BEATRIZ RESERVOIR INTAKE TUNNEL

Tunnel Feasibility Evaluation

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Table of Contents

1 Introduction 1

1.1 Project Description 1

1.2 Scope of Work 1

1.3 Geology 2

1.4 Anticipated Subsurface Conditions 3

2 Probable Tunneling Methods 3

2.1 Tunnel Boring Machine 4

2.2 Drill-and-Blast Methods 4

2.3 Final Lining Considerations 4

2.4 Probable Tunnel Size 6

3 Construction Staging Areas and Site Constraints 6

3.1 Outlet Portal 6

3.2 Intake Portal 7

4 Potential Project Impacts 8

4.1 Tunnel Instability Due to Encountering Fault Zone 8

4.2 Excessive Groundwater Inflows 8

4.3 Flooding 8

4.4 High Rock Strength 9

5 Probable Range of Construction Costs 9

5.1 Key Assumptions 9

5.2 Conceptual Cost Estimate 9

6 Preliminary Construction Schedule 10

7 Conclusion 10

8 References 11

9 Revision Log 27

List of Tables

Table 1. Anticipated Subsurface Conditions 5
Table 2. Tunnel Construction Cost Estimate Summary 10

List of Figures

Figure 1. Project location map. 12
Figure 2. Tunnel plan and profile. 13
Figure 3. Geologic map of project area. 14
Figure 4. View looking upstream on the Quebrada Sonadora adjacent to the Outlet Portal. 15
Figure 5. Sandstone and Siltstone bedrock outcrop across from the Intake Structure site. 16
Figure 6. View of the Intake Portal site on the west bank of the Rio Turabo. 17
Figure 7. Typical main-beam TBM for rock excavation. 18
Figure 8. Typical excavated tunnel section using drill-and-blast methods. 19
Figure 9. Typical tunnel section with CIP final lining. 20
Figure 10. View of Outlet Portal staging area on the east bank of Quebrada Sonadora 21
Figure 11. Conceptual layout of Outlet Portal construction staging area. 22
Figure 12. Typical staging area at mining portal. 23
Figure 13. View of Rio Turabo and the Outlet Portal site arrangement. 24
Figure 14. Conceptual layout for Intake Portal construction staging area. 25
Figure 15. Preliminary construction schedule. 26

1 Introduction

This report summarizes the results of the site reconnaissance and feasibility evaluation performed by Jacobs Associates for the proposed Beatriz Reservoir Intake Tunnel in Municipio de Caguas, Puerto Rico. The results will be used as background information in the EIS document for the project. The work was performed at the request of Gregory Morris Engineering. Based on the results of the evaluation the proposed tunnel is considered feasible.

1.1 Project Description

The proposed Beatriz Reservoir Intake Tunnel will divert and convey flows, by gravity, from the Rio Turabo into the Beatriz Reservoir. The tunnel will have a 2.4 to 3.0 m finished diameter and will be approximately 3.3 kilometers long. Figure 1 shows the location of the project.

The tunnel will be excavated from a portal located at the back end of the reservoir adjacent to the Quebrada Sonadora, herein referred to as the Outlet Portal. The tunnel slopes up uniformly at about 0.25% to the invert of the river intake herein referred to as the Intake Portal. Figure 2 shows the proposed tunnel alignment plan and profile. Figure 2 also shows preliminary tunnel stationing that is referenced herein. The accuracy of the surface topography, and tunnel and portal elevations discussed herein is limited by the accuracy of the USGS quad sheet topography upon which the profile is based. The invert elevation at this Outlet Portal and Intake Portals will be approximately 151 meters and 159.5 meters, respectively.

The tunnel alignment cuts through forested mountainous terrain to connect the Quebrada Sonadora with the Rio Turabo. The alignment passes beneath four ridges separated by valleys that have been cut by streams. Ground cover above the tunnel invert 250 m beyond each portal ranges from approximately 60 to 215 m (Figure 2).

1.2 Scope of Work

The general scope of work is to evaluate the feasibility of constructing a tunnel between the Rio Turabo and the proposed Beatriz Reservoir. The scope of work included a field reconnaissance to observe rock exposures and to assess surface conditions at the ends of the tunnel. The site reconnaissance was performed on April, 4, 2007. The tunnel feasibility evaluation includes the following items:

- Anticipated tunneling conditions
- Surface conditions at the proposed Outlet and Intake Portal sites
- Probable tunnel construction methods and feasible tunnel size
- Construction staging area considerations and key site constraints
- Project components that involve potentially significant impacts and potential mitigation measures
- Probable range of tunnel construction costs
- Preliminary construction schedule

Information that reviewed as part of our tunnel feasibility evaluation includes:

- Geologic mapping of the Caguas Quadrangle conducted by Rogers (1979)
- Geotechnical Data Report (GDR) and an Addendum (No. 1) for Beatriz Reservoir Dam prepared by GeoConsult (1994).

1.3 Geology

Figure 3 shows an enlargement of the geologic map for the project area, with the location of the proposed alignment and portal locations identified. The following geologic units and faults are anticipated along the tunnel alignment based on existing geologic mapping and the field reconnaissance. The units are listed by location along the tunnel alignment (west to east). Geologic symbols (as shown on Figure 3) are shown in parentheses after the unit name.

1.3.1 Quaternary Alluvium (Qal)

Alluvium consists of uncemented clay to boulder-sized material deposited in major and minor stream channels and broad floodplanes. Quaternary Alluvium was observed at the Outlet Portal site but not along the tunnel alignment. This material is anticipated to consist of silt through boulder-sized material mantling shallow tonalite bedrock.

1.3.2 Tonalite (Kto)

Tonalite bedrock located along the tunnel alignment consists of medium-grained, slightly altered, crystalline intrusive rock of upper Cretaceous age. Jointed (fractured) tonalite bedrock was penetrated in borings conducted along the axis of the Beatriz Reservoir Dam. During the April 4th site visit, hard, slightly altered tonalite bedrock was observed within the Quebrada Sonadora creek bed in the vicinity. Tonalite outcrops are visible in Figure 4. Bedrock jointing in tonalite bedrock at the Outlet Portal area ranged from very close to moderately close spacing (1/2 inch to several feet). Caribbean tonolites are known to have zones for hydrothermal alteration, creating mixed face conditions. This condition does not outcrop because of weathering.

The predominant joint orientation measured in the tonalite during the site visit was N 45° W, 85° NE. This joint orientation parallels the Quebrada Sonadora stream channel at the Outlet Portal location and approximately parallels a fault mapped along this section of this stream. Close jointing was observed near the center of the channel indicating that the mapped fault may be located within the channel at this location.

1.3.3 J Formation (Kj)

The J Formation consists of volcanic breccia interlayered with tuff, a few thin lava flows, and scarce siltstone and sandstone (Rogers, 1979). Both the tuff and breccia contain calcareous cement. The tuff contains highly pumaceous zones. The breccia contains blocks up to 1m in diameter. Andesitic dikes are present in unit. No information is available on the bedding angle or structure of this unit due to limited outcrops but the structure is expected to be chaotic or irregular. TBM tunnels in a similar formation in the Dominican Republic encountered good ground except when faults dumped large amounts of water in the heading for many days. There is a tunnel currently under construction in this unit in the DR called the "Pinalito".

The contact between the J Formation and the younger Tonalite is located in the vicinity of the Outlet Portal, but no exposures of J Formation bedrock were observed. Cobbles of breccia were observed within the Quebrada Sonadora streambed. The breccia cobbles were generally hard and relatively unweathered.

1.3.4 Turabo Fault

The Turabo Fault trends north-south and cuts across the alignment at approximately Sta. 17+80. This fault does not show evidence of movement over the last 50 million years, based on existing geologic mapping (Rogers, 1979). The Turabo Fault is down-dropped to the east and juxtaposes the older J Formation volcanic bedrock against the younger Sandstone and Siltstone of the Robles Formation. Commonly forces exerted on adjacent bedrock along major faults, such as the Turabo Fault, cause intense fracturing on both side of the fault plane. An apparent splay of the Turabo Fault parallels the proposed alignment between Sta. 17+80 and Sta. 29+20.

1.3.5 Metamorphosed Sandstone and Siltstone (Krs)

The sandstone and siltstone of the Robles Formation consists of metamorphosed sandstone and siltstone with some tuff beds and scattered lava flows that is located on the east side of the Turabo Fault (Rogers, 1979). Bedrock outcrops in the vicinity of the Intake Portal on the Rio Turabo consisted of very hard, metamorphosed, interbedded sandstone and siltstone. At this location the unit dips both at approximately 12 to 20 degrees to the northwest and southwest. The orientation of bedding is approximately perpendicular to the proposed tunnel alignment. Figure 5 shows the typical bedded metamorphosed sandstone and siltstone at the Intake Portal site. The bedding shown in this figure dips gently (14 degrees) to the southwest, similar to the mapped orientation of bedding within this unit. Figure 6 shows the outcrop of metamorphosed sandstone and siltstone along the west bank of the Rio Turabo at the Intake Portal.

As shown on Figure 3, an unnamed fault cuts across the proposed tunnel alignment at approximately Sta. 29+20 within the sandstone and siltstone unit (Rogers, 1979). The fault trends N 70° E and is down-dropped to the east.

1.3.6 Terrace Deposits (Qt)

Terrace deposits consisting of older, potentially weathered and/or weakly cemented alluvial deposits mantle the hill slope along the west bank of the Rio Turabo. Large boulders through silt-sized material were observed. These deposits overlie metamorphic interbedded sandstone and siltstone bedrock. The bedrock contact extends to approximately 3 m above the river level at the Intake Portal location (see Figure 6) so this unit is not at the tunnel horizon.

1.4 Anticipated Subsurface Conditions

Table 1 summarizes subsurface conditions that are anticipated during tunnel construction based on the geologic reconnaissance, previous field investigations, and existing geologic mapping. Major construction issues and adverse ground conditions are also identified. Adverse ground conditions are only expected for narrow intervals along the tunnel. Approximate geologic contacts are shown for each of units/features based on existing geologic mapping and site observations. The majority of tunnel is anticipated to encounter strong, intact rock that will require minimal support. Geologic contacts, faults and shears, when encountered, will likely require more robust support and groundwater control efforts to maintain a stable excavation. These zones are anticipated to be of narrow extent due to the extensional tectonic environment. If the project moves forward, additional geotechnical drilling and photo/surface mapping will be required to refine the location of these features and better define the characteristics of the bedrock and groundwater conditions along the alignment. Preliminary borehole location recommendations are shown on Figure 2.

2 Probable Tunneling Methods

This section describes tunnel excavation and lining methods that are considered generally applicable to this project. Drill-and-blast and tunnel boring machine (TBM) techniques are considered the most probable methods to excavate the tunnel. The contractor's selection of the excavation method will be primarily driven by economic factors. Typical tunnel final lining options for a gravity tunnel of this size include an unlined tunnel where the ground and groundwater conditions are favorable; shotcrete and cast-in-place concrete (CIP) used where ground stability and or water loss issues are critical. The general applicability and limitations of these methods to this project are discussed in the following sections.

2.1 Tunnel Boring Machine

A TBM is considered the most efficient method for excavating the intake tunnel because the TBM will provide more continuous excavation and support operations that can generally sustain higher overall rates of advance than drill-and-blast methods. The type of TBM that would likely be used for this project is an "open" main-beam TBM. Figure 7 shows a typical "open," main-beam TBM. The machine is advanced by hydraulic rams supported by grippers between the main-beam of the machine and the rock wall. Initial support systems for this type of machine are installed directly behind the cutterhead and usually consist of various combinations of rock bolts, mesh and steel ribs depending on the ground conditions. TBM's can excavated a smooth, circular bore which offers hydraulic advantages, particularly if the tunnel is left unlined, as well as some advantages for construction of the initial supports and final lining. One such advantage is that TBM excavations minimize rock mass disturbance, which reduces ground support and final lining requirements. Used TBM's similar to the one shown in Figure 7 are normally available and could be refurbished and mobilized for this project. The main disadvantages of a TBM are high capital cost of the machine and a potentially long mobilization time to Puerto Rico.

2.2 Drill-and-Blast Methods

Drill-and-blast methods are used mainly for the excavation of tunnels in hard rock. Drill-and-blast construction generally includes of four steps: 1) drilling a pattern of holes in the tunnel face and loading the holes with explosives, 2) blasting the round and ventilating blasting gases, 3) mucking the blasted rock, and 4) installing initial ground support as needed. The main disadvantages of drill-and-blast construction include 1) additional disturbance and loosening of the rock mass; 2) the unavoidable breaking of ground beyond the intended excavation lines which constitutes an increase in materials costs for tunnel lining and support and requires "smoothing" of the excavated perimeter with shotcrete or concrete for hydraulic efficiency; and 3) an uneven excavation profile which reduces the efficiency of ventilation and increases tunnel support costs. Figure 8 shows a typical tunnel excavated by drill-and-blast. The main advantage of drill-and-blast methods is that long lead times are not required to acquire the equipment needed for construction. This method is typically slower compared to other mechanical excavation techniques such as TBM's for a single heading. In our experience, drill-and-blast tunneling is usually limited to tunnel drives less than 1 mile in length; beyond this length a mechanical excavation method is generally considered more efficient. Advance rates for drill-and-blast tunneling are typically limited to around 15 to 30 feet per day per heading, depending on the tunnel size and number of shifts worked per day. If two headings can be maintained and a rail based mucking system is used the total advance rates can often meet or exceed TBM excavation rates.

2.3 Final Lining Considerations

The primary functions of a final tunnel lining for this project will be to (1) prevent erosion/deterioration of the rock around the tunnel; (2) provide a structural support system capable of withstanding the anticipated external loads; and (3) provide a smooth interior surface to improve hydraulic performance. Typical tunnel final lining options for a gravity tunnel of this size include an unlined tunnel where the ground conditions are favorable, shotcrete, and cast-in-place concrete (CIP).

As shown in Figure 8, drill-and-blast excavations can result in a very rough flow profile and it is often necessary to line the tunnel with concrete or shotcrete to improve the hydraulic performance. As a minimum it is anticipated that the invert would be lined in a drill-and-blast tunnel. In many cases a TBM can excavate a smooth bore which can be left unlined. Some basic considerations for the final lining systems discussed in more detail in the following subsections.

Table 1. Anticipated Subsurface Conditions

approximate Station (m)	Geologic Unit/Feature	Material	Construction Issues
Outlet Portal excavation	Quaternary Alluvium	Silt through boulder-sized material mantling shallow tonalite bedrock	• Low strength material that requires additional tunnel and portal (shaft) support. • High groundwater levels in soil at Outlet portal may require groundwater control to maintain stable cut slopes.
1+50 to 2+40	Tonalite	Hard, fractured crystalline tonalite	• Highly fractured tonalite is anticipated at the Outlet Portal due to nearby faulting. • Exact location of contact with J Formation along tunnel alignment unknown. • Potentially highly altered and/or weathered contact with J Formation may reduce rock quality and tunnel stability. • Potential for groundwater inflows along contact with J Formation and along unmapped faults (if present).
2+40 to 17+80	J Formation	Volcanic breccia interbedded with tuff, a few thin lava flows, and scarce siltstone and sandstone	• Potential for differential rock hardness in mixed materials along tunnel alignment: • Highly fractured bedrock zones, if present, will require additional initial ground support. • Potential for localized heavy groundwater inflows along contacts between tuffs and dikes/flows, along contacts with siltstone lenses, and along fractured/faulted zones where there is a source of recharge above the tunnel, • Unmapped fault zones may be present within the J Formation that will reduce tunnel stability and may channel groundwater inflows into tunnel excavation. • Impact to existing water wells above tunnel alignment.
17+80	Turabo Fault	Fractured and sheared bedrock and fault gouge	• Bedrock in the vicinity of the fault is anticipated to be highly fractured. • Soil-like material along the intersection of the fault trace. • Bedrock stability is anticipated to be reduced in the vicinity of the fault zone. Additional temporary tunnel support is expected to be required to stabilize the excavation. • Significant groundwater inflow is anticipated along fractured rock within tunnel excavations in the fault zone.
17+80 to Intake Portal (Sta. 32+45.91)	Sandstone and Siltstone of the Robles Formation	Hard, fractured, metamorphosed, interbedded sandstone and siltstone	• Diorite porphyry dikes that cut this unit may have altered and/or fractured adjacent sandstone and siltstone beds reducing rock quality. Dikes may also serve as a vertical conduit for groundwater into tunnel excavation.
29+20	Unnamed Fault	Fractured and sheared bedrock and fault gouge	• Bedrock in the vicinity of the fault is anticipated to be highly fractured. • A shear zone containing soil-like material is expected along the intersection of the fault trace. The width of the shear zone, if present, is unknown. • Bedrock stability is anticipated to be reduced in the vicinity of the fault zone. Additional temporary tunnel support is expected to be required to stabilize the excavation. • Significant groundwater inflow is anticipated along fractured rock within tunnel excavations in the fault zone.
30+60 to 32+42 (Above tunnel alignment)	Terrace Deposits	Potentially weathered and/or weakly cemented older alluvial deposits consisting of silt through boulder-sized material	• Terrace deposits consist of low strength materials that require additional ground support within Intake Portal and Exit Shaft excavations. • Boulders that may impact construction of ground support in the Exit Shaft excavation. • High groundwater levels in soil at Exit Shaft and Intake Portal require dewatering or watertight excavations.

2.3.1 Shotcrete Lining

A shotcrete final lining typically consists of a continuously applied, full circumference shotcrete lining extending from either side of the tunnel invert. The shotcrete would typically be reinforced to control crack widths and to improve the flexural strength of the lining.

2.3.2 Cast-in-Place Concrete

Construction of a CIP 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 removal of the forms after the concrete has set. The lining is designed to resist external loads; including ground loads, grouting pressures, and groundwater pressures. The typical thickness of a CIP concrete lining is 8 to 12 inches which is an important consideration for selecting the excavated tunnel diameter. Figure 9 shows typical CIP tunnel sections for TBM and drill-and-blast methods.

2.3.3 Final Lining Assumptions

Based on the information collected to date, it is anticipated that the majority of the tunnel can be unlined if excavated with a TBM. In areas with weak rock conditions a concrete invert or a CIP lining may be necessary to protect the wetted surface from long term erosion and deterioration. In poor ground conditions, associated with shears and faults, a CIP concrete lining may be necessary to resist ground loads and also to control leakage that could further reduce tunnel stability. For drill-and-blast methods, the all of the tunnel is anticipated to be lined with a concrete invert to provide a smooth invert. A CIP concrete or shotcrete lining will be required in areas that are supported by steel ribs and mesh to protect the steel from corrosion in both TBM and drill-and-blast tunnels.

2.4 Probable Tunnel Size

It is our understanding that the finished diameter require to convey the design flow is about 1370 mm. This diameter is significantly smaller than the diameter considered necessary to efficiently excavate the tunnel. Therefore, the minimum excavation diameter will be primarily controlled by construction factors such as the size of tunneling equipment selected by the contractor, the type of muck removal system selected by the contractor, thickness of initial support systems, and the required size of ventilation systems. Construction clearances between utilities, locomotives, initial support systems and other tunnel facilities will also be considered by the contractor in sizing the tunnel. The minimum excavated diameter for a TBM tunnel is expected to range between 2.5 and 3.3 m. If drill-and-blast methods are used the minimum excavated dimension is expected to be about 6 ft wide by 7 ft tall.

3 Construction Staging Areas and Site Constraints

This section describes concepts, layouts and construction consideration for the tunnel portals.

3.1 Outlet Portal

The Outlet Portal would be used as the mining portal since few restrictions are expected on the location and size of the staging area because the area will be inundated by the new reservoir. Other reasons this site is the preferred location to stage tunnel construction include:

- Tunneling uphill allows for groundwater inflows to drain out of the tunnel by gravity.
- Access roads can be easily constructed within the reservoir site.
- Tunnel muck can be less expensively transported and stockpiled in the reservoir site for use in dam construction or stockpiled for sale as commercial aggregate.

The major site constraint is the proximity to the Quebrada Sonadora and the potential for river flooding. Figure 10 shows the location of the portal and staging area relative to the channel and east bank of the river. The portal and staging area will need to be located above the east bank of the river channel in order to provide some level of flood protection. In addition, the freeboard can be increased along the east bank of the river by constructing a barrier or berm. Upon completion of the tunnel, it is expected that an outlet structure and concrete channel would be constructed to convey the flow from the tunnel into the Quebrada Sonadora.

Figure 11 shows a conceptual layout of the Outlet Portal site. A total staging area of one to two acres is desirable for tunnel construction. The staging area would be graded to develop level areas to construct a tunnel portal and staging area for tunnel construction. To gain access to the tunnel portal the work would include both cut and fill construction along the east side of the river to create access roads, level pads, and crossings of several tributary creeks.

Figure 12 shows a typical staging area around a mining portal that is considered applicable to this project. The staging area will include space for the following elements:

- Office trailers for Owner/Designer, Contractor, Inspectors, and change rooms (dry house) for workers
- Parking and turn around areas
- Maintenance shop, tool containers
- Crane, loader, generator, and other surface support equipment
- Material stockpile area (including initial and final lining materials)
- Tunnel water handling storage ponds/tanks and treatment facility
- Temporary muck stockpile area

3.2 Intake Portal

The invert of the tunnel at the intake structure is anticipated to be about 1.5 m below the river level. During the field reconnaissance concerns arose about constructing a daylight portal adjacent to the river due to the risk flooding the tunnel during high river flow events. A concept was developed for an exit shaft located on top of the northwest bank of the river to isolate tunnel construction from flooding. This shaft would temporarily serve as the end of the tunnel until completion of intake structure. The shaft would be used to remove tunnel excavation equipment, provide access and ventilation for tunnel lining activities, and provide staging for construction of a connection tunnel over to the intake structure. Figure 13 shows location of the exit shaft and intake portal.

3.2.1 Exit Shaft

Figure 14 shows a conceptual layout of the exit shaft site. The size of the shaft excavation would be sized by the contractor to retrieve tunneling equipment, provide tunnel ventilation, support tunnel lining activities, and to construct the tunnel connection with the Intake Structure. It is anticipated that the shaft will have a minimum diameter of about 20 feet. Although the majority of construction staging will occur at the outlet portal, designated work areas will be required around the shaft for equipment storage, groundwater handling, spoils stockpile and loading area, and a truck and trailer turn around area. It is anticipated that the shaft would be excavated through the terrace deposits using mechanical methods and that drill-and-blast methods would be required in the rock.

3.2.2 Intake Structure

The intake structure will be located on the west bank of the Rio Turabo next to a plunge pool (see Figure 5). A small staging area at the structure site will be required. To gain access to the site a temporary construction crossing of the river will be necessary. Additional staging area is available on the east bank of the river next to an existing abandoned building. It is anticipated that some surface excavation and scaling will be required to expose and clean the rock in the slope behind the structure. In addition, a

temporary diversion dam will be required along west side of the pool to excavate the foundation for the structure.

4 Potential Project Impacts

Tunnel construction projects, in general, carry several types of risk which can impact the success of the project. Risks may involve geotechnical, equipment, environmental, and operational issues that ultimately affect the schedule, cost, quality, and safety of the project. The approach for conducting a risk analysis involves identifying the important potential risks for the project, evaluating the consequences of these risks, and developing a strategy for mitigating the significant risks. This project is not defined well enough to conduct a formal risk analysis. However, based on our review of the existing project information and the field reconnaissance we have identified some significant potential construction impacts that will be important to address if the project moves forward.

4.1 Tunnel Instability Due to Encountering Fault Zone

One of the more significant risks associated with tunneling involves encountering a fault zone containing sheared rock. Commonly, these zones are composed of weaker rock that is highly fractured. Fractured rock reduces tunnel stability and requires additional ground support. The presence of flowing groundwater can further reduce tunnel stability in these zones. Impacts of encountering fault zones in tunnel excavations include project delay to advance past sheared zones, increased labor costs due to delay, increased construction costs for additional ground support, and a claim of differing site conditions. These zones also typically require a more robust final tunnel lining.

4.2 Excessive Groundwater Inflows

Excessive groundwater inflow into tunnel excavations can increase construction costs and reduce tunneling advance rates. Abundant inflow may also reduce tunnel stability. Groundwater inflow into the tunnel will occur through joints, fractures and shears in the rock. Additionally, faulted zones containing sheared bedrock can act as groundwater conduits that channel high volume “flush” flows into tunnel excavations. Large water inflows that are difficult to handle and can overwhelm water treatment facilities back at the mining portal. Since groundwater 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, recharge area at the ground surface, the primary method to mitigate or control this risk is to perform investigations to accurately characterize groundwater levels and rock permeability.

If concerns arise over excessive groundwater inflows or adverse groundwater effects on excavations stability, 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 injection of 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. Drilling drain holes to lower the seepage rates may be effective if borings show limited recharge.

4.3 Flooding

Both portals are located adjacent to rivers that flood outside their banks. The portals will need to be protected or isolated from flooding. The impacts of floodwater inundation at portals include safety risk to construction workers, damage to equipment and facilities, washed out access roads, all of which could shut the job down. As discussed above, an exit shaft concept was developed above the Rio Turabo in lieu of an at grade portal to isolate the tunnel from the risk of flooding. Similarly, the mining portal and

staging area is located above the east bank of the Quebrada Sonadora. It will be important to understand the magnitude and frequency of high flow events when assessing the flood protection measures that will be implemented at each portal.

4.4 High Rock Strength

The strength and abrasion characteristic of the rock is an important parameter for evaluating excavation advance rates. Field observations of rock outcrops indicate that high strength rock will be encountered in the tunnel. During construction higher than anticipated rock strength can result in slower advance rates, high equipment wear, and ultimately result in a claim for a differing site condition. It will be important to investigate and evaluate the range of rock strengths that will be encountered. In addition, boreability and petrographic testing of rock samples is considered effective for determining abrasion characteristics. Understanding the rock strength and abrasion characteristics and providing the information to a contractor will be important to reduce the risk of a claim for a differing site condition.

5 Probable Range of Construction Costs

This section summarizes the key assumptions for the development of a probable cost range for the tunnel construction. A production-based estimate was prepared for the tunnel. This type of estimate accounts for labor, equipment, and construction methods in the same way that a contractor would bid the work. This approach segregates the work into discrete tasks, which in turn requires development of crew sizes, equipment spreads and production rates for each task. The estimate assumes the tunnel will be excavated with a TBM because this method is considered the most efficient and economical method and it is expected that it will allow significant portions of the tunnel be unlined. However, if a second tunnel heading can be maintained from the Rio Turabo side, drill-and-blast methods from two portals may compete with a TBM operation and would likely attract more bidders. If explorations show favorable ground conditions the cost and schedule for drill-and-blast could become significantly more attractive.

5.1 Key Assumptions

- Total tunnel length equals 3.3 km.
- Cast-in-place concrete lining for 30% of the tunnel.
- Unlined tunnel for 70% of tunnel.
- Refurbished 11-foot diameter main-beam TBM. This size TBM is normally available in the marketplace, but it is expected that it may require 9 to 12 months of lead time from notice to proceed to arrive on site.
- The starter and connector tunnels will be excavated using drill-and-blast methods.
- Muck transport performed by unit muck trains.
- Muck will be disposed of on site.
- TBM retrieved from a 17 m deep exit shaft.
- Work is performed 5 work days per week and two 10 hour shifts per day.
- Average overall TBM advance rate is 50 feet per day.
- Wage rates based on Federal Davis-Bacon Wages for Puerto Rico assuming some skilled miners from mainland.
- Site work performed by others.
- Intake structure performed by others.

5.2 Conceptual Cost Estimate

Table 2 summarizes the probable cost range for the tunnel construction. Key cost elements for tunnel construction are also shown. Direct costs are costs that are attributed to a specific activity. These costs

include categories such as hourly labor charges, equipment operating costs, and materials and supplies. Indirect costs are costs that are chargeable to the project but are applied across various activities. Field supervision, equipment ownership, home office overhead, bonds and insurance, and other general costs for the project are included in this type of cost. The estimate does not include escalation or owner's expenses such as design costs, construction management, property and right-of-way acquisition, and permits.

Table 2. Tunnel Construction Cost Estimate Summary

Cost Item	Estimated Cost
Tunnel Excavation/Support	$ 10,600,000
Tunnel Final Lining	$ 3,300,000
Excavated/Support Exit Shaft	$ 580,000
Excavate/Support Connector Tunnel	$ 140,000
Direct Costs	$ 14,600,000
Indirect Costs	$17,200,000
Direct + Indirect	$ 31,800,000
Contingency (10% to 50%)	$ 3,200,000 to $16,000,000
Construction Cost Range	**$ 35,000,000 to $47,000,000 ($10,600 to $14,250/meter)**

6 Preliminary Construction Schedule

Figure 15 is a schedule that forecasts completion dates of the major construction activities. The total project duration after NTP is estimated to be 31.3 months. This schedule also includes an estimate of the monthly progress payments during tunnel construction. The estimated progress payment amounts do not include the contingencies.

7 Conclusion

Based on our evaluations, a tunnel alignment between proposed Beatriz Reservoir and the Rio Turabo is feasible. Available information indicates that a suitably located tunnel would be located in competent bedrock. Drill and blast and TBM methods are suitable for excavating the tunnel. However, TBM excavation is considered more efficient and economical and is considered in the cost estimate. If explorations show favorable ground conditions and two tunnel headings are possible, drill-and-blast could become a more favorable method. The rock strength, groundwater, and soft ground conditions within fault zones will be risk factors that need to be addressed during design. These factors will impact advance rates, initial support requirements, and the final lining requirements.

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Rodgers, C.L., 1979, Geologic Map of the Caguas Quadrangle, Puerto Rico: U.S. Geological Survey Miscellaneous Investigations Series Map I-1152, 1:20,000

Figure 1. Project location map.

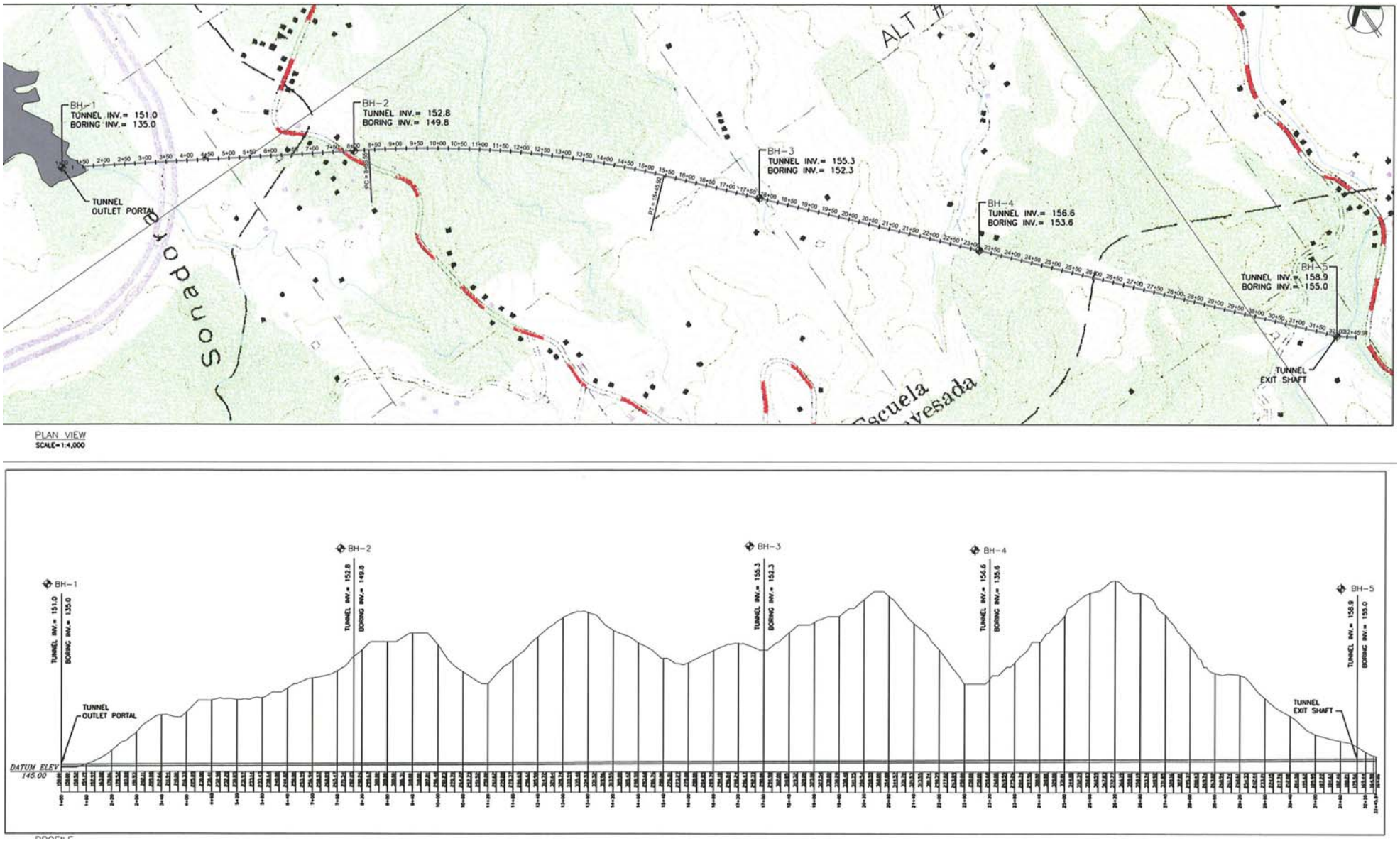

Figure 2. Tunnel plan and profile.

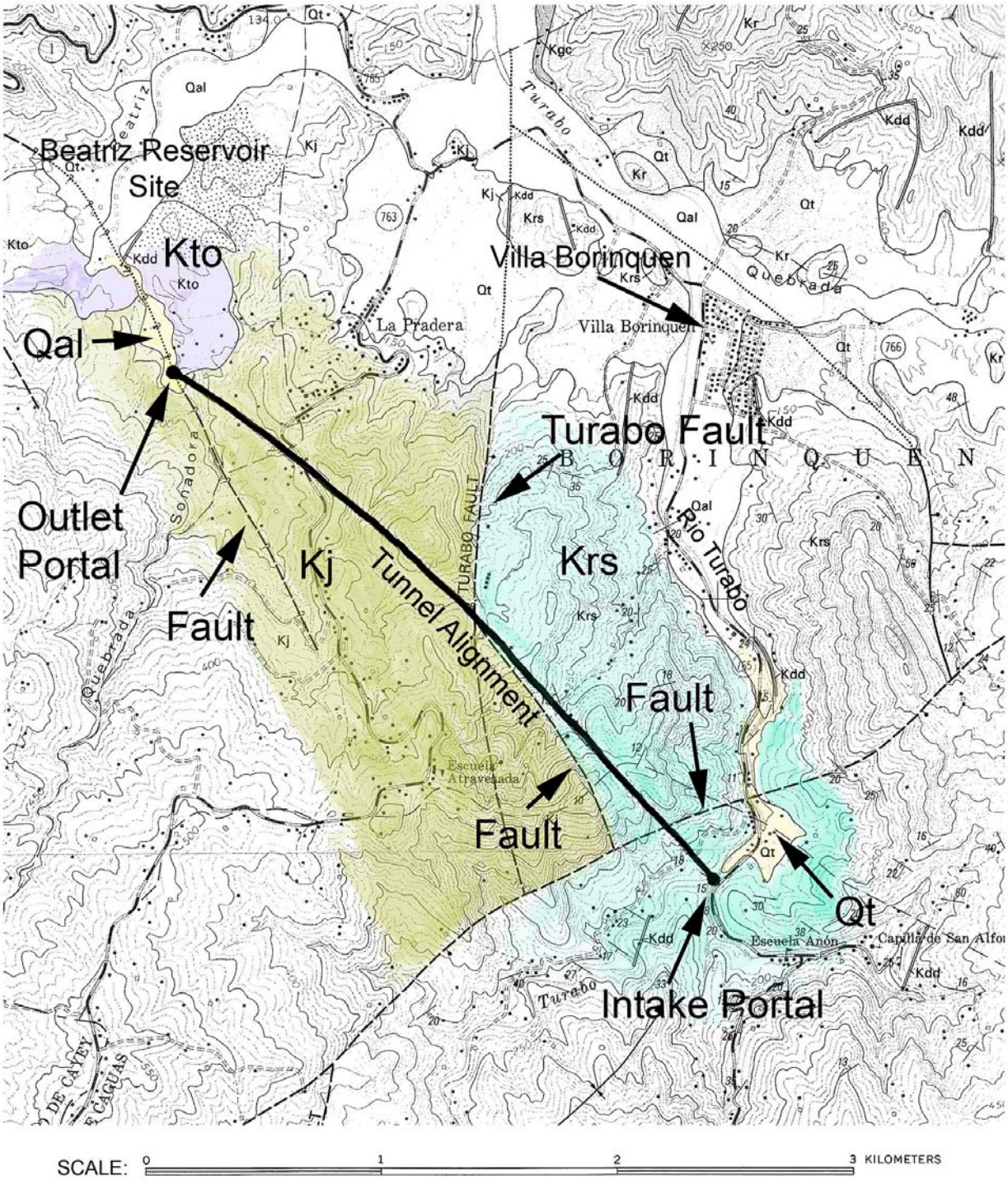

Figure 3. Geologic map of project area.

Figure 4. View looking upstream on the Quebrada Sonadora adjacent to the Outlet Portal.

Figure 5. Sandstone and Siltstone bedrock outcrop across from the Intake Structure site.

Figure 6. View of the Intake Portal site on the west bank of the Rio Turabo.

Figure 7. Typical main-beam TBM for rock excavation.

Figure 8. Typical excavated tunnel section using drill-and-blast methods.

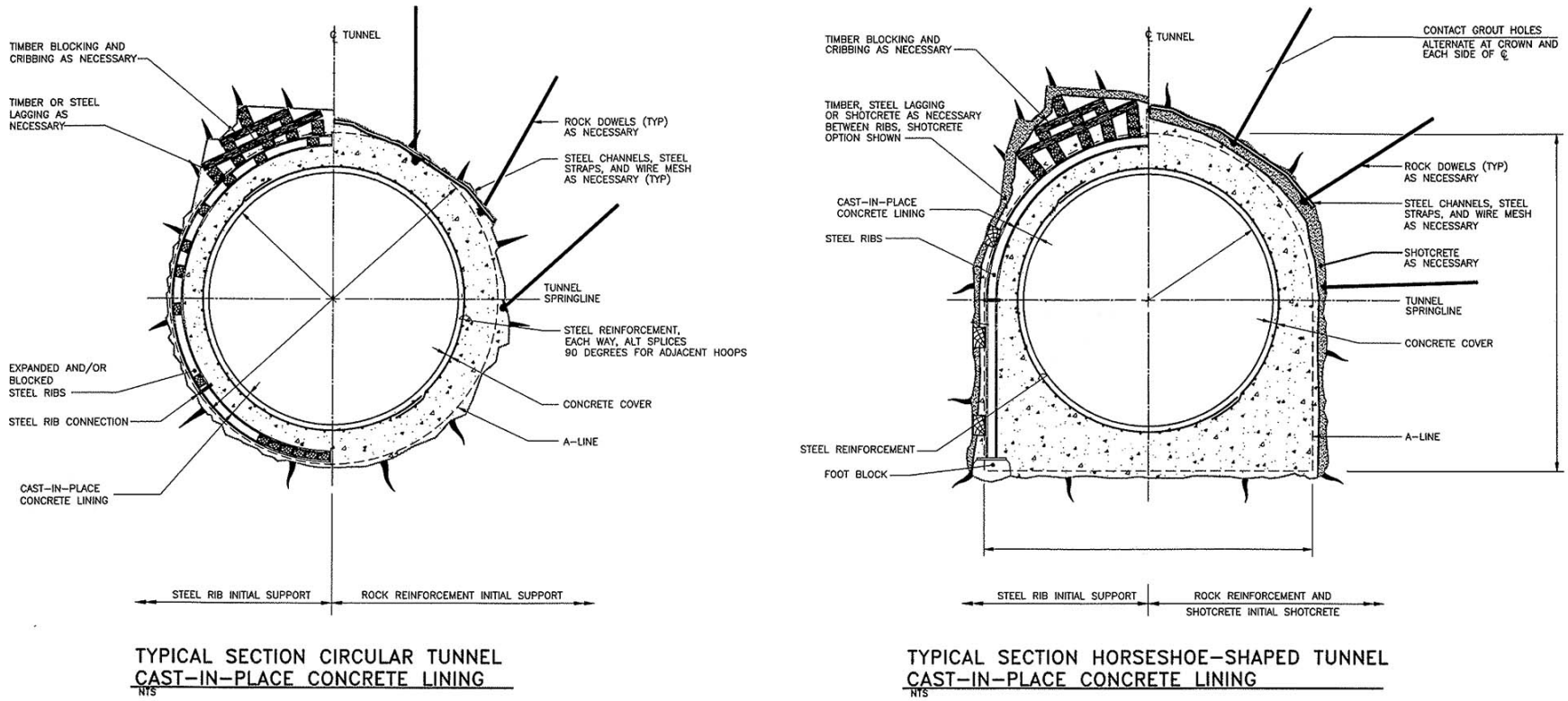

Figure 9. Typical tunnel section with CIP final lining.

Figure 10. View of Outlet Portal staging area on the east bank of Quebrada Sonadora

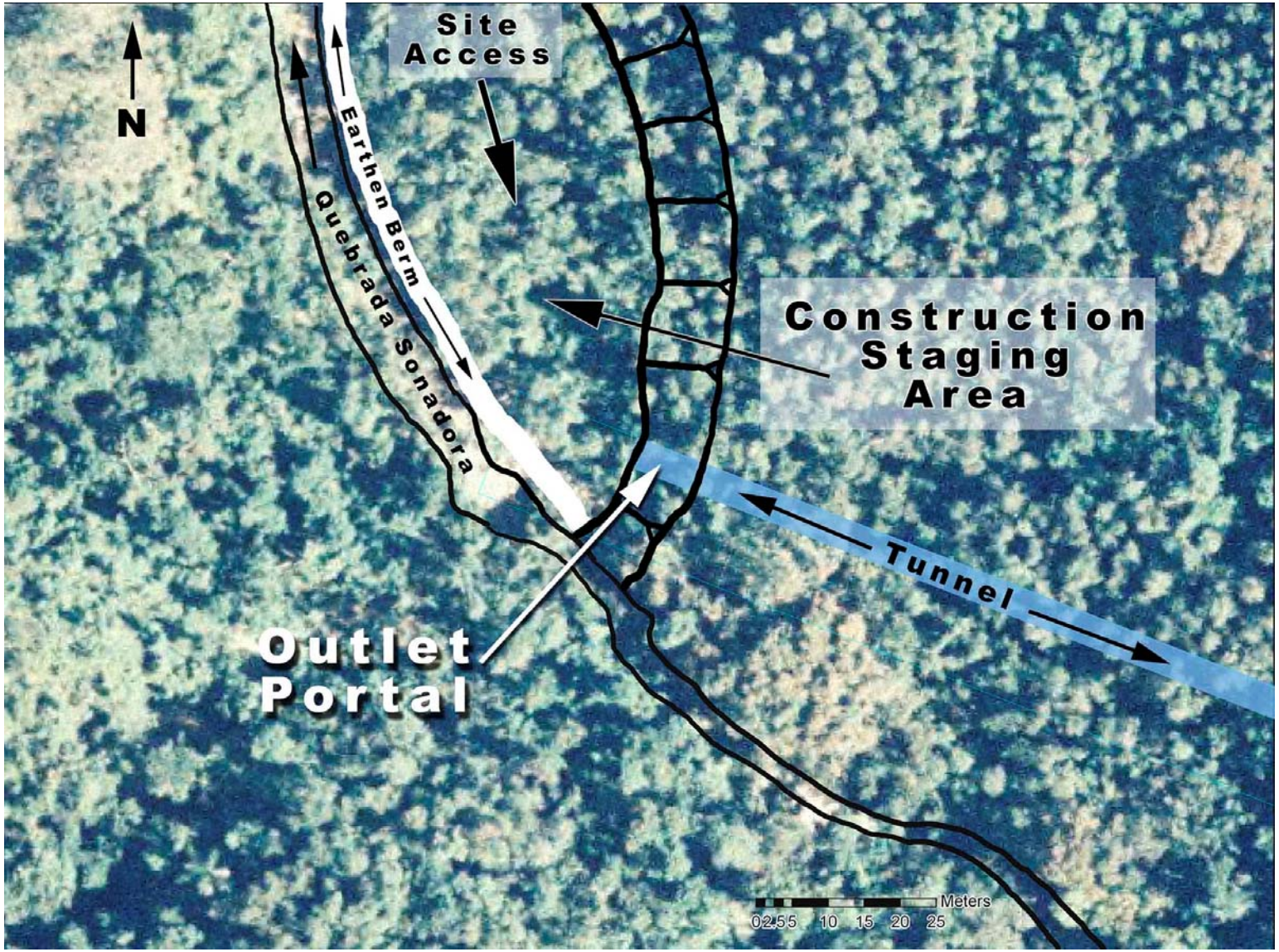

Figure 11. Conceptual layout of Outlet Portal construction staging area.

Figure 12. Typical staging area at mining portal.

Figure 13. View of Rio Turabo and the Outlet Portal site arrangement.

Figure 14. Conceptual layout for Intake Portal construction staging area.

Jacobs Associates ♦ Engineers/Consultants
San Francisco ♦ San Diego ♦ Pasadena ♦ Seattle ♦ Portland ♦ Las Vegas ♦ Boston

Project	Client	Job No.	Estimator	Rev	Date
Beatriz Resevoir Intake Tunnel - Feasability Study **Conceptual Design Estimate**	**Greg Morris Engineering**	**4038.0**	**PWD**	**0**	**4/17/2007** **4:25 PM**

CONSTRUCTION SCHEDULE AND MONTHLY PROGRESS PAYMENTS

Activity No. and Description	Bid Amount	Dur. (mos)	Start mo	End mo	YEAR 1	YEAR 2	YEAR 3	YEAR 4	YEAR 5
N NTP			6.00		NTP: Month 6.0				
M1 Project Complete			37.25					Project Complete: Month 37.3	
S Submittals		2.00	6.00	8.00					
011 Equipment Mobilization	$0	11.75	6.50	18.25					
012 General Mobilization	$949,280	2.00	13.75	15.75					
013 Demobilization	$818,810	1.00	36.25	37.25					
001 Excavate/Support Starter Tunnel (Note 4)	$351,837	0.75	17.00	17.75					
002 Erect Tunneling Plant	$481,322	0.50	17.75	18.25					
003 Assemble TBM / Trailing Gear	$929,774	1.00	18.25	19.25					
004 Excavate/Support TBM Tunnel	$19,219,317	10.25	19.25	29.50					
005 Remove TBM / Backup and Plant	$893,582	1.00	29.50	30.50					
006 Install CIP Concrete Lining	$6,014,645	4.00	30.50	34.50					
007 Contact Grout CIP Lining	$550,202	0.50	34.50	35.00					
008 Tunnel Final Cleanup/Patching	$282,884	0.75	35.00	35.75					
009 Exc/Support Retrieval Shaft (Note 4)	$1,186,257	1.25	15.75	17.00					
010 Excavate/Support Connector Tunnel (Note 4)	$282,601	0.50	35.75	36.25					
Opinion of Probable Unescalated Construction Bid, Duration, and Monthly Progress Payments	**$31,960,510**	**31.3 months**			$0.0 K	$106.8 K, $427.2 K, $533.9 K, $854.1 K, $533.2 K, $844.2 K, $1,474.9 K, $1,687.5 K, $1,687.5 K, $1,687.5 K $9,836.9 K	$1,687.5 K, $1,687.5 K, $1,687.5 K, $1,687.5 K, $1,687.5 K, **$2,424.1 K**, $1,138.7 K, $1,428.5 K, $1,428.5 K, $1,428.5 K, $1,236.9 K $19,210.5 K	$403.0 K, $717.6 K, **$1,792.5 K** $2,913.1 K	$0.0 K

1. Project calendar is based on 9 holidays/year. Data are rounded to the nearest 1/4 month.
2. Payments are made 30 days after receipt of payment applications and reflect a 10% deduction for retention, which is reduced to 5% at 75% of completion, estimated at 24 months after NTP (payments releasing retention bolded).
3. Indirect costs to be spread to direct cost activity items comprise $17,234,193 in costs less $1,768,090 in costs for items 011-013; or $15,466,103.
4. A schedule factor has been applied to this activity.

Figure 15. Preliminary construction schedule.

9 Revision Log

Revision No.	Date	Revision Description