhouse is located on the Skagit River in Newhalem, Whatcom County, Washington, in the Upper Skagit River watershed, and is one of three generating facilities operated by Seattle City Light (SCL) as part of the Skagit Hydroelectric Project. The Gorge Powerhouse is capable of generating 176 megawatts (MW) at a gross head of 380 feet. The nameplate-generating capacity is 207.5 MW. The powerhouse has four turbine-generator units. Three of the units were installed in the 1920s, and the fourth unit was installed in 1951.

Station flow measurements conducted at the Gorge Powerhouse in 1992 to 1993 recorded a maximum station flow of 7,876 cfs. This testing was conducted following turbine overhauls and runner replacements for Units 21 to 23. This testing was conducted prior to the Unit 24 turbine overhaul and runner replacement which occurred in 2006. Information obtained from a high water event in November 2006 has subsequently shown that the Gorge Powerhouse maximum hydraulic capacity is actually 8,112 cfs. However, the station flow is restricted by the Water Rights agreement downstream of the Gorge Powerhouse which currently limits the maximum station output to 7,500 cfs (this figure was previously stated as 7,440 cfs, which formed the basis of the early hydraulic modeling work).

The site is shown on the Site Vicinity Map (Figure 1).

In 1918, the federal government granted the City of Seattle permission to start developing hydroelectric-generating facilities on the Skagit River. The following year, the City's electrical utility, SCL, began constructing the Gorge timber crib dam, the Gorge Powerhouse, and an 11,000-foot-long, 20.5-foot-diameter, concrete-lined tunnel to convey water from the dam to the powerhouse. In 1927, the Federal Power Commission, later called the Federal Energy Regulatory Commission (FERC), issued the first license to the City of Seattle for its facilities on the Skagit, thereafter called Project 553.

The original license has been amended by FERC several times, and the entire Skagit Project 553 was relicensed in 1991. The Gorge operation was most significantly modified in 1949, when SCL increased power generation capacity by installing another generator and adding 88 feet of head on the turbines by building High Gorge Dam. These improvements increased the Gorge Powerhouse's output as intended, but they also sped up the water velocity in the power tunnel, and the corresponding growth of frictional head loss lowered the efficiency of the powerhouse. The full generating capacity of the facility cannot be achieved because of head loss through the existing power tunnel. SCL has determined that a second power tunnel is necessary to reduce system head loss and increase generating efficiency.

A study of a second tunnel began in 2006, concurrent with the approval of Initiative I-937 by Washington State voters, also called the Energy Independence Act (Chapter 19.285 RCW). The Energy Independence Act requires large utilities, such as SCL, to obtain 15 percent of the energy they sell from renewable sources by 2015.

1.2 Purpose

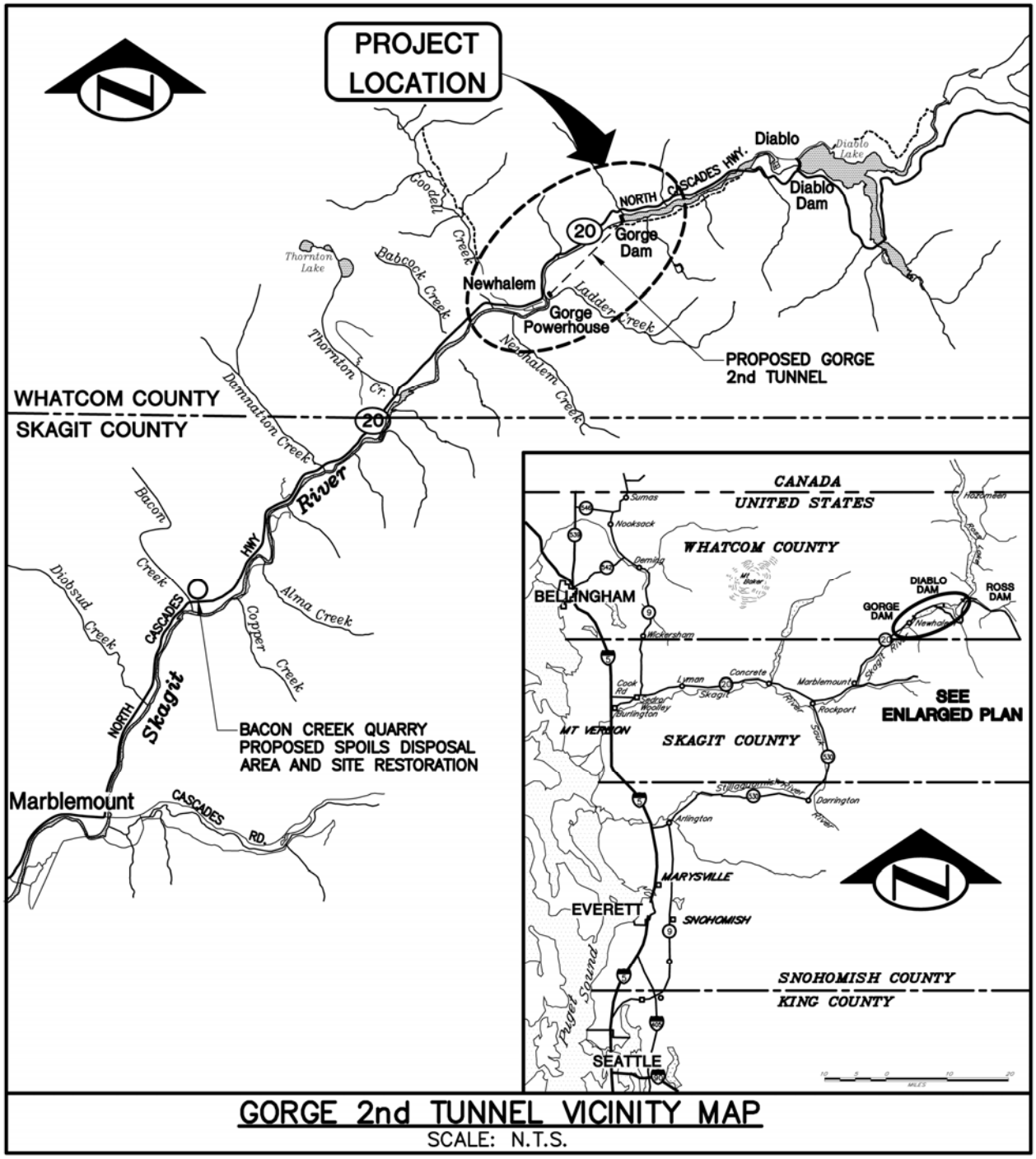
This report summarizes the results of tunnel diameter optimization and layout evaluations, construction cost estimating, and hydraulic analyses for the G2T Project. The objective of the evaluations and analyses is to develop a project configuration that optimizes hydraulic performance, construction cost, and power production. Economic energy analyses will be performed to confirm this project objective and the preferred project configuration will be taken forward into Final Design.

1.3 Scope

This report details preliminary design tasks undertaken to evaluate and recommend the most economical diameter and layout of the G2T. It provides a discussion of the parameters affecting the selection of the tunnel alignment and layout as well as the design criteria used in the tunnel diameter optimization study. This includes the determination of hydraulic, operation and maintenance, and system operating requirements. Hydraulic baseline modeling of the existing power tunnel and powerhouse configurations was performed to form the basis for hydraulic analyses of alternative configurations for G2T. The methodologies and criteria for the screening and evaluation of alternative configurations were developed and confirmed in a Decision Workshop conducted on June 9, 2009. Notes from the Decision Workshop are included as Appendix A. Additional hydraulic modeling and construction cost estimates were carried out and developed following the draft submission of this report in August 2009.

1.4 Construction Methodology

The G2T will parallel the existing power tunnel. A tunnel boring machine will be used to excavate through rock to construct the G2T, and drill-and-blast methods will be used for the TBM starter tunnels as well as the connections between the new and existing power tunnels. The TBM will be launched from a portal/access site located in the portal staging area next to the Gorge Powerhouse. Access to the site is via the existing steel Gorge Powerhouse Bridge crossing the Skagit River. Material excavated from the tunnel will be transported offsite for use as recontouring fill material at a nearby mitigation site.



GORGE 2nd TUNNEL VICINITY MAP
SCALE: N.T.S.

Figure 1. Site Vicinity Map

2 Preliminary Geologic Conditions

Geotechnical exploration and interpretation activities are ongoing and will not be completed until after the issue of this report. The G2T Field Investigation Planning is included as Appendix C. Available geotechnical information and surface mapping observations have been used for an assessment of preliminary geologic conditions.

2.1 Regional Geology

The project area is located in the North Cascades range of north-central Washington State. The geology of the North Cascades is a tectonically complex series of geologic terranes that have accreted over the last 400 million years to form the landmass of western North America. The geologic terranes that are now joined together include remnants of volcanic island arcs, deep ocean sediments, ocean floor basalt, old continental crust, submarine fans, and subcrustal mantle. The processes of accretion, burial, exhumation, and volcanism have resulted in rocks that are strongly folded, faulted, and metamorphosed.

The terranes of the North Cascades can be tectonically divided into three sections separated by two major strike-slip fault zones, the Straight Creek Fault and the Ross Lake Fault Zone. Rocks to the west of the Straight Creek Fault and east of the Ross Lake Fault Zone are generally less metamorphosed than those rocks that lie between the two faults and within the “metamorphic core.” These metamorphic core rocks have been uplifted approximately 7 to 11 miles relative to the rocks on either side.

Newhalem and the Gorge Powerhouse are located in the metamorphic core of the North Cascades, the zone of deformation between two major fault zones. The bedrock at the project site has been thoroughly metamorphosed and exposed to various episodes of faulting and shearing, which has created zones of weakness that must be considered throughout the development of the project. Geology specific to the project site is discussed in more detail in the following sections.

A summary of observations and measurements is provided in the Preliminary Geologic Assessment, included as Appendix D.

2.2 Anticipated Geologic Conditions

Bedrock along the tunnel alignment is expected to consist primarily of orthogneiss, a foliated (mineralogically banded), metamorphosed igneous rock of granitic composition that crops out mostly between Newhalem and Ross Lake, to the east. Orthogneiss consists primarily of the minerals plagioclase feldspar, quartz, biotite, and hornblende. Specifically, the orthogneiss in the alignment area consists of primarily medium- to coarse-grained (1-mm- to 20-mm-long crystals) foliated hornblende-biotite tonalite gneiss and quartz diorite gneiss.

A pegmatite (very coarse-grained intrusive igneous rock) with crystals greater than 20 mm in length, containing a mineral assemblage similar to the orthogneiss but with a much higher feldspar and quartz content (JA, 2008), is also present in the form of large concordant and discordant intrusions through the bedrock. Based on observations within the Devil’s Elbow Adit, these intrusions can be greater than 100 feet in width (HMM, 2007).

According to the HMM 2007 study, the overall rock mass (orthogneiss and pegmatite) may be made up of up to 25 percent quartz, a hard mineral, and may contain approximately 2 to 3 percent garnet. As based on HMM’s October 12, 2006, field observations and petrographic analysis of samples collected during this

visit (HMM 2007), no erodible minerals are expected within the rock mass. Erodible minerals are relevant to tunnel excavation and construction because of their capacity to weaken the rock and be easily removed by water.

2.2.1 Weathering

Bedrock in the vicinity of the proposed tunnel alignment is generally fresh and unweathered, but is moderately to highly weathered in fault/shear zones (HMM 2007). The 1954 geologic report for the Gorge High Dam (SCL, 1954a) indicates the presence of highly weathered zones of bedrock around joints and faults, and emphasizes the presence of unevenly distributed iron sulfides (which can lead to accelerated bedrock weathering) in the rocks at Gorge High Dam. These observations were supported by more recent field observations (JA, 2008).

2.2.2 Rock Strength

Uniaxial compression strength (UCS) tests conducted as part of the HMM 2007 study indicate rock strengths ranging from 7,800 to 19,500 psi. Two samples that were deemed to have failed prematurely along weak fractures were filtered out of the overall results. Discounting these two values, the average bedrock strength value is approximately 18,100 psi.

2.2.3 Mineralogy

Petrographic analysis results indicate a primary composition of quartz, plagioclase feldspar, hornblende, and biotite (HMM, 2007). Hard minerals relevant to tunnel excavation are present in the form of significant quartz content (10 to 25 percent), and a minor amount (2 to 8 percent) of garnet. No indications of sulfides were observed in the samples. Minor bedrock alteration was observed, with some sericite and a very limited amount of microfracturing present.

2.2.4 Foliation

Preferential alignment and banding of minerals in metamorphic rock (known as foliation) is reported to range from weak to strong in the proposed tunnel alignment vicinity. The HMM 2007 report indicates that bedrock is strongly subhorizontally foliated. In a site visit log (JA, 2008), it is reported that in the Devil's Elbow Adit, the orthogneiss is commonly, though not exclusively, subhorizontally banded, and indicates observed weak to moderate foliation. Weakly to moderately foliated bedrock was observed during the March 10, 2009, reconnaissance of the downstream portal and several of the drill sites for the proposed 2009 geotechnical investigation (Aspect, 2009).

2.2.5 Joints

The tunnel alignment area rock mass is generally very widely jointed with joint spacing ranging from 3 to 10 feet and with zones of extremely wide joint spacing of greater than10 feet. Moderate jointing is expected at the site of the new downstream portal, located in a hillside to the north of the Powerhouse. The joints are generally tight, unstained, and unmineralized, with the exception of shears and fault-related sheared zones.

2.2.6 Faults

Several faults have been observed in the project area. Additional large-scale lineaments of unknown origin have been identified over the course of initial stages of the current study, as well as in the HMM 2007 report (based on topography, aerial photos, and geologic maps).

2.3 Groundwater

Regional maps do not show perennial surface water features that cross the alignment. Except during flood releases from Gorge Dam, the entire flow of the river is diverted through the existing Gorge tunnel. The riverbed lies below the level of the tunnel, and the pools and minor flow in the riverbed are believed to be fed by groundwater flow, with minor contributions from ephemeral surface flows from the bedrock slopes above the river and streams on the northwest side of the river. The topographic position of the river below the tunnel suggests that groundwater at the site will result from infiltration of rainfall and seasonal snowmelt into fractures and other structures in the rock mass.

Based on field observations, the groundwater surface is expected to generally follow the surface topography of the tunnel alignment. Therefore, the entire tunnel will be excavated beneath the groundwater table. Groundwater levels will vary seasonally and depend on the amount of precipitation.

3 Tunnel Alignment and Layout

This section describes key preliminary design criteria and considerations that were used to develop and evaluate alignment and layout configurations for the G2T. The criteria and considerations fall within five general categories: horizontal alignment, tunnel grade, upstream tunnel connection, downstream tunnel connection, and portal location. Considerations from each category were balanced with other requirements such as hydraulic performance, constructability, risk, and cost to develop the recommended alignment.

3.1 Horizontal Alignment

The criteria and considerations for layout and evaluation of the G2T horizontal alignment are to:

- Maintain safe offset from existing power tunnel;
- Maintain a straight departure from the tunnel portal;
- Maintain 1,000-foot minimum radius horizontal curves;
- Minimize the number of horizontal curves; and
- Keep the alignment within the surface boundaries to the maximum extent possible.

3.1.1 Horizontal Offset from Existing Power Tunnel

During construction of the G2T, the existing power tunnel will be in operation (i.e., it will be pressurized). The separation between the two tunnels must be large enough so that construction of the G2T does not impact the integrity of the existing power tunnel. As G2T is excavated, the stresses in the rock will redistribute around the opening. The redistribution of stress will be more pronounced near the opening and diminish radially as the distance from the opening increases. Altered stress fields can result in local instabilities and increased stresses around the tunnel profile. The separation between tunnels must be sufficient so that local instabilities in the new tunnel do not impact the stability of the rock around the existing tunnel.

Stress field interaction between the two tunnels was evaluated. Figure 2 illustrates the zone of stress redistribution during excavation of a circular tunnel. A separation range of 1.75 to 2.0 times the excavated tunnel diameter is considered sufficient to avoid adverse stress interaction effects. For an assumed tunnel diameter of 18 feet, this gives an “unfactored” separation range of about 31.5 to 36.0 feet.

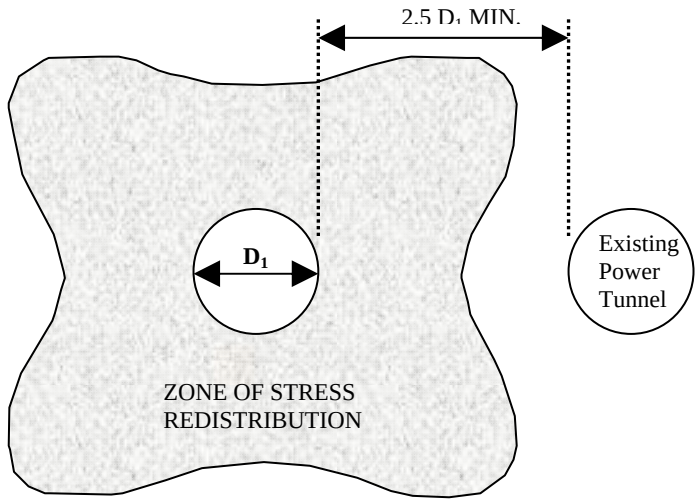


Figure 2. Suggested Separation between Adjacent Tunnels

For preliminary design, a separation of 50 feet is recommended to provide a margin of safety (i.e., buffer separation outside of the potential zone of stress redistribution). This will take into account any local instabilities or weakened zones of rock that may exist around the existing tunnel. Additional evaluations using numerical modeling will be necessary to verify that this separation is adequate and to assess potential impacts of leakage from the existing power tunnel into the G2T excavation. These evaluations will be performed during Final Design as further geologic data become available.

3.1.2 Straight Line Departure

With the use of a TBM, mining an initial straight section is desirable so the TBM can be properly set up and commissioned without the additional “difficulty” of horizontal alignment control in a curve. The TBM will also have trailing gear and setting up on a straight tangent allows for easier connections and testing of the TBM systems. The trailing gear is typically 250 to 300 feet long. An initial straight section of at least the length of the TBM and trailing gear is considered good practice to set up and launch the machine and allow development of mining/construction procedures and efficient cycle times before reaching the first curve.

3.1.3 Minimum Horizontal Curve Radius

The minimum horizontal radius curve a tunnel can excavate is a restricted by the tunnel diameter, length of the machine shield, cutterhead features, and the machine articulation capabilities. A minimum radius of 1,000 feet is considered appropriate for TBMs of the size anticipated for the G2T. Although a slightly tighter curve would be feasible, it would increase steering difficulties and can slow down the advance rate of the tunnel as the machine excavates and advances in a series of smaller cuts. Conversely, a flatter curve could be used, however this would result in an increase in the length of curve, and increased efforts required to survey and control the horizontal alignment.

3.1.4 Number of Horizontal Curves

Preplanning and continuous survey control efforts are required to start the curve correctly and to guide the TBM excavation through the curve to the end. It is generally good tunnel design practice to minimize the number of curves to the extent possible to simplify construction.

3.1.5 Project Surface Boundaries

Two distinct project surface boundaries have been identified that the tunnel alignment must avoid crossing to the best extent possible. These are the Stephen Mather Wilderness boundary and the Federal Energy Regulatory Commission (FERC) boundary. Both project boundaries are briefly described below.

3.1.5.1 Stephen Mather Wilderness Boundary

The Stephen Mather Wilderness boundary generally follows the 500-meter (1,640 foot) contour around and above the Skagit River to the intersection of Colonial Creek. Approximately 1,700 feet to the east of the Gorge Powerhouse, the Wilderness boundary cuts North across a valley dropping below the 1,640-foot contour before it re-intersects the contour and then resumes following the contour line. The G2T alignment is allowed to cross the Wilderness boundary, but construction activities will not be permitted to disturb the ground surface or require surface access within the Wilderness boundary (i.e., the tunnel can pass beneath the Wilderness area but cannot use any surface area within the boundary for staging or construction activities). The existing power tunnel currently cuts across the Wilderness boundary but there are no surface indications of the tunnel. Figure 3 shows the Wilderness boundary in the vicinity of the Gorge powerhouse and dam.

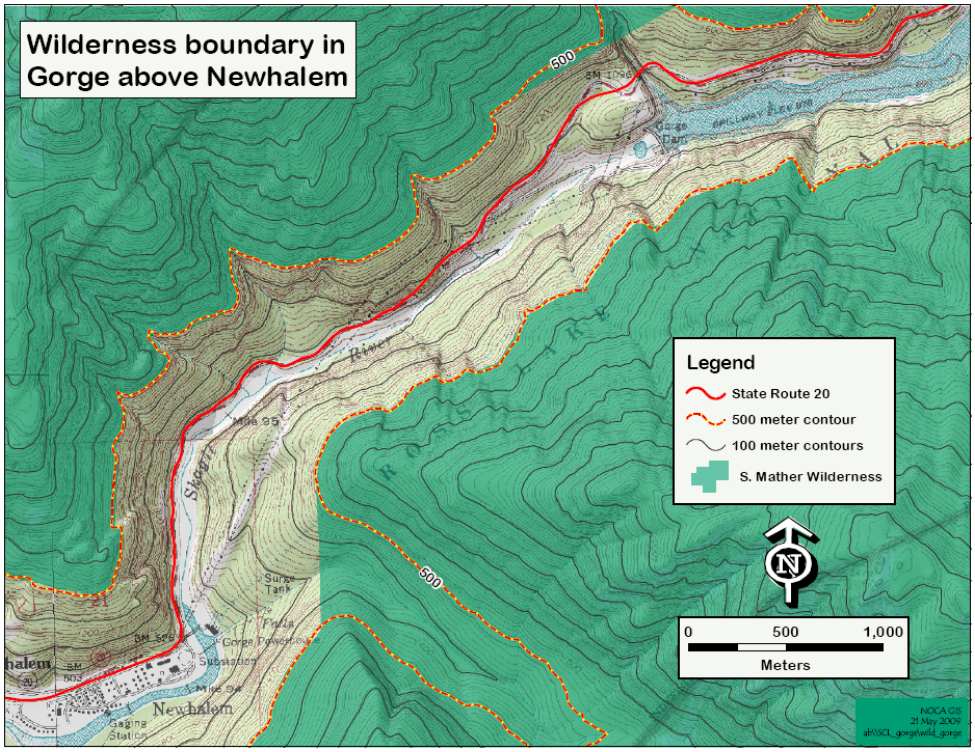


Figure 3. Wilderness Boundary in the Vicinity of the Gorge Powerhouse and Dam

3.1.5.2 Federal Energy Regulatory Commission (FERC) Boundary

The FERC "project boundary" is the area established by the FERC to enclose the lands, waters, and structures needed to operate the Skagit Hydroelectric Project. The FERC boundary is generally a 200-foot-wide corridor centered along the existing Gorge tunnel. It also includes an approximately 300-foot-wide area that follows the power line alignment to the north of the existing tunnel. Near the Gorge Dam and Gorge Powerhouse, the boundary expands to include the structures associated with the Skagit Hydroelectric Project. The proposed G2T alignment has been selected to stay within the FERC boundary as much as possible without introducing any potential increased tunneling risks (e.g., proximity to existing tunnel). Figure 4 shows the FERC boundary in the vicinity of the Gorge powerhouse and dam.

3.2 Tunnel Profile

The criteria and considerations for layout and evaluation of the G2T profile are:

- Maintain efficient tunnel grades;
- Maintain safe offset from existing Devil's Elbow Adit; and
- Existing elevations of connection locations.

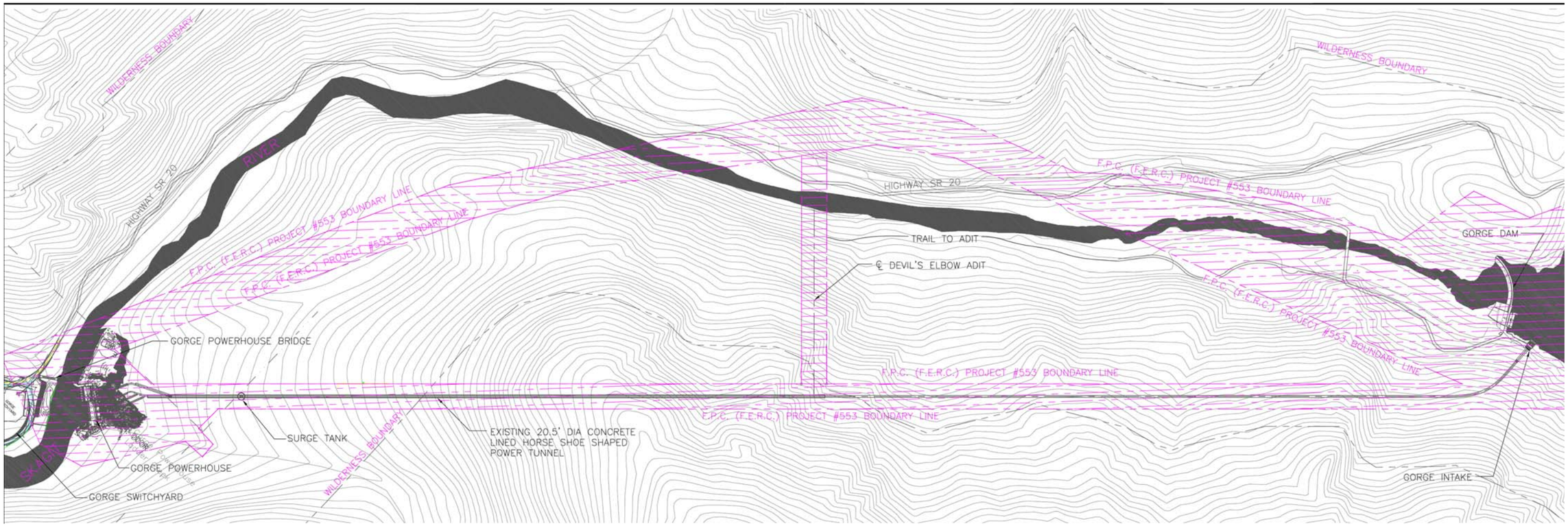


Figure 4. FERC Boundary in the Vicinity of the Gorge Powerhouse and Dam

3.2.1 Tunnel Grade

The G2T will be driven upslope from the portal to an upstream connection with the existing power tunnel. Driving the tunnel upslope has constructability benefits in that any construction process water and natural groundwater flow into the tunnel will drain to the portal by gravity. Conversely, downgrade drives require systematic pumping to remove water from a tunnel. There is also a safety benefit in driving a tunnel upslope in that water is not able to accumulate at the tunnel face thereby eliminating the potential for water inundation underground.

A flat tunnel grade provides for safe rail equipment operation; however, it can result in ponding water within the tunnel through dips in the grade. For the G2T, an initial flat grade is desirable to simplify TBM launching and to provide a safer rail grade transition before exiting the portal. A transition to a slightly steeper grade after an initial flat section will provide benefits for water drainage as the potential for inflows increase due to higher groundwater levels as the G2T length increases.

3.2.2 Vertical Separation from Devil's Elbow Adit

The separation between the G2T and the Devil's Elbow Adit must be large enough so that construction and operation of the G2T does not impact the integrity of the existing adit. Using the same approach as described in Section 3.1.1, a vertical separation ranging from 1.75 to 2.0 times the G2T diameter should be provided beneath the adit.

3.2.3 Connection Elevations

The downstream and upstream connections are at largely fixed vertical locations and the G2T must be driven between these two connection points. The slope from the portal will have to be selected so that the G2T will be at the optimum elevation for the downstream connection to the existing power tunnel and the slope from the downstream connection point upstream will have to be selected so that the G2T will be at the optimum elevation for the upstream connection to the existing power tunnel.

3.3 Upstream Connection

The upstream connection will require the use of drill-and-blast techniques to make the connection between the G2T and the exiting power tunnel. The upstream connection angle is anticipated to range from 30 to 45 degrees and have an approximate length of 45 feet. Figure 5 provides a plan view of the likely G2T upstream connection, showing where the TBM driven tunnel will end, and where the drill-and-blast connection will be made. In profile, the G2T will connect to the existing power tunnel at the base of the existing intake ramp. Figure 6 provides a profile of the G2T upstream connection. The connection between the two tunnels will require a complete shut-down and complete drainage of the existing power tunnel. The location of the upstream connection was selected for the following reasons:

- During the construction of the High Gorge Dam intake structure, a large amount of grouting was carried out in the area surrounding the access tunnel connection to the existing tunnel. This grouting reduces the potential for rock instability by stabilizing the rock mass. Additionally, there is a zone of backfill concrete around the access tunnel connection which allows for a more stable and reliable break-in location to the existing tunnel.
- The position of this connection maximizes its hydraulic efficiency (lower head loss) by maximizing the length of TBM driven tunnel and optimizing the connection angle.

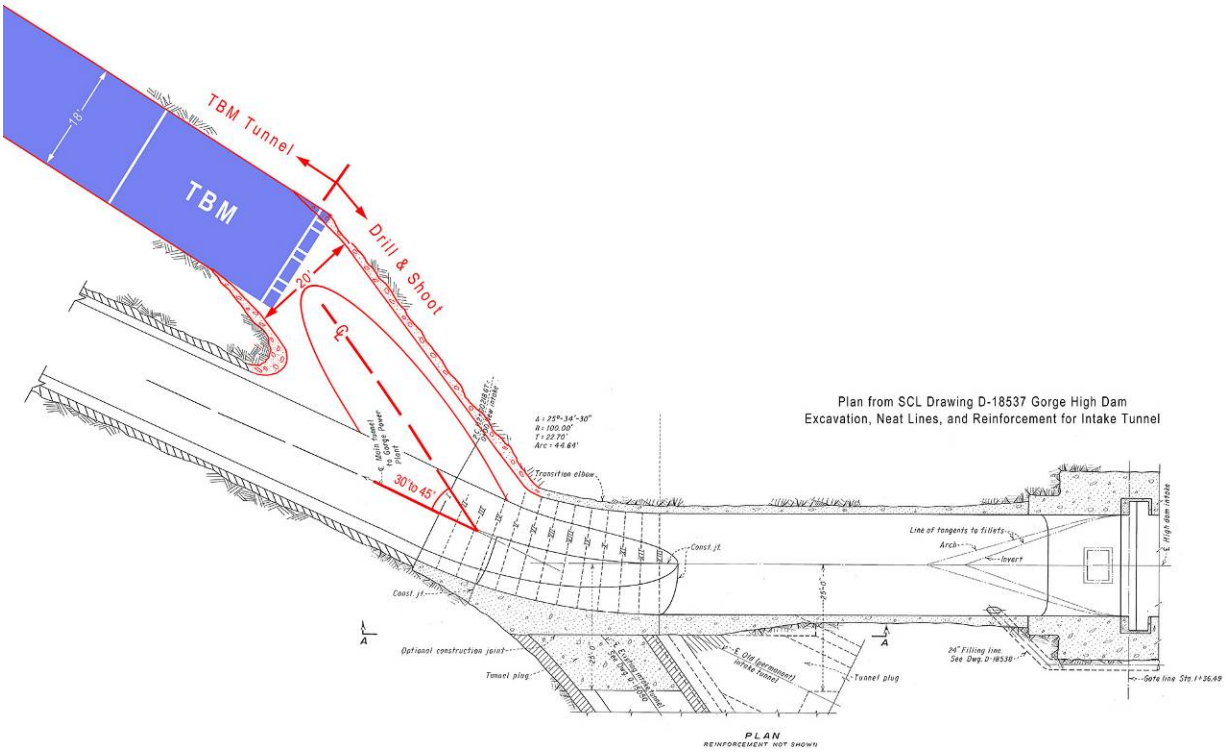


Figure 5. G2T Upstream Connection Plan

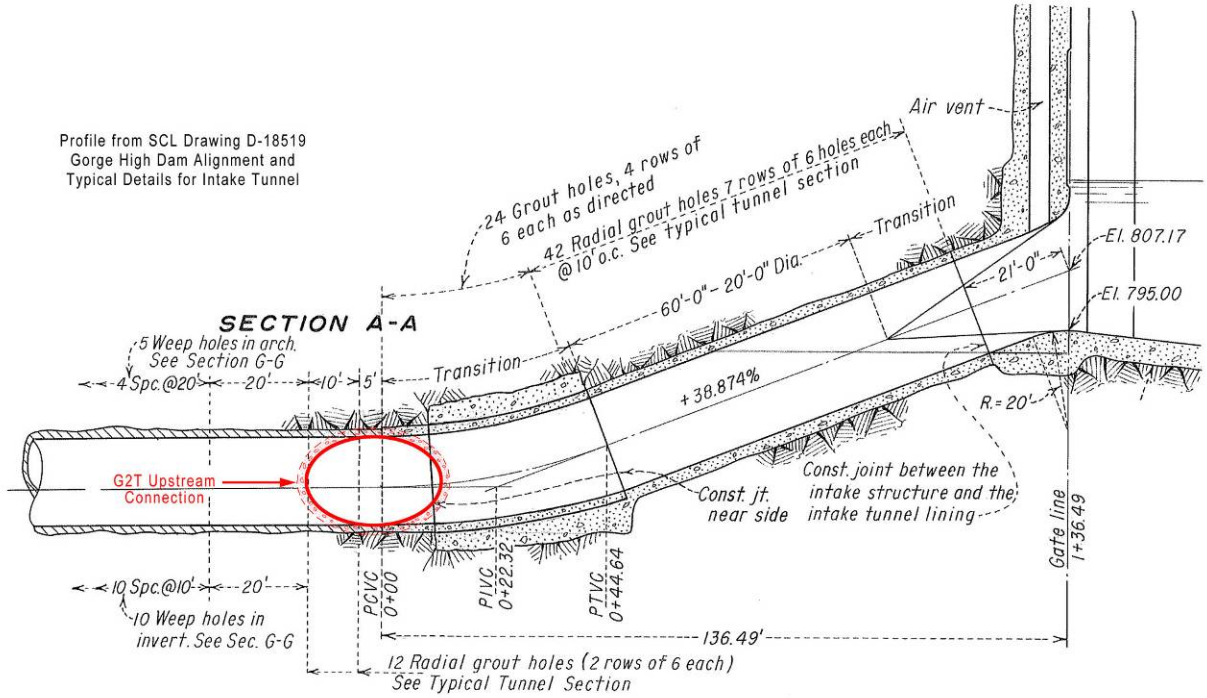


Figure 6. G2T Upstream Connection Profile

During Final Design it is recommended that the upstream connection detail be examined in greater detail in order to further refine the connection angle and connection tunnel length and minimize outage time.

3.4 Downstream Connection

The location and orientation of the downstream connection has the most significant influence on the flow balance between Units 21 through 23, and Unit 24, and on the surge shaft effectiveness for the system. The connection is also one of the higher risk elements to the project because it will require special sequencing, nonuniform support, and challenging lining transitions. Four alternative downstream connections were developed and evaluated for the G2T. Details of these four connections are briefly summarized below and presented pictorially in Figure 7. Based on visual observation, Links 1 and 2 are skewed toward Unit 24 operation and would result in bidirectional flow if Unit 24 is off. Link 3 eliminates bidirectional flow. Link 4 connects the G2T to the existing surge shaft and is included for transient analyses evaluations.

3.4.1 Link 1

This connection ties into existing Unit 24 branch tunnel just before the Unit 24 penstock. The adit excavation departs the G2T at a 45-degree angle. The connection length is approximately 75 feet. This connection is skewed towards Unit 24 operation.

3.4.2 Link 2

This connection ties into the midpoint of the Unit 24 branch tunnel. The adit excavation departs the G2T at a 45-degree angle. The connection length is approximately 75 feet. This connection is slightly skewed toward Unit 24 operation.

3.4.3 Link 3

This connection ties into existing power tunnel just upstream of the bifurcation that feeds Units 21 through 24. The adit excavation departs the G2T at a 45-degree angle. The connection length is approximately 85 feet.

3.4.4 Link 4

This connection ties into the existing power tunnel just upstream of the existing surge shaft. The adit excavation departs the G2T at a 45-degree angle. The connection length is approximately 75 feet.

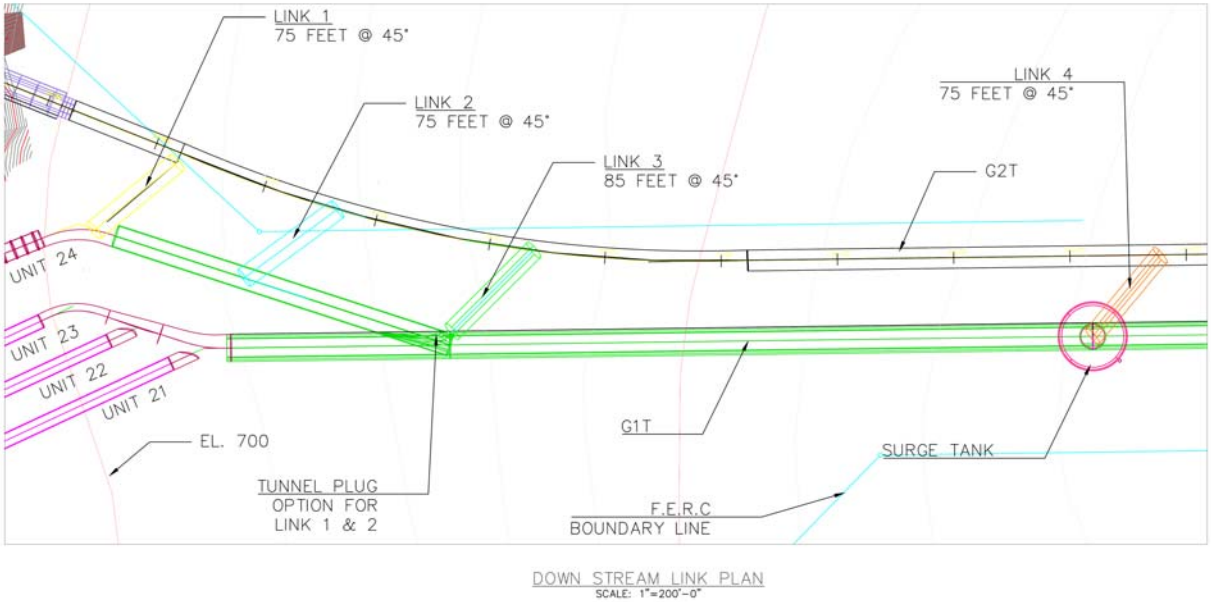


Figure 7. Downstream Connection Alternatives

3.4.5 Preferred Downstream Connection

Link 3 was selected as the preferred downstream connection primarily based on the results of the hydraulic analyses discussed in Section 7. Link 3 eliminates bidirectional flow issues and has a better overall hydraulic performance. Additional details of the downstream connection are shown on Drawing D-111 in Appendix G.

During Final Design it is recommended that the downstream connection detail be examined in greater detail in order to further refine the connection angles, minimize associated friction losses, and provide additional input data for final hydraulic modeling of the preferred tunnel diameter and configuration.

3.5 Portal Layout

The portal staging area will be located immediately north of the Gorge Powerhouse in an area that is approximately 65,400 square feet. Currently, the area consists of an asphalt parking lot and a large gravel and grass area enclosed by a chain link fence containing four greenhouses, a septic drain field (which will be abandoned prior to the start of construction), and several concrete block structures used for storage.
The staging area is bordered to the north and east by steep, near vertical rock slopes, and on the west by the Skagit River bypass reach. Vegetation on the site includes grass, trees, small brush, and ground cover on the steep slope between the site and the Skagit River bypass reach. The portal will be located at the base of the mountainside on the north side of the existing power tunnel access adit.

3.5.1 Portal Site Characterization

The portal area is characterized by numerous rock outcrops and varying amounts of talus (loose boulders) on the slope. The outcrops are cut by joints (fractures) and some of the outcrops have been undercut at the base to form overhanging blocks. Both adverse jointing (fracture planes dipping out of slope) and overhanging outcrops create potentially unstable blocks that could be dislodged during construction. The size and apparent stability of these blocks varies across the portal area. The slope is populated with trees.

3.5.2 Portal Alternatives

Six alternative portal locations were evaluated for the G2T. The locations of the portal alternatives were selected based the general criteria listed below:

- Maintaining portal safety by reducing potential for rockfall hazard
- Minimizing the cost of rockfall protection (slope stabilization) above the portal
- Reducing site impacts (tree removal)
- Initiating tunnel in competent bedrock with minimum rock cover for “turning under”
- Optimizing the initial alignment by reducing the curve and length of the starter tunnel, excavated by drill-and-blast methods, downstream of the connection to existing tunnel
- Providing contractor flexibility for portal staging layout

Figure 8 presents the six portal location alternatives that were evaluated. Criteria for horizontal alignment, tunnel length, and the downstream connection were also considered in portal selection and evaluation.

3.5.3 Portal Evaluations

The Portal 2 site was selected as the preferred portal location. Figure 9 provides a photographic view of the preferred portal area. The primary advantages of the Portal 2 site include:

- Falls outside of the trajectory of an extremely large rock block
- Reduced tunnel length
- Reduced length of horizontal curve required for the connection to the existing tunnel,
- Reduced rockfall mitigation efforts for the outcrop above the portal
- Located away from portal area fault

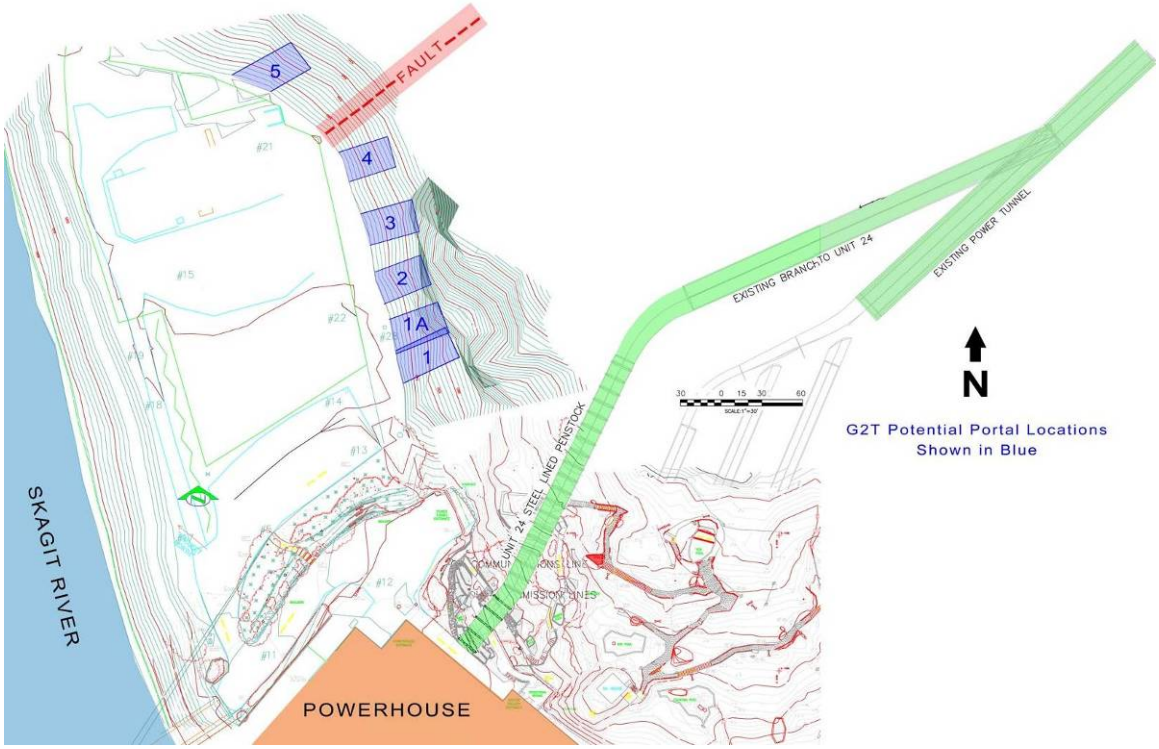


Figure 8. Portal Location Alternatives

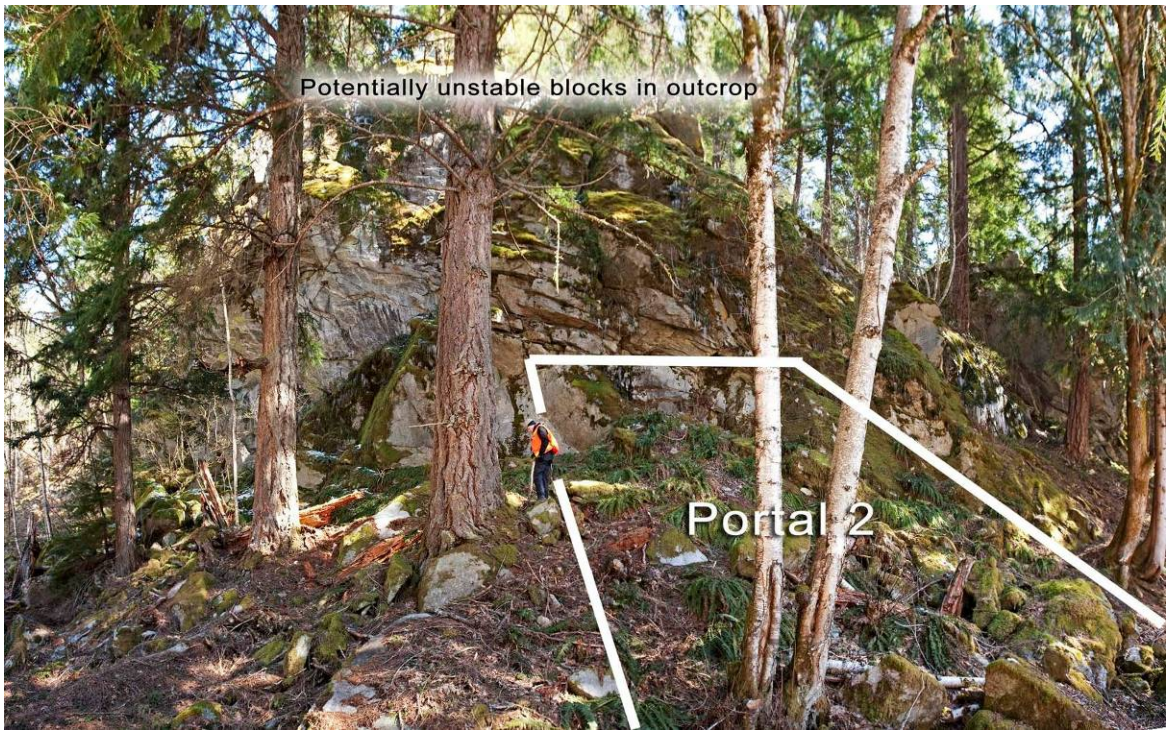


Figure 9. Portal 2 Location

3.6 Recommended Alignment

Based on the criteria and evaluations discussed in this section, the results of the hydraulic modeling discussed in Section 7, and the constructability, risk and cost evaluations discussed in Section 8, a recommended alignment was developed for the G2T. The alignment is summarized in Figure 10 with preliminary tunnel stationing. A set of full- and half-sized drawings of the recommended alignment are included in Appendix G.

Key features for the alignment are summarized in Table 1.

Table 1. G2T Alignment Summary

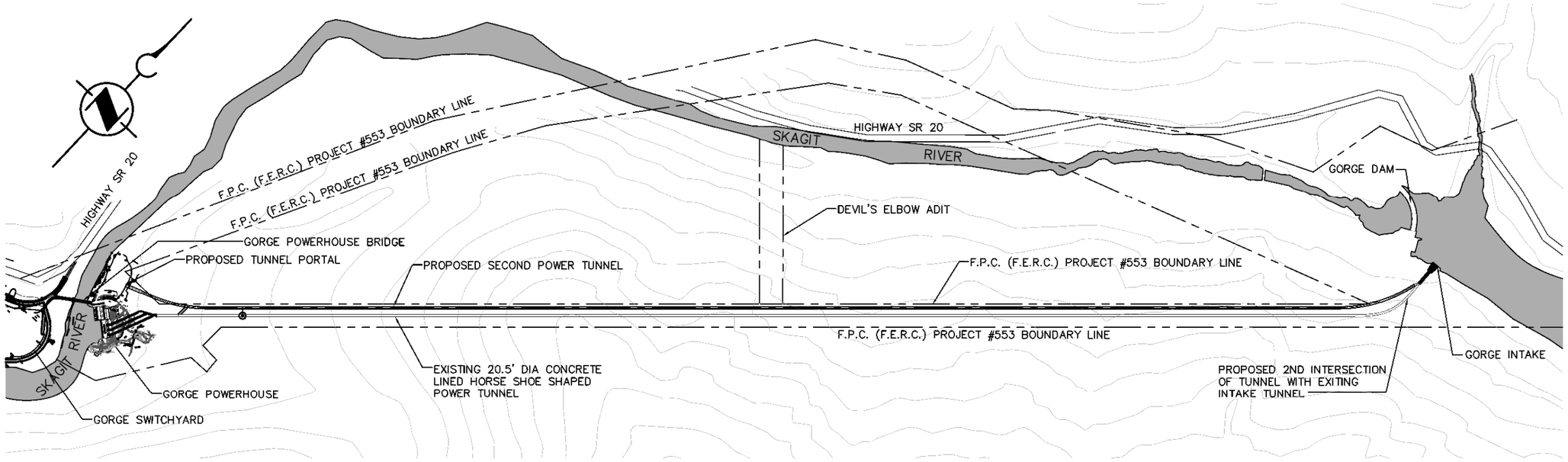
Length • *Excavated* • *Operating Power Tunnel*	 10,942 feet 10,500 feet
Upstream Connection • *Length* • *Elevation*	 29 feet 755 feet
Downstream Connection • *Length* • *Elevation*	 89 ft 530 ft
Horizontal Curve Radius	1,200 feet
Offset • *Existing Power Tunnel* • *Devil's Elbow Adit*	 50 feet 50 feet

3.6.1 Horizontal Alignment

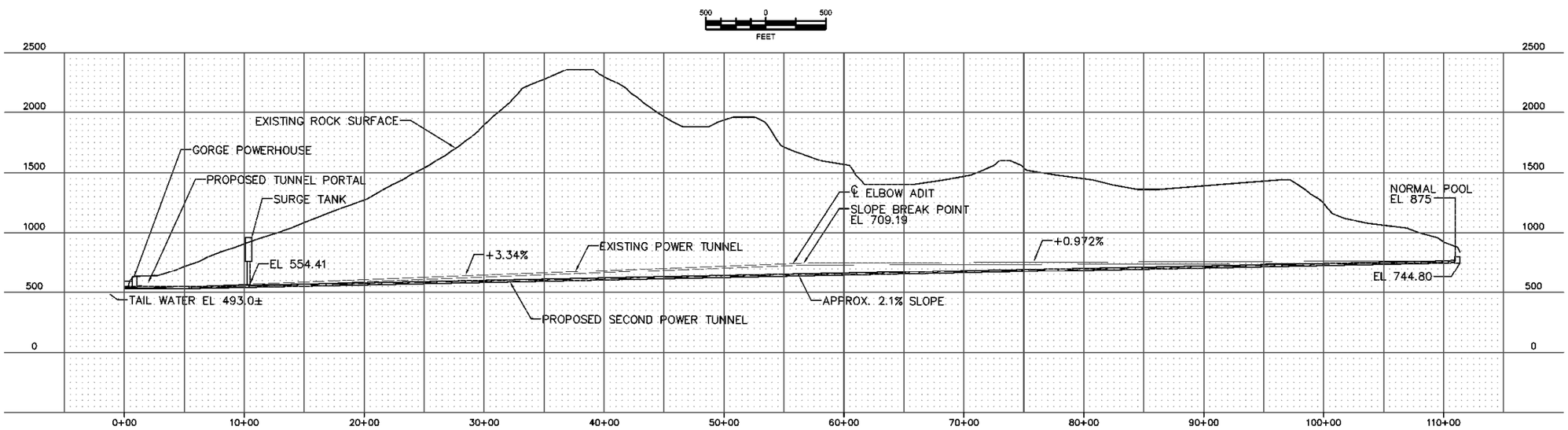
The recommended alignment stays within the FERC boundary as much as possible; however, the combination of horizontal curves and offset from the existing power tunnel pushes about 219 feet of the tunnel outside the FERC boundary at the downstream end (near Station 3+00) and at the upstream end. The impacted area is about 4,110 square feet.

3.6.2 Tunnel Grade

The G2T departs the proposed portal site (Portal Site 2) at a relatively flat grade of 0.36 percent and remains at that slope for approximately 500 feet. This grade places the G2T at the correct elevation for the downstream connection to the existing power tunnel (Link 3). Immediately after the downstream connection (at approximately Station 5+75), the tunnel grade increases to 2.08 percent for the remainder of the tunnel length until reaching the upstream connection. The combination of tunnel grade and grade lengths provides adequate separation between G2T and the Devil's Elbow Adit. It is anticipated that a slightly steeper grade will be necessary for the short upstream connection to match the existing tunnel elevation at the connection point.



GORGE 2ND TUNNEL – PLAN



GORGE 2ND TUNNEL – PROFILE

Figure 10. Preferred G2T Alignment

4 Tunnel Construction

The purpose of this section is to evaluate key tunnel construction methods, sequencing and staging requirements, and other considerations that will influence project configuration, hydraulic performance, and construction costs.

4.1 Tunnel Size

The tunnel cross-sections that were initially considered are 14-, 16- and 18-foot excavated diameters for a circular tunnel. Following the draft issue of this report, and the findings contained therein, larger tunnel diameters of 20 and 22 feet were also considered.

4.2 Proposed Tunnel Construction Method

A main beam TBM is considered the most efficient and economical method for excavating the tunnel. The TBM will provide relatively continuous excavation and support operation that can generally sustain higher overall rates of advance than drill-and-blast and roadheader methods. It will also provide a smooth, circular bore, which offers hydraulic advantages, particularly where the tunnel is left unlined, as well as some economic advantages for construction of the initial support and lining. One such advantage is that TBM excavations minimize rock mass disturbance, which reduces ground support requirements. The main disadvantages of a TBM are the high capital cost of the machine and a potentially long mobilization time. It is anticipated that drill-and-blast methods will be used at the portal to excavate a short enlarged tunnel section, referred to as a “starter tunnel,” to aid in the launch of the TBM and also for making the connections into the existing power tunnel.

A main-beam TBM consists of a cutterhead mounted on a longitudinal main beam, and is advanced by hydraulic rams supported by grippers between the main-beam of the machine and the rock wall. Initial support systems for a tunnel excavated with a main beam TBM are installed directly behind the cutterhead.

4.3 Preliminary TBM Performance Estimates

A TBM performance prediction model was used to evaluate TBM size alternatives and performance predictions to determine the appropriate penetration and advance rates, key design features, and procurement options for the machine. The estimation model is based on job site studies and statistics from more than 35 job sites and more than 135 miles of tunnel. The existing geotechnical data was further supplemented by the preliminary geological investigation e.g. field mapping and desk-top studies, carried out to date during preparation of this report. Best practices were used in determining the other TBM related parameters which generally agree with estimates obtained from TBM manufacturers when presented with the same preliminary geotechnical data.

The penetration rates obtained from the model for the range of TBM diameters and ground conditions anticipated ranges from 7.0 to 14.6 feet/hr; these ranges would provide favorable production rates for TBM excavation. The values obtained through the model compare favorably with the results provided by the TBM manufacturers Herrenknecht and Robbins.

Relatively good massive rock is expected, so there should be less of a need to stop for ground support installation and related activities. The intact nature of the rock mass and the few fractured or faulted zones should minimize water infiltration and related production slowdowns. However, cutter consumption is

expected to be above average since the rock is fairly strong with a typical quartz content greater than 20 percent. This increases downtime because of the need for frequent cutter changes and related cutterhead maintenance. Based on available information and site observations, the Skagit Gneiss is expected to be good boreable rock of medium strength and abrasivity.

Geotechnical information reviewed to date has been rather limited, and the data that was collected in the geotechnical investigation, carried out during the summer of 2009, will help to refine the inputs. Refined inputs will provide a better estimate of the penetration rates, cutter wear, and machine availability. The key geological parameters that need to be confirmed, aside from rock strength and abrasivity, are related to rock discontinuities such as dip angle of weakness planes and joint orientation and spacing.

4.4 Key TBM Features

TBM features were evaluated so that the costs of the different size TBMs are adequately captured in the cost estimates. Based on the anticipated ground conditions, the TBM requirements will likely include the following key features:

- Flat face, hard-rock cutterhead with hardened buckets and front-loading cutters
- 35-ton, 19-inch-diameter cutters
- A minimum of 36 cutters for a typical 18- to 20-foot-diameter TBM
- A minimum thrust of 1,260 tonnes
- A minimum cutterhead speed of 10.5 rpm
- A minimum cutterhead torque of 1,000 kip-ft
- Partial finger shield to permit installation of rock bolts
- Rib erector
- Probing and grouting capabilities
- Full integrated operators cabin with ergonomic controls

The TBM shown in Figure 11 is an example of a machine that incorporates most of the features listed above.

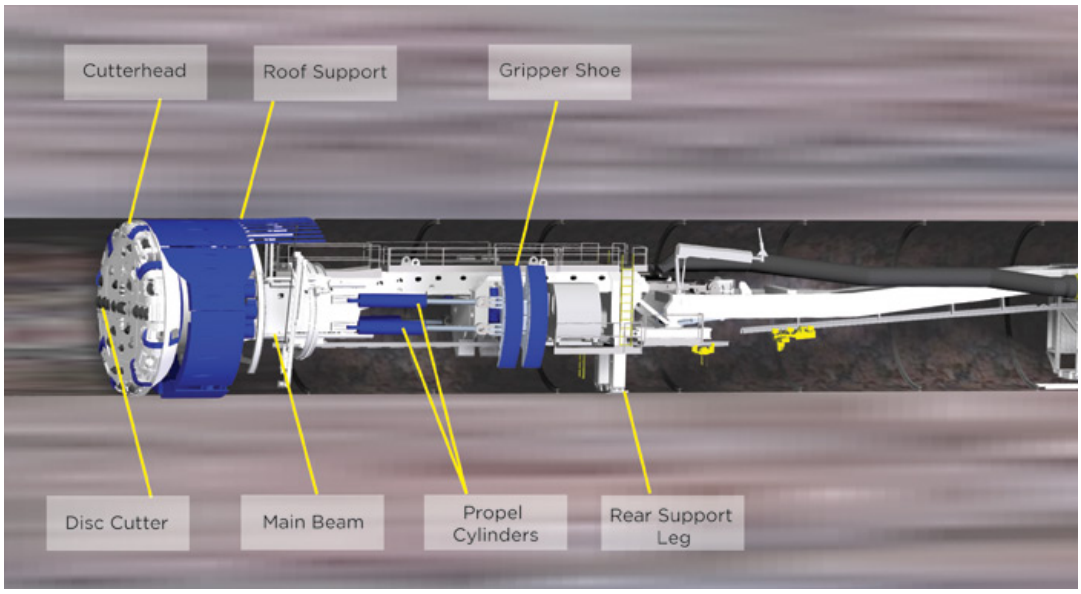


Figure 11. Main Beam TBM

4.5 Methods of Muck Removal

Excavation during tunnel construction will generate loose (i.e., bulked) materials. These bulked materials are commonly referred to as "muck." The two standard methods of muck removal for TBM tunnels are rail-mounted muck cars or a horizontal continuous conveyor belt. The choice of muck transportation methods will be primarily based on contractor preference and equipment availability.

4.5.1 Muck Characteristics

Muck from the TBM excavation is anticipated to consist of flat gravel-sized pieces of rock, commonly referred to as "chips," and sand. A range of estimated gradation is presented in Table 2. Material generated during drill-and-blast excavation is anticipated to be coarser than TBM-generated material, containing cobble-sized and possibly boulder-sized pieces. Ideally, maximum rock size will be limited to three feet. However, the size of blasted material is a function of blast-hole spacing and powder factor, in addition to rock type and quality (joint spacing and condition).

Table 2. Estimated Muck Gradation

Equivalent Sieve Size	Percent Passing
3 to 6 inch	95 to 100
3 inch	95 to 100
3/4 inch	65 to 90
#4	30 to 55
#10	20 to 35
#40	10 to 25
#200	5 to 25

U.S. Department of Transportation, 1984.

4.5.2 Estimated Muck Quantities

The G2T is anticipated to have an excavated diameter of between 14 and 22 feet and a length of approximately 11,000 feet. The excavation profile for the starter tunnel and downstream and upstream adit connections will generally be horseshoe shaped and will be 2 to 4 feet wider and taller than the TBM tunnel diameter to allow for access around the TBM (starter tunnel), or for placement of a cast-in-place concrete lining (downstream and upstream connections).

Table 3 provides preliminary estimates of in-place and bulked spoil volumes for the G2T for various excavated diameters. A bulking factor of 1.8 was assumed for TBM excavation methods. No allowance was made in the table for the short drill-and-blast section of adit—this excavation method typically has a bulking factor of around 1.6. These bulking factors are based upon recent observation of other tunnel projects in similar rock conditions.

Table 3. Estimated Muck Volumes

Tunnel Diameter (feet)	In-Place Volume (cubic yard)	Bulked Volume (cubic yard)
14	62,750	112,950
15	72,000	129,600
16	81,950	147,500
17	92,500	166,500
18	103,700	186,650
20	128,000	230,400
22	154,900	278,800

4.5.3 Storage and Transportation of Muck

Muck will be transported to a site that has been identified by SCL. The site is located next to Highway 20, approximately 10 miles southwest of the G2T portal (see Figure 12). The site is an old abandoned quarry known as the Bacon Creek Restoration Site. Excavated material will be used to fill and grade the site as part of a Wildlife Rehabilitation Project.

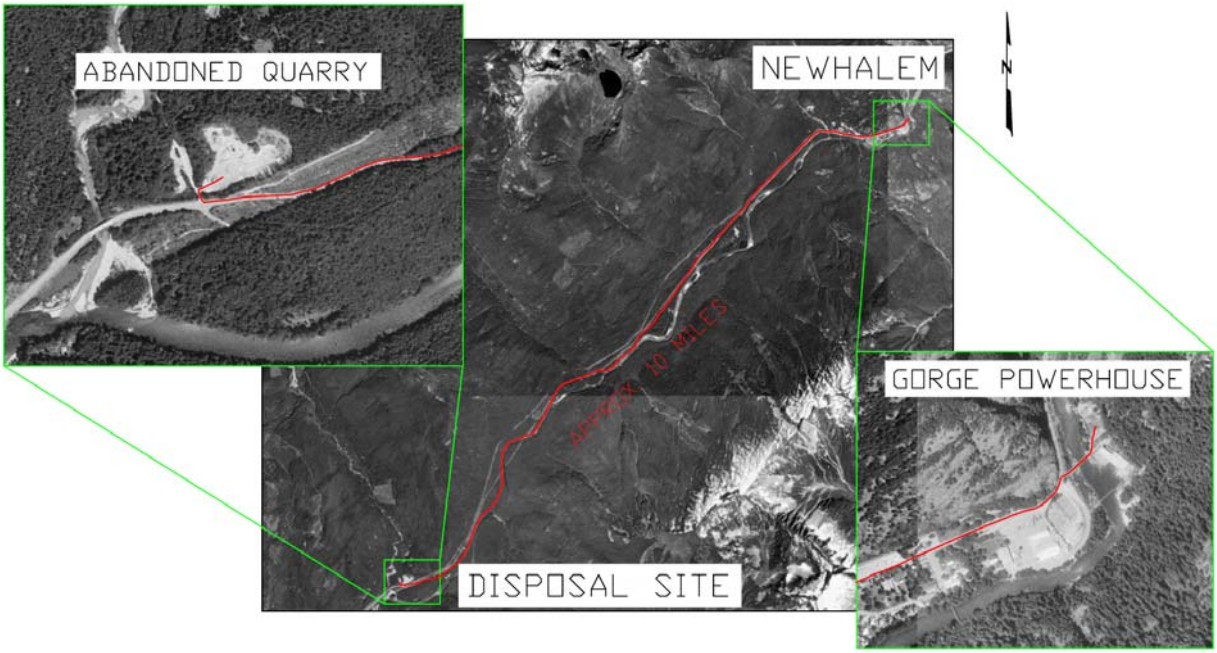


Figure 12. Muck Disposal Location

An on-site storage area will be required to temporarily stockpile muck before haul trucks transport the muck to the Bacon Creek restoration site. It is anticipated that Haulage to Bacon Creek will typically only occur during daytime working hours and this temporary stockpile area will need to be sized accordingly. Because of the relatively small available work area at the portal site (located north of the powerhouse), it is proposed that the temporary muck storage site be located across the Skagit River from the powerhouse, adjacent to the existing switchyard. This area is currently a gravel lot used for visitor parking. The muck would be transported from the tunnel entrance to this location via a conveyor belt traveling across or adjacent to the existing powerhouse bridge. Temporarily stockpiling the muck in this location has two main benefits. First, it frees up valuable room at the portal site so tunneling activities, including material deliveries, can proceed in a more efficient manner. Second, the proposed area next to the switchyard has much better truck access for muck removal than does the portal site, allowing the muck to be removed from the temporary site more efficiently, and preventing haul truck movements from blocking access to the powerhouse or the powerhouse bridge. Because of the proximity of the Gorge Switchyard to this proposed temporary stockpile, dust mitigation will be particularly important. Mitigation measures will be addressed during Final Design and are likely to include damping for dust suppression, enclosed disposal areas, and barrier walls.

Table 4 presents estimates of the average amount of bulked muck that will be generated daily. This represents the volume of material that will need to be stockpiled, transported, or placed on a daily basis. The quantity of material is a function of the advance rate of the TBM. The day to day advance rate of the TBM will vary, but for spoil handling estimating purposes an average advance rate of approximately 70 feet per day has been assumed.

Table 4. Muck Generation per Day

Tunnel Diameter (feet)	Bulked Muck Volume (cubic yard)
14	720
15	825
16	940
17	1,060
18	1,190
20	1,470
22	1,775

It is reasonable to assume that occasionally up to two days worth of muck may be present at the temporary storage area, depending upon the coordination of mining and muck removal schedules. This equates to a maximum volume at the temporary muck storage site of about 1,450 to 3,550 cubic yards, depending upon tunnel diameter. Table 5 summarizes the estimated number of trucks per day for tunnel diameters ranging from 14 to 22 feet. These estimates assume 20-cubic-yard-capacity trucks hauling an average of 14 cubic yards per trip.

Table 5. Muck Trucks Trips per Day

Tunnel Diameter (feet)	One-Way Truck Trips per Day
14	52
15	60
16	68
17	76
18	85
20	105
22	127

4.5.4 Natural Contaminants

Natural contaminants may be present in the Skagit Gneiss and therefore may be present in the tunnel muck. Gneiss is believed to contain less than one percent sulfide minerals (pyrite, marcasite, and pyrohotite). This information is based on bedrock observations during the 1954 geologic investigation for the Gorge High Dam. Iron staining, which is commonly observed in fault zones and major fractures along the proposed G2T alignment, is thought to be caused by the weathering of these sulfides and other dark, iron-rich minerals. The presence of sulfides along the G2T alignment will be evaluated using rock core samples obtained during the geotechnical investigation program.

Arsenic occurs naturally in biotite (black mica), pyrite, and other sulfide minerals that are present in the Skagit Gneiss along the proposed G2T alignment. According to a review of the scientific literature, arsenic is soluble in reducing, high pH (>7.5) conditions, but is relatively insoluble in oxidizing, neutral to lower pH (>7.5) conditions, which are the expected conditions within the tunnel excavation and the muck storage area. Based on these expected conditions, arsenic concentration is not expected to exceed background levels within tunnel muck or discharged groundwater.

The Skagit River, which is fed by snow melt, rainfall, and groundwater, already flows across the Skagit Gneiss bedrock and is expected to contain background levels of dissolved arsenic. The background arsenic level is not expected to be significantly increased by tunnel muck and discharge water, nor is the pH of the discharge water expected to be reduced significantly due to oxidized sulfides. Testing of muck and groundwater prior to discharge is recommended to confirm low arsenic concentrations and neutral pH conditions.

5 Tunnel Support

The initial support for the tunnel consists of the ground support elements designed, furnished, and installed by the contractor to support the ground, maintain excavation stability, and provide a safe working environment for the tunnel excavation and final lining work. In some cases, initial support can be incorporated into the tunnel final lining. This section describes the anticipated tunnel support systems and provides an estimate of the quantities of each support type to be used in the development of the construction cost estimates.

5.1 Feasible Initial Support Systems

For a TBM excavation, appropriate initial support methods include rock reinforcement and steel ribs with varying amounts of lagging. Additional surface treatment such as steel straps and wire mesh could also be used as necessary to stabilize the tunnel. Shotcrete could also be required to protect the section of the tunnel where exposed rock is susceptible to deterioration after excavation. Considerations for the initial support systems are discussed in more detail in the following subsections.

5.1.1 Rock Reinforcement

Rock reinforcement includes the use of rock bolts, rock dowels, and/or rock anchors and may be used in conjunction with surface treatment such as steel straps, wire mesh, and shotcrete. The generic term “rock bolt” is used herein and no distinction is made between rock dowels or tensioned rock anchors. Rock bolts can be resin or cement grouted rock dowels, mechanically anchored rock bolts, and friction rock bolts. Rock bolts can also be coated to provide a level of corrosion protection for long-term durability. Typically, a rock bolt length equal to one half the tunnel span is required for a tunnel span less than 20 feet and typical spacing for a pattern of rock bolts is about one half the bolt length.

5.1.2 Steel Rib Supports

Steel ribs consist of wide flange beams bolted together at butt plate connections to form a complete rib. Ribs can be fabricated in sections to provide full circular or horseshoe shaped support elements. Lagging can be installed to support the ground between adjacent ribs in poor quality rock or localized shear zones. Lagging options include timber, steel channels, liner plates, and shotcrete. Steel ribs are particularly effective in drill-and-blast tunnels as the spacing can be easily modified to suit the ground conditions. Bracing between ribs is generally provided by tie-rods and collar braces to prevent shifting of ribs and posts. Steel ribs are considered to be an effective support method in poor quality, closely jointed rock.

5.1.3 Shotcrete

Shotcrete consists of concrete that is sprayed onto the rock surface to stabilize exposed rock. It can be used in tunnels and surface excavations. A pump is used to deliver the concrete mix pneumatically at high velocity through a hose and nozzle to the area being stabilized. Two basic types of shotcrete are used; wet mix and dry mix. Water is added at a batch plant for wet mix shotcrete, while in a dry mix, the water is added at the nozzle. A typical shotcrete mix consists of cement, fine aggregate, and water with additional admixtures (silica fume, superplasticizers, or accelerators) to produce the desired mix. In tunnels, various types of fibers can be used in the mix to provide reinforcing, or wire mesh can be installed to the area being treated and shotcrete can be applied through and over the mesh. Shotcrete is widely used as an initial support system in drill-and-blast and roadheader operations but is less common in TBM tunnels

because tight space constraints and other activities make it difficult to apply it close enough to the tunnel heading to be fully effective.

5.2 Ground Characterization

Estimates of ground conditions are needed for evaluation of tunnel initial support systems. The ground (or rock) loads that must be supported by the initial support systems are a function of the rock quality, stresses induced in the ground by tunnel excavation, orientation and spacing of discontinuities, and the span, or diameter, of the tunnel.

A preliminary characterization of ground conditions was made to estimate the initial support requirements for the G2T. These initial evaluations are generally based on anticipated Terzaghi's ground condition categories for rock tunnels (Terzaghi, 1946). More detailed ground characterization evaluations (using the Q, Rock Mass Rating, and other applicable systems) will be conducted during Final Design once more geotechnical data becomes available. Table 6 summarizes the basic ground conditions definitions developed by Terzaghi and modified by Proctor and White, and Deere.

Table 6. Relevant Definitions of Ground Condition Terms
Modified from Terzaghi (Proctor and White, 1968), and Deere et al. (1969).

Firm	Rock or soil is a material which will stand unsupported in a tunnel for several days or longer.
Hard, stratified	Rock consists of individual strata with little or no resistance against separation along the boundaries between strata. The strata may or may not be weakened by transverse joints. In such rock, the slabbing and spalling conditions are common.
Massive, moderately jointed	Rock contains joints and cracks, but the blocks between joints are in contact and are intimately interlocked.
Blocky and seamy	Rock consists of chemically intact or nearly intact rock fragments, separated from each other by joints or other discontinuities that are imperfectly interlocked. In such rock, vertical surfaces may require support. When individual blocks are larger than one foot, the rock is called moderately blocky and seamy; when blocks are smaller than one foot, the rock is called very blocky and seamy.
Crushed rock	Rock consists of chemically intact or nearly intact rock fragments, separated from each other by joints or other discontinuities that are imperfectly interlocked. In such rock, vertical surfaces may require support. When individual blocks are larger than one foot, the rock is called moderately blocky and seamy; when blocks are smaller than one foot, the rock is called very blocky and seamy.

Generally, the Skagit Gneiss at tunnel depth is anticipated to be fresh, strong to very strong, and massive to only slightly fractured. The rock is expected to exhibit characteristics similar to three of Terzaghi's categories for rock tunnels: hard and intact; massive, moderately jointed; and moderately blocky and seamy rock. The blocky and seamy rock is expected to be associated with weathered rock in shear zones.

The preliminary rock quality descriptions that correlate to these ground conditions terms are good to very good rock, fair rock, and poor rock.

5.3 Anticipated Initial Support Requirements

Rock reinforcement is considered to be a feasible and cost-effective initial support method for the G2T, except where weathered or highly fractured rock (also termed blocky and seamy rock above) is encountered. Rock reinforcement will most likely consist of mechanical or friction dowel systems because of the rapid installation time for these types of elements. Occasional use of steel channels, steel straps, and wire mesh is anticipated in conjunction with the rock bolts for surface treatment to stabilize poor quality, and closely jointed rock. In weathered rock or highly fractured rock, steel ribs would be needed for initial support. Shotcrete could be used in conjunction with either rock reinforcement or steel ribs where drill-and-blast methods are utilized. Full-perimeter lagging or shotcrete would be required between ribs in crushed rock that could be encountered in occasional shear or fault zones. Where overbreak is encountered, the cavity outside the limits of the steel rib and lagging support will need to be stabilized to prevent further deterioration.

Initial support requirements will vary along the tunnel because of the range of ground conditions expected during construction. A preliminary assessment of the likely tunnel initial support for the G2T is summarized in Table 7. These estimates will mainly serve as a basis for the construction cost estimates.

Table 7. Preliminary Estimate of Initial Support Requirements

Anticipated Rock Mass Quality (Ground Condition)	Initial Support Type	Percentage of Tunnel
Good to very good rock (massive to moderately jointed)	Occasional or as needed friction rock bolts (1 bolt per 10 feet of tunnel)	85%
Fair rock (moderately blocky and seamy)	Pattern friction rock bolts (3.5 per 5 feet of tunnel)	13%
Poor rock (very blocky and seamy to slow raveling)	Shotcrete lagged, full perimeter steel ribs (4 foot spacing)	2%

6 Tunnel Lining

The tunnel final lining will be designed to provide a durable and smooth interior surface that minimizes hydraulic head losses, controls leakage from the tunnel, maintains a high level of serviceability, and minimizes maintenance during its design life. In addition, the final lining must be designed to withstand the internal water pressures, ground loads, and external water pressures as applicable. This section presents the considerations and approach taken for the G2T lining design,

The internal pressures within the tunnel and the frictional hydraulic head losses are key design considerations for the G2T lining. The internal pressures are based on a static hydraulic grade line created by the maximum reservoir level at elevation 875 feet. Net internal pressures (static internal pressure minus external groundwater pressure) will be evaluated when additional groundwater level data becomes available.

6.1 Feasible Tunnel Lining Systems

Typical tunnel lining options for a power tunnel of this size are cast-in-place (CIP) reinforced concrete, shotcrete, and where rock conditions and hydraulic conditions are favorable, an unlined tunnel. The applicability of these lining options is discussed in the following subsections.

6.1.1 Unlined Tunnel

When subjected to internal hydrostatic pressures an unlined tunnel will develop excessive leakage if there is inadequate confinement provided by the rock mass and external groundwater pressure to safely contain the water pressure and prevent hydraulic jacking. Hydraulic jacking occurs when the internal pressure inside the tunnel exceeds the minimum in situ stress in the rock mass acting across a discontinuity, or when the internal pressure exceeds the sum of the minimum principal stress and tensile stress of the rock. Under these conditions, existing discontinuities (joints, bedding planes) dilate or new fractures form resulting in a dramatic increase in the hydraulic conductivity of the rock mass. This normally results in unacceptable leakage rates from a pressure tunnel.

A lining is required in areas with insufficient rock cover to satisfy confinement requirements and prevent hydraulic jacking which may cause excessive leakage. A lining is also recommended where the ground is either highly deformable (i.e., low modulus) or has a high permeability (i.e., the rock mass is either pervious or highly fractured).

The required length of the lining in the tunnel is based on an evaluation of the minimum cover that would provide adequate confinement to prevent hydraulic jacking. The preliminary evaluation of confinement requirements for the tunnel is based on the topography and rock cover above the tunnel. Adequate rock cover to avoid excessive leakage due to hydraulic jacking was determined using the Norwegian method (Brekke and Ripley 1986). Results indicate that sufficient rock cover exists for the proposed G2T alignment and hydraulic jacking is therefore not expected to occur.

6.1.2 Cast-in-Place (CIP) Reinforced Concrete

Construction of a CIP-reinforced concrete tunnel lining involves the erection of specially made forms inside the tunnel, pumping of the concrete through a slick line extended from the pumping location to the forms, stripping and removing the forms after the concrete has set, and contact grouting of any voids outside of the lining. For constructability reasons, a minimum lining thickness of 8 to 12 inches is usually

specified to allow a good flow of concrete around the forms during placement and to account for variation in the erection tolerances applicable to the initial supports. Low-pressure, contact grouting will be required to fully fill any voids behind the concrete lining.

Design of the concrete and reinforcing steel for the lining will be primarily based on evaluations of external and internal pressures and leakage. Similar to shotcrete, properly distributed reinforcing steel can distribute strain resulting from concrete shrinkage along the tunnel, thereby minimizing the width of any individual shrinkage crack, which will limit tunnel leakage.

Installation of a CIP lining in a TBM tunnel will locally reduce the hydraulic diameter of the tunnel resulting in some head loss. Additional evaluations on the sensitivity of the hydraulic performance of the tunnel to sections of reduced diameter will be performed during Final Design. Alternatives to a CIP lining will be evaluated such as a steel lining will be considered as necessary to reduce head loss in the tunnel.

6.1.3 Shotcrete

The shotcrete lining would consist of a continuously applied, full circumference shotcrete lining. The shotcrete would be reinforced to control crack widths and improve the flexural strength of the lining. Reinforcement would be provided by the addition of fibers to the shotcrete mix or by applying unreinforced shotcrete over wire mesh. It is anticipated that the minimum shotcrete thickness would be approximately two inches. Systematic use of shotcrete linings is not desirable for power tunnels because it would result in higher friction values, thereby increasing head loss.

It is anticipated that shotcrete will be applied in areas that are supported by steel ribs and mesh. If used as a permanent lining the shotcrete layer would need to be built up to a minimum of three inches over the inside surface of the steel rib or mesh to adequately protect the steel from corrosion.

Shotcrete linings could also be used to line sections with complicated shape transitions that are difficult to form such as those anticipated for the downstream and upstream connections. In these cases, mesh or fiber would be required to reinforce the layers of applied shotcrete. Shotcrete transitions with CIP concrete would be constructed by feathering the shotcrete against the edge of the concrete.

6.2 Anticipated Tunnel Lining Requirements

The majority of the rock is anticipated to be strong and durable enough to resist long term deterioration. Therefore, relatively “light” tunnel support is expected and the tunnel, at this design stage, is assumed to be predominantly unlined. It is anticipated that a concrete or shotcrete lining will be required in very blocky and seamy rock. This lining will be designed to support the full ground loads and will also protect and prolong the life of the steel components of the initial support system (i.e., ribs). A CIP reinforced concrete lining will likely be required to resist ground loads in crushed zones associated with shears zones. The lining types, estimated lengths and percentage lengths are summarized in Table 8.

Table 8. Anticipated Tunnel Lining Requirements

Final Lining Type	Length of Tunnel (feet)	Percentage of Tunnel
Unlined	9,750	90
Upgraded Rock Reinforcement	300	3
CIP Concrete	800	7

During tunnel construction, local instabilities within the tunnel may manifest behind the TBM as the rock stress is relieved. It is anticipated that in some areas, permanent rock bolt support systems will be required to upgrade intervals of rock reinforcement support for permanent support in order to allow sections of the tunnel to remain unlined. Upgraded support is anticipated to consist of a pattern of CT-bolts. This type of bolt provides immediate support using a mechanical anchor and is later grouted to provide permanent support. CT-bolts have a polymer sleeve around the steel bolt that provides double corrosion protection for a permanent support application.

Localized treatment of “dental” concrete is also anticipated to smooth the hydraulic profile in areas with rock block fall-out or erosion of weathered material from discontinuities.

6.3 Tunnel Friction

Friction factor is a key parameter in calculating the frictional head loss for the design of hydropower tunnels. Frictional head loss was determined using the theoretical Darcy Weisbach friction factor and the empirical coefficient of friction (Manning number). Existing information on surface roughness and friction losses in rock tunnels from past projects was also reviewed and compared to calculated values. A summary of the friction evaluation is provided in the following subsections. Full details of the friction evaluation are provided in the project memorandum, *Gorge 2nd Tunnel Friction Coefficient* (Jacobs Associates, February 2009).

6.3.1 Existing Information

Existing data from unlined, TBM-bored tunnels were reviewed to help determine the friction factor for use in the G2T hydraulic model for various lining options. Projects reviewed include several in South Africa (Pennington, 1998) and two projects in the Sierra Nevada Mountains of California. Additional sources of friction loss information were also reviewed. Table 9 summarizes recommended friction values from the Army Corp of Engineers (U. S. Army Corps of Engineers [USACE], 1997) and the Electric Power Research Institute (Brekke, 1987).

Table 9. Typical Friction Values

Source	**Manning Number**
USACE	
TBM, Unlined	0.018
Cast-in-Place Concrete	0.013
Drill-and-Blast, Unlined	0.038
Electric Power Research Institute	
Drill-and-Blast, Unlined	0.027–0.041
TBM, Unlined	0.010–0.02
Shotcrete	0.020–0.025
Cast-In-Place Concrete	0.013
Steel	0.013

6.3.2 G2T Friction Calculations

Darcy–Weisbach friction factors were calculated for G2T using unlined tunnel surface roughness values ranging from 1/16 to 3/32 inches and excavated diameters ranging from 16 to 20 feet. The calculated Manning number ranges from 0.0143 to 0.0152. The results are summarized in Figure 13.

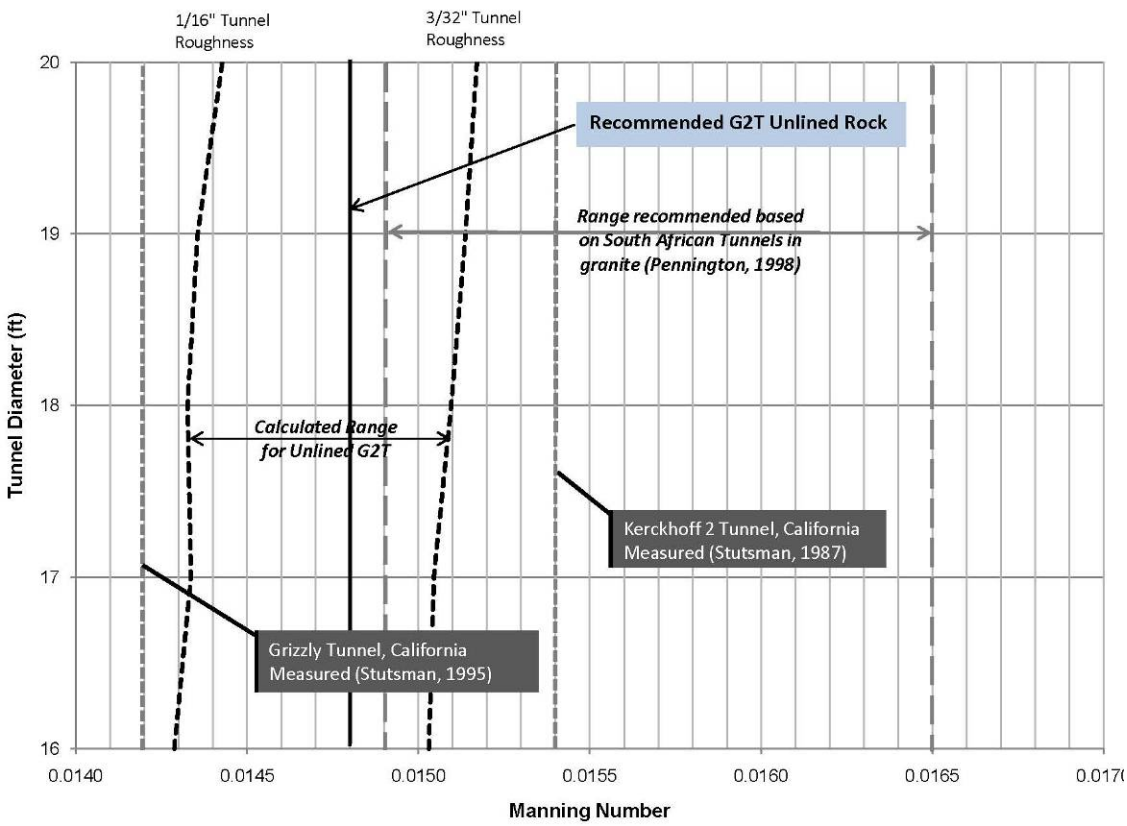


Figure 13. Rock Friction Factors for TBM-bored Tunnel

There is general agreement among calculated, measured, and guideline values of friction. Recommended Manning numbers for the G2T hydraulic model are summarized in Table 10 for various tunnel linings. As indicated in Figure 13, the recommended friction value for G2T for unlined rock is on the lower end of the range of the collected data from previous projects; however, this assumption is considered appropriate given modern tunneling techniques and the similarity between the rock at the Grizzly Peak project and the G2T project.

Table 10. Recommended Manning Number

Lining Type	Manning Number
Unlined Rock	0.0148
Cast-In-Place Concrete	0.010
Shotcrete	0.0175

7 Hydraulic Modeling and Transient Study

7.1 Purpose

The Preliminary Water Transient Study report summarized the initial friction loss (head loss) and load rejection transient results for the G2T system. The hydraulic modeling undertaken included the existing tunnel configuration (G1T), which served as the baseline case, together with four alternative downstream connection options (links) for the proposed second tunnel. The model simulations were run using HDR/DTA's PTURB hydraulic analysis and transient program. The objectives of these analyses were:

- Evaluation of the friction loss reduction for the proposed G2T alternatives;
- Evaluation of transient impacts for the proposed G2T alternatives;
- Evaluation of the need for an additional surge chamber for the proposed G2T alternatives;
- Evaluation of station power outputs for a maximum flow of 7,700 cfs; and,
- Revised frictional loss coefficients for input into Seattle City Light's energy model.

The results of the hydraulic modeling provide the initial step in the tunnel optimization process. The optimization process includes the following components:

- **Friction Loss / Transient Evaluation.** Friction loss evaluation for G2T alternatives to produce revised friction loss coefficients for input into the Seattle City Light (SCL) energy model. SCL will run comparative energy model cases with the current configuration (G1T) and with the new tunnel (G2T) configuration. The preliminary transient results indicate that a second surge chamber is not required.
- **Tunnel Cost Evaluation.** Preliminary tunnel construction costs were developed as presented in Section 10 of this report for the preferred tunnel configuration and the various tunnel sizes under consideration.
- **Energy Model / Revenue Evaluation.** The power generation benefit and associated revenue for the different tunnel sizes will be performed by SCL. This step determines the net benefit of each tunnel alterative under consideration. This step is being performed independently of the Design Team's effort and will follow after the final draft issuance of this report.
- **Tunnel Optimization.** The tunnel optimization includes the three components and respective inputs described above. An overall benefit / cost evaluation for each alternative being considered will be performed. The tunnel optimization will be performed by the Design Team in conjunction with SCL staff once the generation and revenue benefit analysis has been completed. The recommended tunnel diameter and configuration will be recorded in the final report.

7.2 Maximum Flow Assumptions

The hydraulic modeling discussed in this section considers maximum station flows of 7,440 cfs and 7,700 cfs. The station flow is restricted by the 1995 Water Rights agreement downstream of the Gorge Powerhouse which currently limits the maximum station output to 7,440 cfs (this figure has since been confirmed to actually be 7,500 cfs). A station flow of 7,700 cfs was evaluated for comparison purposes because it represents the upper limit for all four turbines combined at the Gorge Powerhouse as stated in the June 2008 Planning Document for the project.

7.3 Description of Alternative Connections

Four downstream connection alternatives (Links 1, 2, 3, and 4) were developed and included in the initial transient analyses. Figure 14 shows the conceptual layout of these four downstream connections, also referred to as "Links."

All intersections between the G1T and the proposed G2T are assumed to be at a 45 degree angle. Links 1 and 2 connect directly with the Unit 24 branch tunnel downstream of the Unit 24 bifurcation point. While Link 1 is almost a direct connection to the Unit 24 penstock, Link 2 moves the connection point further upstream towards the bifurcation point. Links 3 and 4 are located upstream of the Unit 24 bifurcation, with Link 3 connecting just upstream of the bifurcation point, and Link 4 connecting upstream of the existing surge tank.

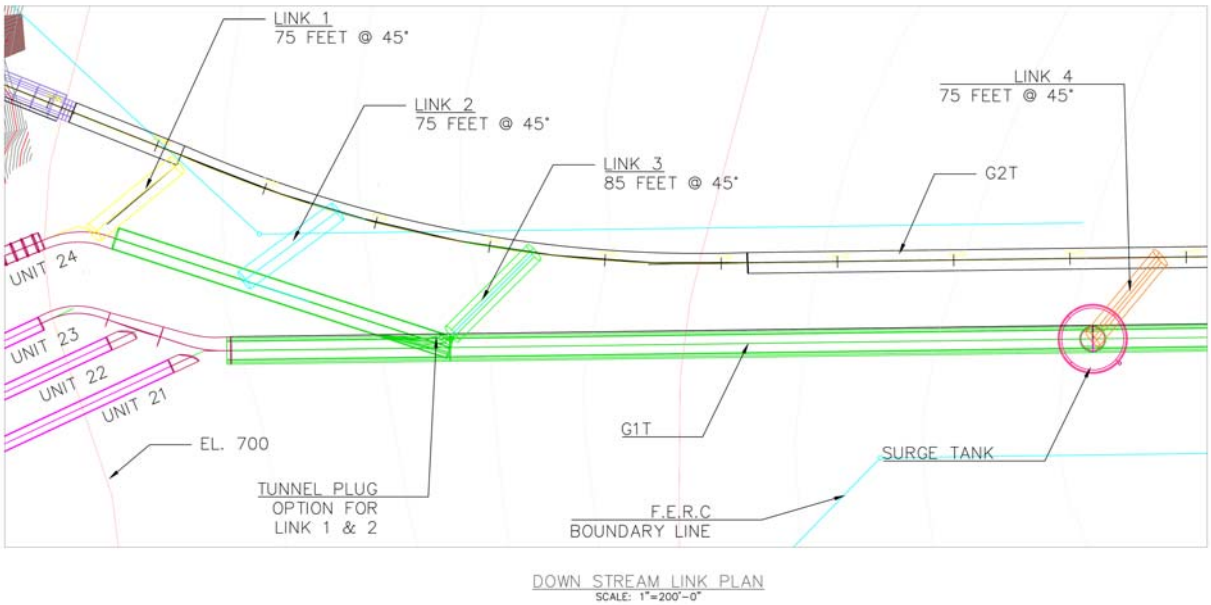


Figure 14. Downstream Connection Alternatives

7.4 Description of Hydraulic Analysis

These four downstream connection alternatives, together with tunnel diameters of 14 feet, 16 feet, and 18 feet, were modeled for hydrodynamic performance. This resulted in 12 tunnel configuration alternatives. The PTURB program was used to evaluate hydraulic losses (head loss) and to perform hydraulic transient analyses for this suite of 12 tunnel alternatives. The predicted output with an 18-foot-diameter tunnel was below the desired station capacity of 207 MW, so three additional larger diameter tunnels of 20, 22, and 25 feet were evaluated at station flows of 7,440 cfs and 7,700 cfs.

The PTURB program can evaluate any type of water conveyance system configuration including various intakes, lined and unlined tunnels, penstocks, penstock bifurcations, trifurcations and manifolds. The hydraulic model is input as a series of interconnected pipe and pipe junctions to form a hydraulic network. The hydraulic model can solve steady-state or transient conditions. Form loss, or minor loss, friction factors used with the PTURB program are calculated from the *Corps of Engineers Hydraulic Design*

Criteria and from the *Handbook of Hydraulic Resistance*. Form-related head losses would include conveyance features such as bends, contractions, wyes, etc. Minor losses would include items such as trash racks, entrance and exits losses, valves, orifices, etc.

The PTURB program uses the well-known Darcy Weisbach friction formula in the format $H_L = f\ (L/D)\ V^2/\ 2\ g$, where:

H_L= head loss in feet
f = dimensionless friction factor
L = pipe length in feet
D = pipe diameter in feet
V = the velocity in the pipe in ft^2/sec

In order to derive a friction factor "f" for each pipe in the PTURB program, the form loss or minor head losses are added to the straight conduit friction losses. An equivalent friction factor "f" is then derived based on the individual pipe length, which accounts for all frictional loss components.

Table 11 presents a summary of the input parameters used in the development of the configuration alternatives.

7.5 Load Cases and Combinations

The hydraulic modeling simulated current total systems flows of 1,000, 2,000, 3,000, 4,000, 5,000, 6,000, 7,000, 7,440, and 7,700 cfs, with unit operation based on the Gorge dispatch model. Transient cases were run with four-unit, simultaneous, load rejections (load-off) at station flows of both 7,440 and 7,700 cfs to simulate a plant separation from the transmission system. This load case generates the highest normal turbine and penstock pressure rise associated with the various tunnel alignment alternatives. More than 125 various hydraulic load cases were analyzed. A summary of the hydraulic load cases that were modeled is included as Appendix M.

Table 11. Summary of Hydraulic Model Parameters

	Existing Power Tunnel (G1T)	**G2T**
Tunnel Physical Characteristics		
Length	10,784 feet (Intake bend to Unit 24 bifurcation)	11,000 feet (Nominal) (Diverging bifurcation to converging bifurcation)
Diameter Cross section	20.5 feet, Horseshoe	14 to 22 feet, Circular
Darcy Weisbach friction factor	0.0100, concrete lined	0.0148, TBM unlined 0.0100, concrete lined
Tie-ins		4 links connected at 45° angle to G1T, concrete lined

	Existing Power Tunnel (G1T)	G2T
Hydraulic Model Inputs		
Reservoir Water Surface Level	Normal maximum water surface elevation, Elev. 875	
Tailwater Water Surface Level	Normal water surface elevation, Elev. 494.5	
Generating Flow	Varied: 1,000 cfs to 7,700 cfs	
Turbine/Generator	Turbine diameter, turbine speed, moment of inertia (WR^2), distributor centerline, turbine hydraulic characteristics*	
Turbine Shutoff Valves	Diameter, centerline, discharge characteristics	
Wicket Gate/Governor	Closing rate, type of stroke, cushion, timing	
Pressure Relief Valve	Discharge characteristics, servo timing	
Surge Chamber	Main diameter 57.5 feet, orifice diameter 18 feet, orifice coefficients—inflow 0.68, outflow 0.90	

**Because of the lack of available model test data for the Gorge turbines, the Gorge Powerhouse turbines have been modeled using a representative turbine data file of similar specific speed.*

7.6 Modeling Calibration

Unit performance testing carried out in 1993 provided the baseline turbine flow data (Seattle City Light, 1993). At that time, the flow testing indicated a maximum flow rate of about 7,876 cfs. This flow testing was conducted prior to the Unit 24 runner replacement in 2006.

An initial hydraulic baseline model for the current configuration (G1T) was developed to evaluate friction factors based on flow and head losses from the 1992–1993 testing. The single tunnel (G1T) hydraulic model was calibrated to flow test data collected by SCL in 1993. This initial single tunnel (G1T) hydraulic model formed the basis for subsequent hydraulic models incorporating the various second tunnel (G2T) configurations.

SCL staff indicated that no load rejection testing had been conducted following the 2006 runner replacements for any of the units. Load rejection testing was conducted on May 12 and 13, 2009, to obtain calibration data for the transient model relative to Unit 24 hydraulic response during a load rejection event. This testing included two partial load tests for Unit 24 at 38 MW and 60 MW to provide calibration data points for the model. Units 21 and 22 were operating at 10 MW for these tests. No multi-unit load rejection tests were performed due to plant load and flow restrictions for the time of year. Full details of these load rejection field tests are presented in HDR/DTA memorandum dated May 22, 2009, included as Appendix K. The pressure transducer installed in the surge tank to measure the maximum water level during the May 2009 testing could not be retrieved after the 60 MW load rejection test. The transducer was later recovered during the dewatered tunnel inspection in 2010 and the data for the maximum water level during the 60 MW rejection test was recovered and analyzed. The difference

between the model predictions and field test results was less than 2 percent for pressure rise and less than 4 percent for speed rise. This level of accuracy is acceptable for this level of analysis.

A comparison between the model predictions and actual field tests for the key analysis parameters is shown in Table 12.

Table 12. Load Rejection Testing – Unit 24, May 12 to 13, 2009

	Unit 24 – 38 MW Rejection			Unit 24 – 60 MW Rejection		
	Model	Field Test	Difference	Model	Field Test	Difference
Surge Tank Max Elevation (ft)	897.1	890	7.1 ft	908.7	913	4.3
Peak Scroll Case Pressure (psig)	180.8	178.8	1.2 percent	181.4	184.7	1.8 percent
Max Unit Speed (rpm)	191.3	189.7	0.6 percent	221.3	214.2	3.3 percent

7.7 Results

Hydraulic modeling results are summarized in the following subsections. Head loss results for G2T alternatives, Links 1 through 4, are summarized in Tables 13 through 15. Table 16 provides a head loss comparison for various diameters of the G2T, Link 3 configuration. These results are also shown graphically in Figure 15. Results of the hydraulic transient analyses, for station flows of 7,770 cfs, are summarized in Table 17. Tables 18 22 and 23 summarize station outputs for the Link 3 alignment for various station flow rates and tunnel diameters. Station output verses tunnel diameter is shown on Figure 17. Multiple load cases and combinations thereof were evaluated through modeling. Relevant observation from the model studies are also discussed in the following subsections.

7.7.1 Head Loss

Head loss evaluations were performed to evaluate the connection alternatives for the three tunnel diameters initially considered. The results are summarized in Tables 13 through 15, and in Figure 15. System curve graphs showing the head loss data are included in Appendix L.

Links 1 and 2 indicate the highest flows of the four alternatives evaluated when all four units are operating. However, these two configurations are skewed towards operation of Unit 24. For operation of Units 21 to 23 without Unit 24 in operation, these two configurations have an undesirable hydraulic flow path (“S” type pattern) back to the main power tunnel which supplies Units 21 to 23. The flow path is more adverse for Link 1 due to the connection point at Unit 24 penstock. Link 4 offers no advantage in terms of head loss, capacity or transient benefit.

For dispatch operation flexibility, Link 3 offers the greatest hydraulic benefit over the full range of dispatch operations and flow rates. Since head loss for Link 3 is only slightly more than Links 1 and 2 and

it eliminates the bidirectional flow issues associated with Links 1 and 2, Link 3 is the preferred connection alternative based on overall hydraulic performance.

Table 13. Head Loss Comparison for All Link Alternatives – 18-Foot-Diameter Tunnel

Flow (cfs)	Head Loss (feet)			
	Link 1	Link 2	Link 3	Link 4
1,000	3.0	3.0	2.8	2.8
2,000	4.8	5.1	4.7	4.8
3,000	8.1	8.2	7.8	8.0
4,000	12.1	12.4	13.4	13.8
5,000	16.9	16.9	17.9	19.3
6,000	23.4	23.6	24.2	24.7
7,000	32.0	32.0	32.8	33.7
7,440	35.4	35.5	37.2	38.3
7,700	38.4	38.3	39.1	42.4

Table 14. Head Loss Comparison for All Link Alternatives – 16-Foot-Diameter Tunnel

Flow (cfs)	Head Loss (feet)			
	Link 1	Link 2	Link 3	Link 4
1,000	3.0	3.1	2.8	2.8
2,000	4.9	5.1	4.8	4.8
3,000	8.1	8.4	7.9	7.9
4,000	12.6	12.7	13.6	15.6
5,000	17.4	17.4	18.7	19.7
6,000	25.1	25.0	25.7	26.8
7,000	34.5	34.3	35.9	38.1
7,440	38.8	38.6	41.0	43.0
7,700	43.8	43.4	44.4	47.2

Table 15. Head Loss Comparison for All Link Alternatives – 14-Foot-Diameter Tunnel

Flow (cfs)	Head Loss (feet)			
	Link 1	Link 2	Link 3	Link 4
1,000	3.0	3.0	2.8	2.8
2,000	4.9	5.1	4.8	4.8
3,000	8.2	8.5	8.0	8.7
4,000	13.5	14.4	15.4	15.9
5,000	20.0	20.1	21.7	22.2
6,000	28.6	28.7	30.0	30.9
7,000	39.4	39.6	41.3	43.1
7,440	44.7	44.6	47.0	49.2
7,700	50.0	49.8	50.5	53.3

Table 16. Head Loss Comparison for Various G2T Link 3 Tunnel Diameters

Flow (cfs)	Head Loss (feet)				
	Link 3 – 22’	Link 3 – 20’	Link 3 – 18’	Link 3 – 16’	Link 3 – 14’
1,000	2.8	2.8	2.8	2.8	2.8
2,000	4.7	4.7	4.7	4.8	4.8
3,000	7.8	7.8	7.8	7.9	8.0
4,000	12.9	13.0	13.4	13.6	15.4
5,000	16.6	17.8	17.9	18.7	21.7
6,000	21.7	23.4	24.2	25.7	30.0
7,000	28.1	31.1	32.8	35.9	41.3
7,440	28.9	32.9	37.2	41.0	47.0
7,700	30.5	34.9	39.1	44.4	50.5

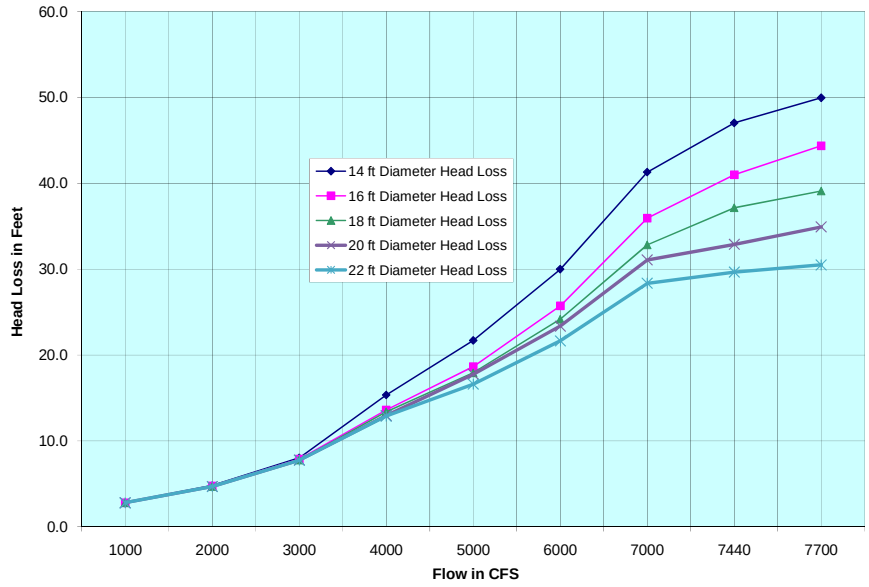


Figure 15. Head Loss Comparison for Link 3 with Multiple Second Tunnel Diameters

7.7.2 Transient Pressures and Capacity of Existing Surge Tank

Hydraulic transient analyses were performed to evaluate the impacts of tunnel diameter and connection alternatives on transient pressures and to evaluate the location and elevation controls of the existing surge tank relative to transient pressures. Table 17 summarizes the results of the hydraulic transient analyses for a maximum station flow of 7,700 cfs.

The results indicate very little change in predicted pressure and surge tank levels with the addition of the G2T compared to the current single tunnel. The predicted maximum pressure rise and surge tank levels for the four alternatives (Link 1, 2, 3 and 4) are less than the existing single tunnel configuration. Indicated surge tanks levels are below the maximum 960-foot surge tank elevation in all cases. Therefore, the existing surge tank should be sufficient to manage transient pressures.

Table 17. Summary of Hydraulic Transient Results at 7,700 cfs Station Flow

Alternative	Flow (cfs)	Estimated Steady State Head Loss (ft) at Unit Inlet All Units Generating 100% Load	Maximum Pressure at Scroll Case (psig)	Maximum Surge Tank Water Elevation (ft)
Existing Power Tunnel (G1T)				
G1T	7,700	81.6 (*See Note 1)	194.8	946.7
G2T – 18' Diameter Tunnel				
G1T + G2T Link1	7,700	38.4	189.8	936.5
G1T + G2T Link2	7,700	38.3	190.0	937.4
G1T + G2T Link3	7,700	39.1	190.0	937.7
G1T + G2T Link4	7,700	42.4	190.7	938.0
G2T – 16' Diameter Tunnel				
G1T + G2T Link1	7,700	43.8	191.0	939.5
G1T + G2T Link2	7,700	43.4	190.9	939.3
G1T + G2T Link3	7,700	44.4	190.8	939.4
G1T + G2T Link4	7,700	47.2	191.4	939.5
G2T – 14' Diameter Tunnel				
G1T + G2T Link1	7,700	50.0	191.6	940.7
G1T + G2T Link2	7,700	49.8	191.4	940.5
G1T + G2T Link3	7,700	50.5	191.6	940.7
G1T + G2T Link4	7,700	53.3	191.9	940.7

Notes:

1) Total head loss to units. This includes other losses in addition to main tunnel. SCL Memorandum dated August 7, 2000, estimated total head loss of 76.26 feet for flow of 7,565 cfs based on field test in 1993.
2) All load rejections cases at maximum forebay elevation of 875 feet, tailwater elevation of 494.5 feet.
3) Head loss averaged across all units.
4) Top of surge tank 960 feet.

7.7.3 Station Output at Various Flows with G2T

The predicted station power output for the various simulated system flows and 14-, 16-, 18-, 20- and 22-foot-diameter tunnels are presented in Table 18 for the Link 3 alignment. The Link 3 alignment was selected as the preferred connection in the June 9, 2009 Decision Workshop. Key benefits of Link 3 included dispatch operation flexibility and better hydraulic performance over the full range of operations.

As indicated in Table 18, the predicted output is below 207 MW for an 18-foot diameter tunnel. Two additional larger diameter tunnels were therefore evaluated to determine the station capacity limit for 7,440 and 7,700 cfs. This evaluation indicated that slightly more than 202 MW would be produced with a 22-foot-diameter tunnel and a station flow of 7,700 cfs. The results of the additional station capacity limit evaluations are discussed below.

Table 19 provides a summary of the predicted current G1T plant output under normal forebay and tailwater elevations for various flows ranging between 1,000 cfs and 7,700 cfs. Table 20 provides a summary of the predicted G2T plant output for the Link 3, 22-foot diameter tunnel alternative for three different forebay elevations (880, 874 and 817 feet) and flows ranging between 1,000 cfs and 7,700 cfs. Table 21 provides a comparison of plant output between the G1T and G2T configurations at reservoir elevation 874 feet.

Table 18. Predicted Station Output for Various Flows with G2T Link 3 Alternative

Flow (cfs)	Predicted Station Output (MW)				
	14 ft	16 ft	18 ft	20 ft	22 ft
1,000	28.7	28.7	28.7	28.7	28.7
2,000	56.8	56.8	56.8	56.8	56.8
3,000	84.5	84.5	84.5	84.5	84.5
4,000	108.4	109.0	109.0	109.2	109.2
5,000	133.6	134.7	135.0	135.1	135.5
6,000	157.9	159.9	160.6	160.9	161.7
7,000	178.1	180.9	182.6	183.6	185.1
7,440	186.0	189.4	191.5	194.0	196.2
7,700	190.7	194.2	197.2	199.6	202.2

Notes:
1) Station output estimated at generators terminals and does not included step up losses.
2) Turbine efficiencies: nominal 94 percent average for all units.
3) Generator efficiencies: nominal 98.6 percent average for all units.
4) Power Factor 0.97 for Units 21 to 23, 0.95 for Unit 24.
5) Forebay elevation 875 feet, tailwater elevation 494.5 feet.
6) Head loss averaged across units.

Sample input and output files from the PTURB model are included in Appendix N. A sample calculation for the unit output is included in the same appendix.

Table 19. Hydraulic Model Predicted G1 Output with Forebay El 874 Feet

Station Flow (CFS)	Predicted Plant Output (MW)
1,000	28.7
2,000	56.5
3,000	82.9
4,000	105.4
5,000	128.3
6,000	148.0
7,000	164.7
7,440	169.9
7,700	172.5

Table 20. Hydraulic Model Predicted G2 Output (Link 3, 22' Diam.)

	Predicted Plant Output (MW)		
Station Flow	Maximum Forebay (El 880 ft)	Normal Forebay (El 874 ft)	Extreme Low Forebay (El 817 ft)
1,000	29.2	28.8	24.5
2,000	57.8	56.9	48.3
3,000	85.6	84.2	71.5
4,000	110.7	108.8	92.2
5,000	137.3	135.1	114.2
6,000	162.8	160.0	135.3
7,000	187.3	184.0	154.8
7,440	197.8	194.6	162.7
7,700	203.3	199.9	167.2

Table 21. Comparison of Hydraulic Model Predicted Output at Forebay El 874 Feet

Station Flow	G2 Predicted Plant Output (MW)	G1 Predicted Plant Output (MW)	Increase (MW)	Increase %
1,000	28.8	28.7	0.1	0.2%
2,000	56.9	56.5	0.4	0.7%
3,000	84.2	82.9	1.4	1.6%
4,000	108.8	105.4	3.4	3.2%
5,000	135.1	128.3	6.8	5.3%
6,000	160.0	148.0	11.9	8.1%
7,000	184.0	164.7	19.3	11.7%
7,440	194.6	169.9	24.7	14.5%
7,700	199.9	172.5	27.4	15.9%

Notes for Table 19 through 21:

1) Station output estimated at generators terminals and does not included step up losses.
2) Turbine efficiencies: nominal 94 percent average for all units.
3) Generator efficiencies: nominal 98.6 percent average for all units.
4) Power Factor 0.97 for Units 21 to 23, 0.95 for Unit 24.
5) Head loss averaged across units.
6) Tailwater elevation varies based on SCL supplied data table.

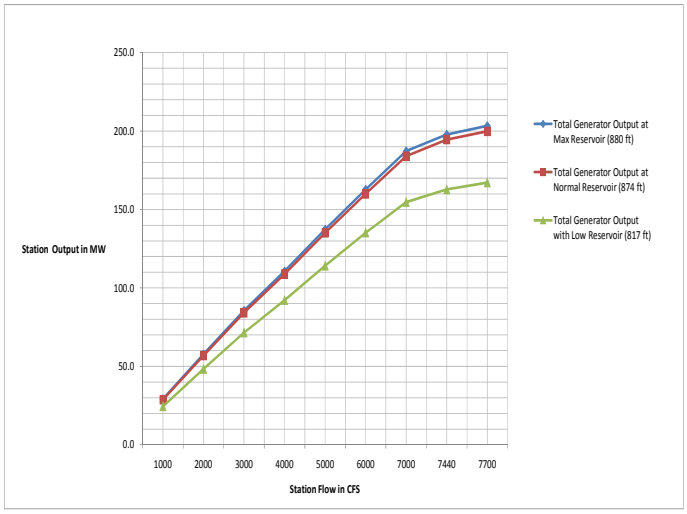


Figure 16. Predicted Station Capacity with G2T Configuration, Link 3 22' Diam.

7.7.4 Flow Capacity and Station Capacity Limit

At various stages of the preliminary design, SCL staff has indicated that there are three limitations on the maximum flow for the Gorge station:

- Maximum normal station flow at Gorge is limited to 7,440 cfs (the water rights for Gorge is 7,500 cfs). During high water events when releasing water through the dam's spill gates, SCL will exceed these flows through the powerhouse;
- The maximum station power production must be within the generator nameplate capacity of 207.5 MW; and
- The plant maximum hydraulic capacity is 8,112 cfs.

The Design Team was requested to provide a projection of total station output at 7,440 and 7,700 cfs for Link 3 alignment. Predicted station output based on maximum flows of 7,440 cfs and 7,700 cfs are summarized in Table 22 and Table 23, respectively. As discussed in Section 7.7.3, three larger diameter tunnels of 20, 22, and 25 feet were analyzed for the Link 3 alignment because the predicted output with the 18-foot-diameter tunnel fell below 207 MW. The maximum hydraulic capacity value of 8,112 cfs was

increased from the original value of 7,700 cfs at the request of SCL towards the end of the preliminary design together with a request for additional modeled output data for this increased flow.

Figure 17 shows the predicted plant capacity verses the G2T diameter. This figure is also included in Appendix L. The results indicate that G2T will not achieve 207 MW at 7,440 cfs. The results indicate that 202 MW can be achieved at 7,700 cfs with a 22-foot-diameter tunnel. The maximum predicted power output at 7,440 cfs is approximately 199 MW and would require a 33-foot-diameter tunnel, after which the power output fails to increase significantly despite increasing the tunnel diameter. As the G2T size increases beyond about 22 feet in diameter, the resulting benefit from increasing the tunnel diameter rapidly reaches a point of diminishing return with regard to head loss. As the G2T size continues to increase, the main tunnel friction losses become a minor portion of the overall system head loss, which includes the intake, manifold and unit penstock losses. It is anticipated that a second tunnel with a larger diameter than the existing tunnel could also introduce hydraulic losses due to the expansion at the inlet to the second tunnel.

Table 22. Predicted Station Output with Flow of 7,440 cfs with Link 3 Connection

Configuration	Frictional Loss	Station Output
G1	72.4 feet	169.9 MW (*See Note 1)
G1 + 14-foot G2	47.0 feet	186.0 MW
G1 + 16-foot G2	41.0 feet	189.4 MW
G1 + 18-foot G2	37.2 feet	191.5 MW
G1 + 20-foot G2	32.9 feet	194.0 MW
G1 + 22-foot G2	28.8 feet	196.3 MW
G1 + 25-foot G2	25.4 feet	198.1 MW

Table 23. Predicted Station Output with Flow of 7,700 cfs with Link 3 Connection

Configuration	Frictional Loss	Station Output
G1	77.8 feet (* See Note 1)	172.5 MW (*See Note 1)
G1 + 14-foot G2	50.5 feet	190.7 MW
G1 + 16-foot G2	44.4 feet	194.2 MW
G1 + 18-foot G2	39.1 feet	197.2 MW
G1 + 20-foot G2	34.9 feet	199.6 MW
G1 + 22-foot G2	30.5 feet	202.2 MW
G1 + 25-foot G2	26.9 feet	204.3 MW

Notes for Tables 22 and 23:

1) Current Gorge tunnel (G1) predicted total frictional losses and station output from HDR hydraulic model. Total frictional loss includes other friction losses besides main tunnel.

2) Unit flows are based on SCL dispatch model: Unit 1:1,629 cfs; Unit 2:1,379 cfs; Unit 3:1,254 cfs; and Unit 4:3,439 cfs.
3) Station output estimated at generators terminals and do not included step up losses.
4) Turbine efficiencies: nominal 94 percent average for all units.
5) Generator efficiencies – nominal 98.6 percent average for all units.
6) Power Factor 0.97 for Units 21–23, 0.95 Unit 24.

Table 24 shows the predicted G2T maximum hydraulic capacities from the PTURB hydraulic model with a 22-foot diameter G2T and the Link 3 alignment. Predicted hydraulic capacities are listed for three different forebay elevation levels.

The predicted maximum individual turbine hydraulic capacities were calibrated to the index tests for Units 21 and 24 performed after runner replacements (original G1 tunnel configuration). This establishes the maximum turbine gate opening in the model. The maximum model gate openings were then used in the G2T model to establish the new overall maximum hydraulic capacities at different forebay elevations.

Units 21, 22 and 23 are modeled as identical hydraulic turbines. Unit 24 is modeled with different hydraulic characteristics since it has a much larger hydraulic capacity, different operating speed, etc.

Table 24. Estimate of Existing and New Maximum Hydraulic Capacity with 22' Diam. G2T

Forebay Elevation	Maximum Calculated Hydraulic Capacity
Maximum G2T (880 ft)	8,260 cfs
Normal G2T (874 ft)	8,220 cfs
Normal Existing G1T* (874ft)	8,112 cfs*
Extreme Low G2T (817 ft)	7,500 cfs

*Existing maximum hydraulic capacity based on recorded flow during high-water event (November 2006)

SCL staff estimate the current maximum combined turbine hydraulic capacity to be 8,112 cfs. This capacity is based on a recorded flow that occurred on November 26, 2006, with a lake elevation of El. 871.54 and a gross head of 375.84 feet. The G2T addition at the Gorge facility is expected to have negligible impact on the plant hydraulic capacity since the turbine hydraulic capacity is the limiting factor. The reduction in head loss will increase the net head to the turbines and the output power. Hydraulic analysis indicates that the Gorge powerhouse with the existing tunnel could produce approximately 182 MW at a flow rate of 8,112 cfs. With a 22-foot diameter G2T, hydraulic analysis indicates that the powerhouse could produce approximately 210 MW at a flow rate of 8,112 cfs.

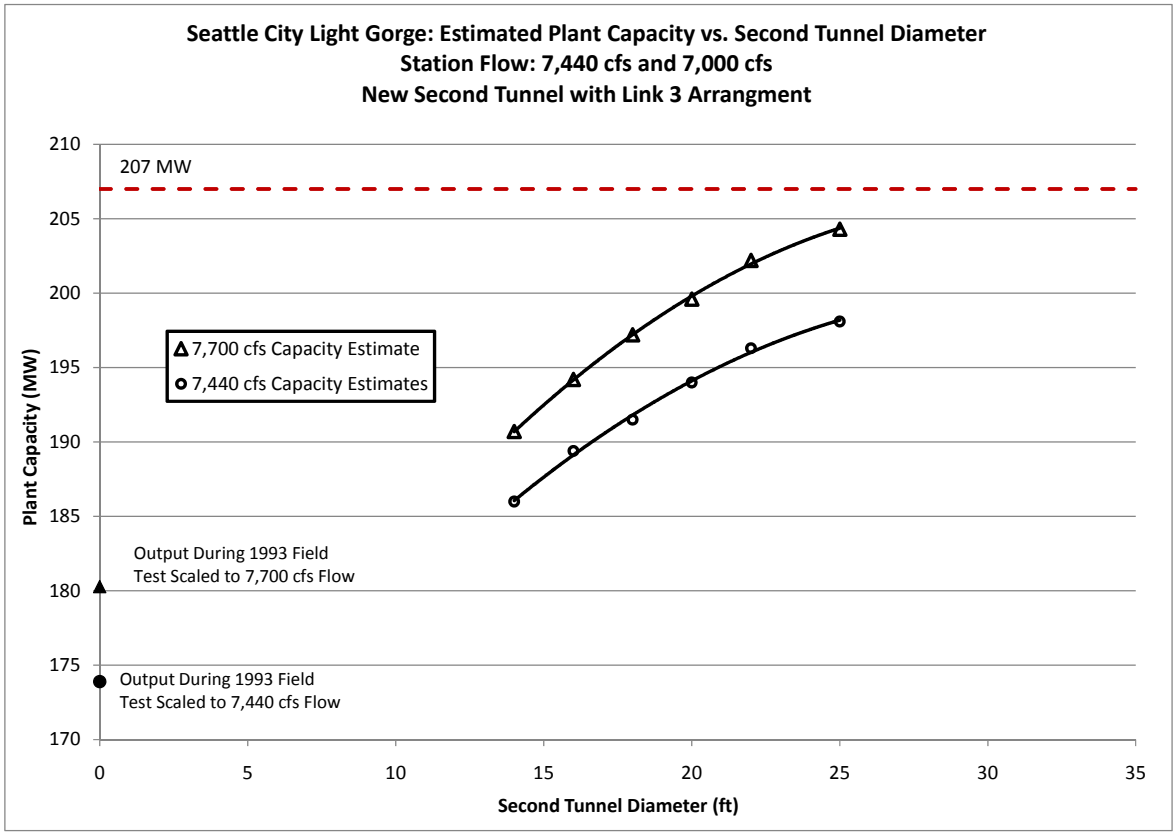


Figure 17. Station Capacity vs. G2T Diameter

7.7.5 Head Loss Benefit

The overall predicted head loss reduction for various G2T diameters is summarized in Table 25 for station flows of 7,440 and 7,700 cfs. This is the predicted benefit from the PTURB GIT hydraulic model as compared to the PTURB G2T hydraulic model.

Table 25. Predicted Head Loss Benefit with G2T Link 3 Alternative

G2T Diameter (feet)	Benefit @ 7,440 cfs	Benefit @ 7,700 cfs
14	25.4 feet	27.3 feet
16	31.4 feet	33.4 feet
18	35.3 feet	38.7 feet
20	39.5 feet	42.9 feet
22	43.5 feet	47.3 feet
25	47.0 feet	50.9 feet

7.7.6 SCL Dispatch Model Main Tunnel Head Loss Coefficients

The SCL dispatch model includes an overall head loss coefficient (K factor) for the main tunnel to the Unit 24 bifurcation and separate head loss coefficients for each of the units from this point to the turbine runner. For evaluation of the generation benefits of the second tunnel addition, new head loss coefficients are required to replace the current values. The new K factors are based on the hydraulic grade line at the Unit 24 bifurcation as taken from the PTURB program output file, and a predicted maximum flow rate of 7,700 cfs.

The main tunnel loss is in terms of: Main Tunnel Head Loss (feet) = K x (Flow in KCFS)^2

The K factor for the existing tunnel (G1T), from the current SCL Dispatch Model, is 0.965.

The calculated K factors for the various G2T diameters under consideration, and using the Link 3 connection, are presented in Table 26 below.

Table 26. Calculated Head Loss Coefficient (K Factor)

G2T Diameter (feet)	K Factor
14	0.750
16	0.648
18	0.560
20	0.490
22	0.412

8 Decision Workshop

A Decision Workshop was held between SCL and key members of the Design Team on June 9, 2009. The purpose of the workshop was to evaluate the data and information of the preliminary Tunnel Diameter Optimization Layout (TDOL) and Water Transient Study (WTS) reports, and to make an informed decision concerning the preferred downstream connection configuration and other project elements. The primary content in this report was presented at the workshop. Notes from the workshop are included as Appendix A.

A preliminary evaluation of the four different connections was performed prior to the workshop. The workshop considered the evaluation work carried out to date, provided feedback on the considerations and criteria used in the evaluation process and agreed on the next steps in the overall optimization process. Table 27 summarizes the evaluation of the four alternatives and illustrates the considerations and criteria used in developing the recommended project configurations, while incorporating the various levels of feedback which arose at the Decision Workshop.

Table 27. Evaluation of Gorge Tunnel Connection Links

	Weighting Factor (1-5)	Link #1	Link #2	Link #3	Link #4	Remarks
Design/ Constructability	**3**					
Angle of the Connection Link		◐	●	●	◐	
Excavation of the Link		●	●	◐	◐	
Length of the Connection Link		●	●	◐	●	
Geological Considerations	**4**					
Connection Geology		◐	●	●	●	
Groundwater (max pressure)		◐	◐	◐	◐	
Risk	**3**					
Hydraulic Configuration		●	●	◐	○	
Original Tunnel Operation Time Impact During Construction		●	●	○	○	
Tunnel Plug Construction Difficulties		◐	◐	●	●	
Operation & Maintenance	**2**					
Future Plant Expansion		●	●	◐	◐	
Maximize flow potential		◐	●	●	○	
Access		●	◐	◐	○	
Cost	**4**					
Cost Impact if a delay to G1T operation		●	◐	○	○	Can not isolate flows to G1T
Constructability		●	●	◐	◐	Easier Connection for Links 1 & 2
Schedule	**1**					
Impacts to Schedule		●	●	◐	○	
Hydraulics	**5**					
Head Loss Performance		◐	●	●	○	
Flow Paths		◐	◐	●	◐	Bi-directional flow for Links 1 & 2
Transient Pressures		◐	◐	◐	◐	
Total		**141**	**149**	**187**	**104**	

Key: ○ *= 1 less desirable* ◐ *= 2 neutral* ● *= 3 more desirable*

Based on the scoring evaluation and construction issues of each individual alternative, and feedback from the Decision Workshop participants, the recommended connection is Link 3. This link offers similar construction issues and risks, schedule, and cost as the other leading alternatives, Links 1 and 2, but it affords significant hydraulic and operating benefits which are attractive to the mode of operation for the Gorge Powerhouse.

Following the decision to proceed with Link 3 as the preferred downstream connection, the Design Team developed construction cost estimates for this layout, for tunnel diameters ranging from 14 to 18 feet. These cost estimates were subsequently supplemented by construction cost estimates for 20- and 22-foot-diameter tunnels. The head loss information and system curve data for the 14- to 22-foot-diameter tunnels will be incorporated by SCL into their energy analysis evaluation.

9 Additional Construction Considerations

This section describes additional construction considerations related to the project configuration and layout and impact the construction costs. These considerations include, control of water, staging area layout, powerhouse bridge capacity, and construction power requirements.

9.1 Control of Water

Three sources of water associated with tunnel construction that need to be addressed are: (1) groundwater inflows into the tunnel, (2) water used for tunnel construction, and (3) stormwater runoff from portal and staging areas disturbed by construction activity. All sources of water must be handled, treated, and disposed of in accordance with applicable laws, regulations, and permits.

9.1.1 Groundwater Inflows

Groundwater inflows into the tunnel may need to be controlled to: (1) improve ground stability and minimize ground movements, (2) prevent possible adverse impacts to the groundwater regime around the tunnel, and (3) prevent adverse impacts on the quality of the final lining. Groundwater inflow into the tunnel will occur through joints, fractures, and shears. The inflow quantity is primarily a function of the spacing, width, orientation, infilling and interconnectivity of the joints, fractures and shears, and the effective hydrostatic pressure at the tunnel depth. Detailed ground water observations during previous construction and inspection are described in the "Preliminary Geologic Assessment," included as Appendix D.

Rough estimates of inflow are presented below to provide some basis for project planning. Accurate estimates of groundwater inflows into the tunnel are not possible because of the limited information that is available along the entire tunnel length at this time.

Table 28 summarizes the preliminary tunnel inflow estimates. Peak heading inflows (flush flows) will occur when the tunnel excavation encounters sheared or fractured zones with stored groundwater. The estimates do not include the contribution from construction-related water (also known as process water). Full details regarding the groundwater inflow estimate calculations are presented in the "Groundwater Inflow Predictions" included as Appendix I. These predictions are based on preliminary assumptions and the model will need to be updated as more data become available.

Table 28. Maximum Tunnel Inflows Estimates

Flow	Flow (gpm)
Instantaneous Inflow (at heading)	1,400
Peak Flow (at portal	1,600
Sustained Inflow (at portal)	300

Based on the inflow estimates, extensive groundwater control measures do not appear to be warranted for the proposed G2T, considering the very low rock permeability. It should be noted that inflow estimates

are very sensitive to variations at the high end of the permeability range. These variations can be influenced by unknown reaches of high permeability rock or by a single feature such as a fault or shear. These isolated features could result in local inflows of several thousand gallons per minute (gpm). The estimates for tunnel inflow do not account for the uncertainty of encountering local zones of higher permeability. The likelihood of encountering such a feature will be investigated during the planned 2009 exploration program.

A statistical groundwater inflow model is recommended to account for several other factors influencing inflows into tunnels such as storativity of the rock mass, variations in permeability, and variations in groundwater head. This approach requires a more extensive database of borings and packer permeability test data; information that was obtained during the summer 2009 exploration program.

9.1.2 Probing and Pre-excavation Grouting

If concerns arise over excessive groundwater inflows or adverse groundwater effects on excavation stability, particularly in the shear and fault zones, then pre-excavation grouting may need to be performed in advance of tunnel excavation. Pre-excavation grouting involves drilling a series of holes ahead of the tunnel face and injecting grout into the rock mass around the tunnel to treat specific high permeable features or to create a zone of low permeability around the future tunnel opening. Effectively performing pre-excavation grouting requires a TBM that incorporates specific design features that allow drilling ahead of the face in a number of locations around the full tunnel perimeter. If the inflows through the probe holes meet or exceed prescribed inflow criteria, a grouting phase is initiated using either the probe holes as grout holes or additional grout holes as necessary. Verification holes would be drilled after grouting of the grout holes to confirm that the rock mass permeability has been reduced to an acceptable level.

9.1.3 Water Management, Handling, and Disposal

Groundwater and construction water inflows into the tunnel will drain by gravity to the portal, where they will be held in a settling pond before being pumped to a treatment facility and discharged back into the ground via percolation. The water will be turbid and could be contaminated with suspended solids particles, hydraulic fluid residue, small amounts of oil, grease and fuel, as well as pH fluctuations due to cement in grout and concrete. The treatment requirements for this wastewater will depend on the standards established as part of the permitting process. Typically settling ponds and oil skimming are sufficient treatment of the water prior to discharge. Efforts to minimize the area of disturbance around the portal require the need for an offsite treatment facility. Additional details regarding water management, handling, and disposal are discussed in the *Water Management Plan* (HDR Engineering Inc., 2009).

9.1.4 Stormwater Runoff

Erosion and sediment control will be necessary to manage stormwater runoff. The goal of managing stormwater runoff will be to disperse the flows rather than creating flow concentration. Control measures would include best management practices such as silt fences and straw bales placed at the base of fill slopes and excavations with positive drainage, hydroseeding, and erosion mats. Additional details regarding stormwater management, handling, and disposal are discussed in the *Water Management Plan* (HDR Engineering Inc., 2009).

9.2 Staging Area Layout

Adequate space for construction is essential to efficiently carry out the work. A total staging area of one to two acres is desirable for tunnel construction. The designated portal staging area provides approximately 1.5 acres. The staging area will include space for the following elements:

- Office trailers for owner/designer, contractor, and CM inspectors, and change rooms (dry house) for workers
- Parking and turn-around areas
- Maintenance shop, and tool containers
- Crane, loader, generator, and other surface support equipment
- Material stockpile area (including tunnel support and tunnel lining materials)
- Tunnel water handling storage ponds/tanks and treatment facility
- Temporary muck stockpile area
- Muck car dump/rollover area (muck car option only)
- Muck transfer area for loading haul trucks

Figure 18 shows a conceptual site layout for a continuous conveyor option while Figure 19 shows a conceptual site layout for a muck car option. The actual site layout is the contractor's decision but the conveyor option is highly likely as it eliminates the need for a muck dump area outside of the tunnel portal and provides more space for tunneling operations. Regardless of whether a continuous conveyor option or a muck car option is used, the contractor will likely use a conveyor to transport the tunnel muck from the portal area to a muck storage area across the bridge so that truck traffic at the portal can be minimized.

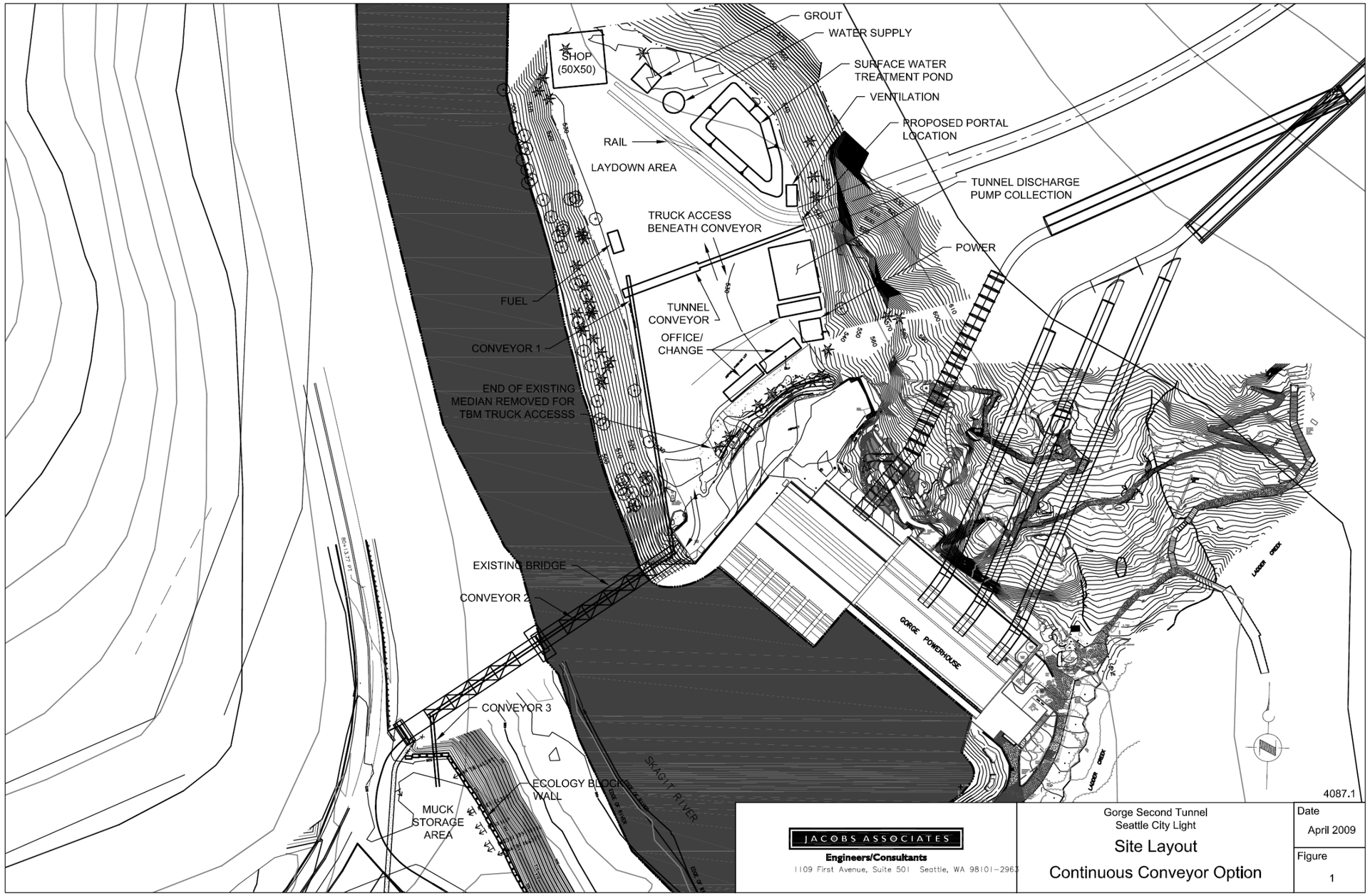


Figure 18. Site Layout for Continuous Conveyor Option

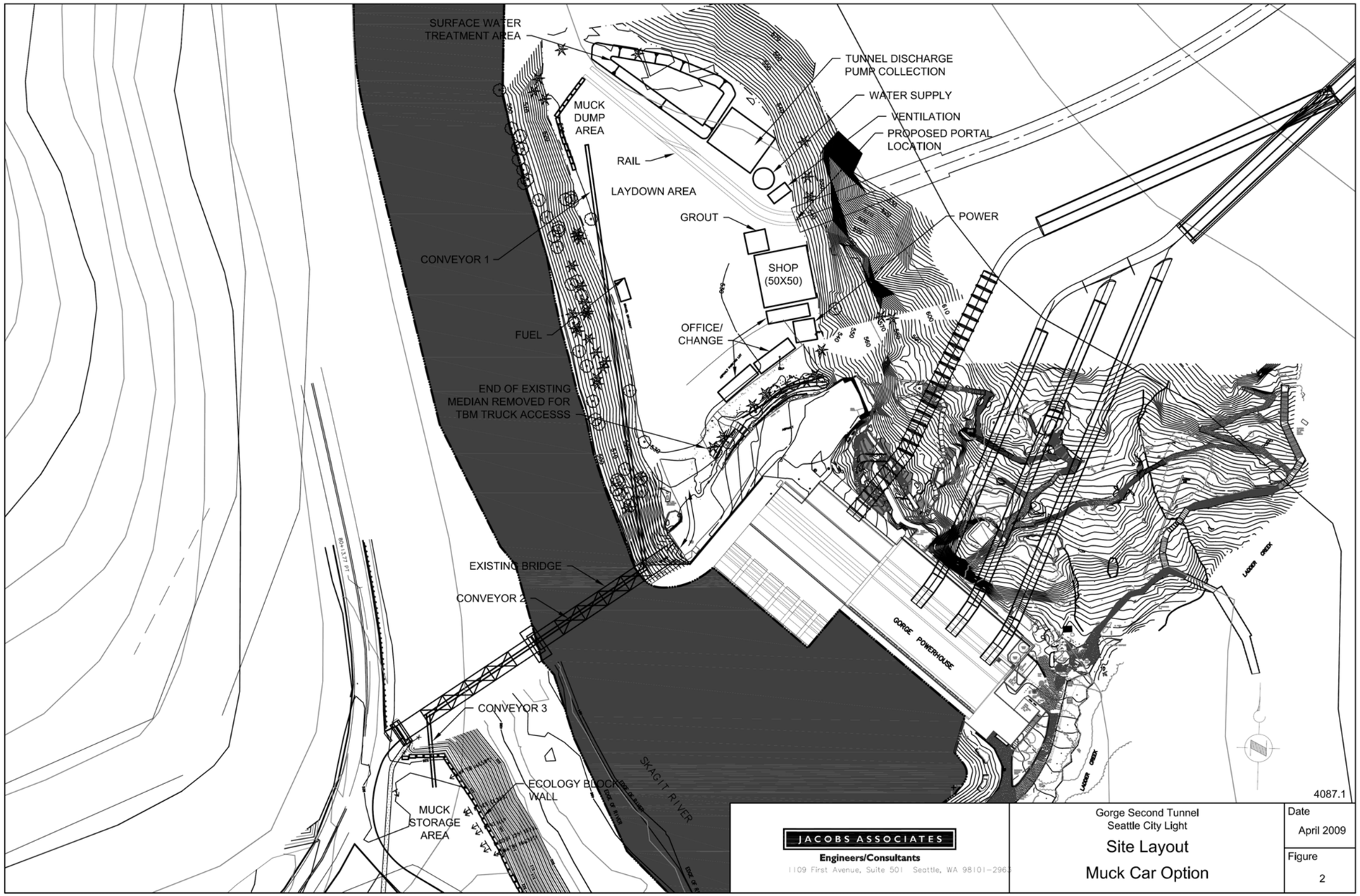


Figure 19. Site Layout for Muck Car Option

9.3 Powerhouse Bridge Capacity Analysis

Access to the site using the Gorge Powerhouse Bridge is necessary for the transportation of heavy tunneling equipment. The information provided by SCL regarding the bridge was reviewed and an independent analysis was carried out in order to better understand the bridge capacity. Preliminary analysis was performed on the major structural components of the bridge to determine allowable truck loads on the bridge. This analysis is preliminary, and a more detailed evaluation will be required during final design. The purpose of this preliminary analysis was to determine if typical TBM elements can be transported across the powerhouse bridge assuming that these elements can be assembled at the portal site by a mobile crane. The main issues considered included the following:

- Does the bridge have enough capacity to safely transport the TBM elements?
- What are the dimensions for a truck and trailer to fit on the bridge and to maneuver on and off?
- How heavy are the loads for a typical TBM in the size range contemplated for Gorge?

The analysis indicates that the TBM for the Gorge 2nd Tunnel project will need to be transported across the Powerhouse Bridge in smaller elements than typical so that the bridge will not be overloaded. The current bridge envelope does not present a bottle-neck for transporting the TBM elements. The truck assumed in this analysis has 7 axles and an overall wheel base of 75 feet. The analysis indicates that the heaviest TBM element required will weigh approximately 90 tons. The bridge was evaluated for the expected wheel loads of the truck carrying this TBM element and the results are summarized in Table 29 below.

Table 29. Powerhouse Bridge Load Analysis

Item	Expected Load (tons)	Bridge Capacity (tons)	Analysis Result/ Recommendations
Truck wheel loads	6.1	3	Deck grating should be reinforced with plates or sleepers to spread load.
Axle loads	25	31.9	Deck floor beams appear to be OK for axle loads.
Sum of loads acting on any 26-foot longitudinal portion of the bridge	49	50	Transverse girders appear OK for truck loads.

9.4 Construction Power Requirements

An initial estimate of the construction power requirements was performed based on the anticipated construction equipment and an assumed tunnel length of approximately 11,000 feet. The results of this exercise were recorded in a technical memorandum, which is presented in Appendix O. Included in the estimate were power requirements for surface operations, underground operations, TBM excavation, a conveyor system for muck removal, and auxiliary equipment. A summary of the estimated construction power requirement for the G2T Project is provided in Table 30. This represents the maximum power

required, which occurs when the tunnel is at its full excavated length of 11,000 feet. The initial total power requirement is estimated to be around 5,200 kW, which represents a minimum power drop of approximately 6,500 kVA.

Table 30. Estimated Construction Power Requirements

Equipment	Load (kilowatt)
Tunnel Boring Machine	2,835
TBM Trailing Equipment	448
Tunnel Lighting	44
Tunnel Ventilation Fans	410
Sump Pumps	224
Muck Conveyor	597
Air Compressor	373
Shop Feed	80
Shop Welders	67
Change House Feed	18
Yard Lighting	5
Portal Lighting	2.5
Office Trailers	80
Total Load	5,183 kW
Maximum Power Draw	**6,479 kVA**

10 Cost Estimate and Schedule

Production type construction cost estimates were initially prepared for the preferred tunnel layout (Link 3) for three different tunnel diameters: 14, 16 and 18-feet. The estimates are based on the assumptions and construction methodologies discussed in the previous sections. This type of estimate accounts for labor, equipment, and construction methods by segregating work into discrete tasks and develops crew sizes and equipment spreads in conjunction with production rates. Key elements of the cost estimate include the following:

- Portal development;
- TBM starter tunnel;
- TBM excavation;
- Upstream connection excavation;
- Downstream connection excavation;
- Tunnel lining;
- Upstream connection and lining;
- Downstream connection and lining;
- Tunnel plug; and
- Portal restoration.

Further to the draft issue of this report, and the need to evaluate larger excavated tunnel diameters, construction cost estimates were also developed for diameters of 20 and 22 feet.

10.1 Cost Estimate Summary

The key cost assumptions and construction methodologies are presented in *Gorge Second Tunnel Cost Estimate Narrative* which is included in Appendix P. In addition, this appendix also includes the three individual cost estimates, presented as a *Summary Opinion of Probable Unescalated Construction Bid Excluding Owner Contingency*, together with full direct and indirect cost information.

The cost estimates includes 10,842 linear feet (lf) of either 14-, 16-, 18-, 20- or 22-foot-diameter unlined main-beam TBM tunnel, a 29 lf upstream connector tunnel, a 89 lf downstream connector tunnel, and a 150 lf starter tunnel constructed using drill and blast methods. It was assumed that Notice to Proceed (NTP) for construction would be October 1, 2012. Construction work hours are based on two shift operation performed on a 5 days per week basis.

The construction bid estimates are broken down into direct and indirect costs. Direct costs include the actual costs to execute the construction work. In the case of the G2T project this consists of site work, portal construction, tunnel excavation, support and lining, and connections. Indirect costs typically consist of equipment ownership, mobilization, maintenance and supervision, and mark-ups.

Following the development of the initial construction cost estimates, they were reviewed by an independent estimating firm, Redmond Construction Engineering, for accuracy validation. This resulted in some minor revisions to the construction cost estimates. The estimated construction costs following this review process are presented in Table 31.

Table 31. Construction Cost Estimate Summary

Cost Item	14 ft	16 ft	18 ft	20 ft	22 ft
Site Work and Portal	$0.25M	$0.3M	$0.3M	$0.35M	$0.4M
Tunnel	$13.0M	$13.9M	$14.7M	$15.55M	$16.6M
Total Direct Cost	$13.25M	$14.2M	$15.0M	$15.9M	$17.0M
Indirect Cost	$33.75M	$35.0M	$36.3M	$38.0M	$39.7M
Construction Cost	**$47.0M**	**$49.2M**	**$51.3M**	**$53.9M**	**$56.7M**

10.2 Schedule

A cost loaded schedule was developed with each cost estimate and is included with the *Summary Opinion of Probable Unescalated Construction Bid Excluding Owner Contingency* in Appendix P .The duration of construction for all the alternatives, from NTP, is anticipated to be approximately 26 to 27 months.

10.3 Contingency Assumptions

In accordance with the American Association of Cost Estimators (AACE) recommended practice as set forth in Publication 17R-97, Cost Estimate Classification System, the accuracy of these estimates are commensurate with the level of project definition. Accordingly, the accuracy of this estimate should be taken as a Class 3 Budget Estimate with an expected accuracy range of +30/-15 percent. This means that the accuracy of each estimate is estimated to be between 85 and 130 percent of the project estimate at 100 percent level of project definition (i.e., the final design submittal).

Contingency is included for each alternative to cover design and construction issues that are still undefined at this preliminary (10 to 40 percent) level of project definition. Rather than including contingent sums in the production rates for the various items of work where the quantification of potential impacts might be construed as an endorsement of their occurrence, it is considered more appropriate to include a contingency allowance. As project definition improves, the amount of estimated contingency will decrease.

10.3.1 Owner Contingency

This contingency allowance consists of two components; Project Definition and Bidding Climate. The Project Definition contingency attempts to compensate for an incomplete view of the whole picture, and as project definition increases, this portion goes to zero because all project conditions that will be disclosed to bidders will be known, and included in the estimate. The Bidding Climate contingency is intended to account for the current level of competitive bidding, including available contractor resources and similar concurrent projects.

For the G2T project definition contingency is primarily related to the influence of unknown ground conditions on TBM performance and initial support and final lining assumptions. Due to the scarcity of data related to the rock mass conditions, the amount of project definition contingency is set to 20%.

Bidding Climate contingency is set at 5 percent. Based on these items we recommend that SCL reserve an amount equal to 25 percent of the total cost of the project for Owner Contingency. Owner contingency is excluded from the cost estimates presented in Table 31.

10.3.2 Contractor Contingency

This contingency is included in the cost estimate and is used to quantify what the contractor perceives as his risk—i.e., what problems he may encounter on the job for which he will not receive payment. This is a function of many things, such as GBR baseline levels, payment provisions, and even how comfortable the contractor is with the accuracy of his bid (i.e., his own estimating accuracy). The point to be made is that this risk will be included in the contractor's bid, and therefore paid for by the Owner. This contingency has been set to 1.4 percent for this estimate.

10.4 Independent Cost Estimate Review

As stated above the initial cost estimates were reviewed by an independent Third Party estimator. The results of this independent review, and the design team responses, are presented in Appendix P. The independent review typically resulted in between a 5 and 6 percent increase in the construction cost estimate for the three different tunnel diameters.

11 Energy and Financial Analysis

11.1 Energy Analysis

Using the results of the hydraulic modeling, SCL System Control Center used in-house Skagit system simulation model to calculate energy benefits for tunnel diameters varying in range from 14 to 22-feet.

For this energy benefit analysis a comparison was carried out between the monthly Ross Dam inflows using the average of the 1910-2009 (total historical data range) and the average of 1986-2009 (years SCL used to simulate the Gorge hourly operation). This comparison showed that the annual inflow from 1986-2009 is about 5 to 6 percent lower than using the inflows from the total historical data range. Thus, it is reasonable to expect that the Gorge tunnel analysis using 1986-2009 data would be 10 percent or less below what would result if the 1910-2009 data were used. Based on this, the energy benefit calculated by SCL is considered to be a conservative estimate.

The simulation used hourly Gorge generation discharge, forebay and tailwater elevation, and unit generation data from 1986 through 2009. For approximately the last 10 years, the data has been extracted from SCL's System Control Center's EMS database. However, data from the first 15 years came from various sources, including back calculation from the unit generations data, USGS archived data and SCL archived data. The model assumes optimal dispatch of generator units based on the hourly generation flows (not including spill to minimize model bias) and then estimates the unit generation.

Originally, the simulation was run for several tunnel diameters (14 feet through 22 feet in 2 foot increments). However, after SCL selected the 22-foot diameter tunnel, the hydraulic model was fine-tuned and the simulation was revised for the 22-foot diameter tunnel only. This revision netted a slightly lower energy benefit than the original simulation. SCL subsequently used the lower, revised energy value in their final financial analysis. In Table 32 below, the average additional energy values for the 14- to 20-foot diameter options are from the original simulation, and the value for the 22-foot diameter are from the revised simulation. The energy benefit (MWh) for each tunnel diameter, in monthly high load and low load hours for each flow year (1986–2009) is presented in Appendix Q. The appendix also provides a more compact presentation of the results with Average Annual MW (aMW) tabulated for each tunnel diameter and each flow year. Note: aMW can be converted to MWh/year as follows:

aMW x 8760 hr/yr = MWh/yr

Table 32. Average Additional Energy Summary

Tunnel Diameter (feet)	Average Additional Energy (MWh/yr)
14	31,871
16	39,337
18	46,152
20	51,760
22	55,844

11.2 Financial Analysis

Seattle City Light (SCL) Finance Unit used the energy analysis results as input to an analysis tool which predicts energy values based on numerous random variables. Variables which most strongly influence electrical energy prices are water flow in the rivers and the price of natural gas. Other variables, such as economic indicators, also play a role. The model used generates a total of 2001 simulations of hourly prices over the course of a year for a given natural gas price. For simplicity each simulation could be considered a 'price year'. The price years are tied to the energy model by way of water flow as follows:

- The price years are sorted from lowest water flow to highest.
- The 2001 price years are distributed evenly among the 24 flow years for each assumed natural gas price. In other words, each flow year will have 2001/24 = 83 different price scenarios for a given natural gas price.
- The 83 scenarios with the lowest water flow are tied to the energy model run using the lowest flow year, the next 83 price scenarios are associated with the next lowest flow year, and so on. By inference, the 83 price years with the highest water flow are associated with the highest flow year in the energy model.

This approach provides a range of possible financial outcomes for each flow scenario. This is consistent with the financial analysis approach used by SCL Power Marketing.

The financial analysis results do not include the Renewable Energy Credit (REC) benefit which I-937 brings to the project. According to SCL's Integrated Resource Plan group, the value currently being estimated for a MWh of 'green energy' is $25. This is a significant contributor to the financial benefit.

Appendix R presents, in tabular form, the levelized annual, and 50-year, value of energy from the Gorge 2nd Tunnel Project at alternative levelized natural gas prices (ranging from $5.00 to $10.00/MmBtu).

For example, for the 22-foot diameter tunnel, assuming a natural gas price of $6/MmBtu, the annual financial benefit without considering the REC value is $2.7 Million. The REC benefit provides an additional $1.4 Million, representing a gain of approximately 65% in value.

12 Conclusions and Recommendations

The presented optimization evaluations have identified a recommended alignment, connection configuration, and other important project features, which have been incorporated into the construction cost estimate. Hydraulic modeling was performed concurrently on alternative configurations to refine these optimization efforts. The results of this modeling and the outcomes of the June 9, 2009 decision workshop have been incorporated into this combined report.

A methodology was proposed and accepted for evaluation of the downstream connection configurations. Site layout evaluations were also performed. These evaluations included portal layout and a powerhouse bridge load evaluation.

The screening criteria presented in the June 2009 preliminary reports were critically reviewed by the project stakeholders, before and during the June 9, 2009 workshop, and validation of the criteria, together with the addition of any further screening criteria, was obtained.

The G2T will be approximately 11,000 feet long and will parallel the existing power tunnel. The upstream connection is at the base of the existing intake structure ramp, a location that maximizes hydraulic efficiency. The downstream connection (Link 3) is located just upstream of the bifurcation that feeds all of the powerhouse units. This location was selected based on the most favorable hydraulic conditions over a range of operating scenarios.

Tunnel boring machine excavation methods are most appropriate for the tunnel construction, with use of drill-and-blast methods for construction of the starter tunnel and connection adits. The majority of the rock is anticipated to be strong and durable enough to remain unlined after construction.

The results of the hydraulic modeling for the G2T addition indicate that:

- The existing surge tank is sufficient to manage transient pressures for the range of tunnel diameters and station flows considered;
- The G2T will not meet the functional requirement of 207.5 MW at a station flow of 7,440 cfs;
- A 22-foot-diameter new tunnel can achieve a station output of approximately -202 MW at a station flow of 7,700 cfs;
- With a maximum hydraulic capacity of 8,112 cfs, a 22-foot diameter new tunnel is able to provide a station output of 210 MW;
- Head loss benefit for a station flow of 7,440 cfs is approximately 43.5 feet for a 22-foot-diameter tunnel and 25.4 feet for a 14-foot-diameter tunnel; and
- Head loss benefit for a station flow of 7,700 cfs is approximately 47.3 feet for a 22-foot-diameter tunnel and 27.3 feet for a 14-foot-diameter tunnel.

Tunnel diameters larger than 25 feet offer only slight capacity increases because the incremental head loss reduction is low since the main power tunnel losses are only part of the overall hydraulic losses. The frictional losses associated with the reservoir intake, manifold, connection configurations, and individual unit penstocks are unaffected by the new tunnel. Further refinements to the connections and modifications to the intake should be evaluated during final design with the objective of reducing overall system head loss.

For tunnels ranging in diameter from 14 to 22 feet, construction cost estimates were prepared, yielding a range from about $47.0 to $56.7 million—less than a 20 percent difference between the smallest and largest. In addition to cost, JA recommends considering several other factors when selecting a tunnel diameter, including the results of the energy analysis, and final lining assumptions (head loss will increase if greater lengths of tunnel require a lining). A larger tunnel would provide a margin of safety for maintaining the desired hydraulic performance.

SCL's energy and financial analyses have produced positive and encouraging results in terms of additional energy production and the value, in dollar terms, of that additional energy. For a 22-foot diameter tunnel, the average additional energy is approximately 56,000 MWh/year, which equates to an estimated annual financial benefit of nearly $2.7 Million. Taking account of the Renewable Energy Credit increases the potential financial benefit to $4.1 Million per annum.

Based on the various studies and analyses carried out as part of this report, it is recommended that the final design should consist of a configuration consisting of a 22-foot-diameter tunnel with an optimized Link 3 connection and a station flow of 7,500 cfs.

13 References

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All other documents referred to within this report are included in the appendices.

Revision Log

Revision No.	Date	Revision Description
1	August 14, 2009	TDOL and WTS reports combined into single report – issued for SCL comment and review.
2	December 4, 2009	SCL comments incorporated. Additional modeling carried out.
3	April 7, 2010	Draft Final Issue – SCL comments incorporated.
4	July 22, 2010	Efficiency Information Added – Final Issue.
5	May 2011	Updated hydraulics and energy information.