New Irvington Tunnel Design Challenges

Glenn Boyce
Jacobs Associates, San Francisco, California, USA

Steve Klein
Jacobs Associates, San Francisco, California, USA

Yiming Sun
Jacobs Associates, San Francisco, California, USA

Theodore Feldsher
URS Corporation, Oakland, California, USA

David Tsztoo
San Francisco Public Utilities Commission, San Francisco, California, USA

ABSTRACT: The San Francisco Public Utilities Commission (SFPUC) is designing the 5.6-km-long (3.5-mi-long) New Irvington Tunnel from the Sunol Valley to the City of Fremont in Alameda County, California, with a minimum finished diameter of 2.6 m (8.5 ft). The tunnel will have a two-pass lining system—an initial support system (such as steel sets) and a final lining consisting of steel pipe, concrete pipe, or cast-in-place concrete. Ground conditions are anticipated to be difficult and highly variable, with groundwater heads of 113 m (370 ft) and potential inflows as high as 95 L/sec (1,500 gpm). This paper discusses some of the issues faced during design of the tunnel, including potential risks associated with construction.

PROJECT DESCRIPTION

The existing Irvington Tunnel (EIT) was constructed between 1928 and 1931 and extends approximately 5.6 km (3.5 mi) from the Sunol Valley to Fremont in Alameda County, California. The tunnel has a finished diameter of 3.2 m (10.5 ft). The east end connects with the Alameda Creek Siphons and the west end connects with the Bay Division Pipelines. The portals are the Alameda West Portal (east end) and the Irvington Portal (west end). Flow through the tunnel is from east to west.

The EIT and the Alameda Creek Siphons are critical lifeline components of San Francisco's Hetch Hetchy Water System, carrying approximately 85 percent of all of the water delivered to the City of San Francisco's customers. The tunnel and siphons are located between two active faults—the Hayward fault and the Calaveras fault. Movement on either of these faults during a major earthquake could seriously damage the tunnel and siphons and disrupt flow to the City's customers.

Because of the need to maintain continuous operations, the tunnel has not been taken out of service for inspection and maintenance in over 43 years (since 1966). To provide seismic reliability and ensure reliable delivery of high quality water to all of its customers, the SFPUC plans to construct the New Irvington Tunnel approximately parallel to the existing tunnel (Figure 1).

The NIT is expected to encounter difficult and highly variable ground conditions. The rock mass is generally composed of weak, intensely fractured and sheared sedimentary rocks (mainly sandstone, siltstone, interbedded siltstone/sandstone, and shale), and also includes some sections of stronger and more massive rock. Along the proposed alignment, the tunnel will also intercept a number of fault zones with abundant clay gouge. The EIT encountered running, caving, flowing, raveling, and squeezing ground in a number of areas along the alignment. Heavy groundwater inflows were also encountered in the EIT during excavation, with reported portal flows that ranged from 500 to 2,000 gallons per minute (gpm).

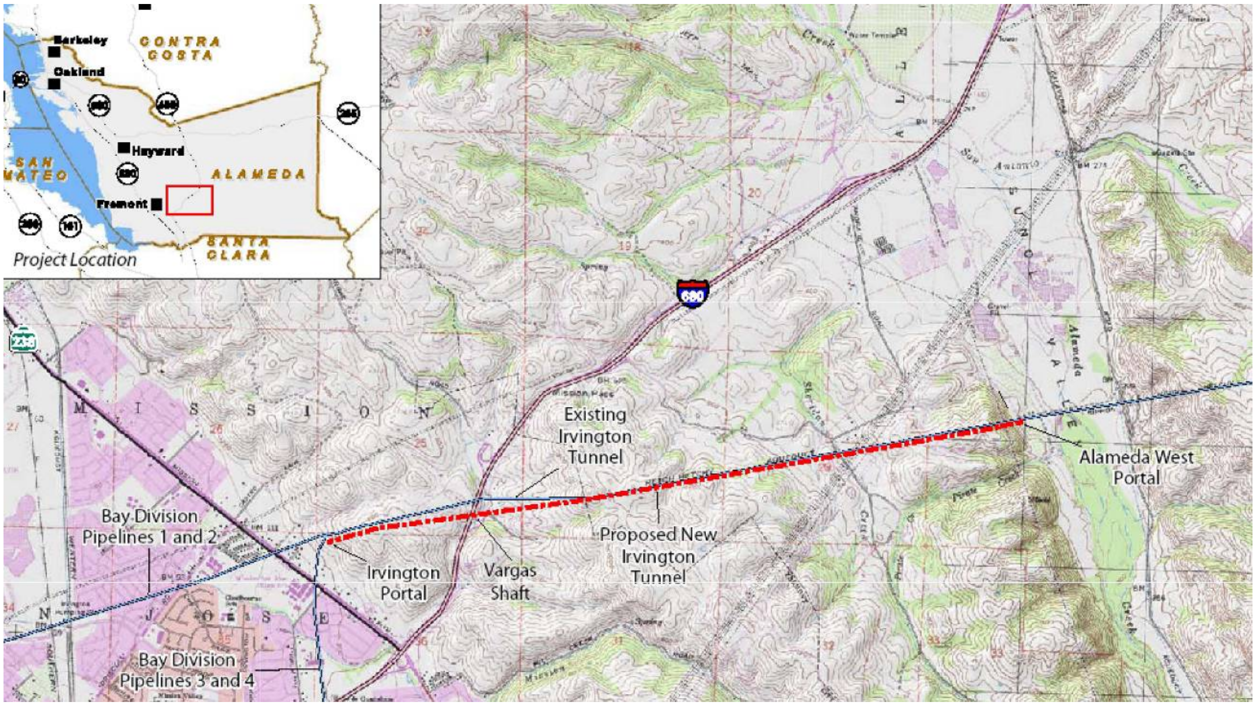

Figure 1. Location map of the New Irvington Tunnel project

The concrete and steel final lining for the new tunnel will serve as the water conduit and steel pipe sections will connect to a combination of new and existing steel pipelines at each portal. The new facilities will include an overflow shaft to control the maximum hydraulic grade line in the tunnel and in the downstream pipelines. The tunnel extends about 5.6 km (3.5 mi) from a new Alameda West Portal to a new Irvington Portal. On its east (Alameda Creek) end, the new tunnel will connect to Siphons 1, 2, and 3, and a new Siphon 4. On its west (Irvington) end, the new tunnel will connect to the existing Bay Division Pipelines (BDPLs) Nos. 1, 2, 3, and 4, and a new pipeline No. 5.

The NIT alignment is about 58 m (190 ft) south of the EIT from the Alameda West Portal (Sta.41+40) to Sta. 154+00, for a distance of about 3,432 m (11,260 ft). From Sta. 154+00 to Sta. 183+50, the horizontal separation between the NIT and the EIT alignments increases to a maximum distance of about 204 m (670 ft). To the west of this point, near where the tunnel crosses I-680,, the separation between the NIT and EIT gradually decreases to zero at Sta. 224+00, where the NIT alignment crosses below the Irvington Portal of the EIT. The NIT extends along this bearing north of the EIT for the remaining 122 m (400 ft) of the tunnel.

The NIT vertical alignment has two slopes. From the Alameda West Portal face (Sta. 41+40) to Sta. 200+00, the design slope is 0.00125. West from this point, the design slope is 0.029. The design invert elevation of the NIT varies from about El. +93 m (305 ft) at the new Alameda West Portal to about El. 62 m (202 ft) at the new Irvington Portal. The NIT will be lower than the EIT for its entire length. The vertical separation between the EIT and NIT alignments ranges from about 9 m (30 ft) at the Alameda West Portal to 37 m (120 ft) at the Irvington Portal.

The Alameda West Portal (AWP) provides access for constructing one of the NIT headings, connecting to the Alameda Siphon Mixing Manifold, and constructing the NIT portal access structure. In addition, the Alameda West Overflow Shaft will be constructed on the hillside above the portal. The NIT will connect to the Alameda Siphon Mixing Manifold via a connecting pipeline, intersecting the new tunnel portal pipe with a wye section.

A temporary construction shaft, referred to as the Vargas Shaft, will be constructed on the east side of I-680 at Vargas Road (Figure 1). The purpose of this shaft is to provide access for the excavation of two tunnel headings in the NIT. One heading will be mined to the west towards the Irvington Portal. The other heading will be mined to the east towards the new Alameda West Portal. The shaft size will be determined by the contractor, and is anticipated to be about 10.7 to 12.2 m (35 to 40 ft) in diameter. The shaft depth will be about 36.6 m (120 ft).

The Irvington Portal provides access for construction of an eastward tunnel heading and to connect the tunnel with the Bay Division Pipelines. The Irvington Portal is located at Sta. 228+00, where the tunnel has an invert elevation of about El. 62 m (202 ft). The Irvington Portal is adjacent to private homes, and most work will be limited to daytime hours in order to minimize impacts. The maximum length of tunnel that can be driven from this portal will be limited to 168 m (550 ft) to reduce the amount of truck traffic in the area.

GEOLOGIC CONDITIONS

The ground surface above the tunnel generally consists of rolling hills covered with grass, brush, and trees. A number of privately owned parcels will be crossed by the tunnel alignment. These parcels have domestic water supply wells, livestock and irrigation wells, and natural springs that indicate the local availability of groundwater in the vicinity of the tunnel alignment.

Ground cover above the tunnel varies along the alignment, ranging from about 6.1 to 228.6 m (20 to 750 ft). Ground elevations along the tunnel alignment vary from about El. 72 m (235 ft) at the Irvington Portal to about El. 320 m (1,050 ft) at the high point in the central portion of the alignment.

The rocks along the NIT alignment consist of marine sedimentary sandstone, shale, siltstone, and chert ranging in age from Miocene to Cretaceous (5 to 144 million years old). Younger Quaternary-period deposits include alluvium and colluvium, which are present at the Alameda West Portal, Sheridan Valley, in the vicinity of I-680, and at the Irvington Portal. Geologic units crossed by the tunnel alignment include (in order of increasing age) unconsolidated Quaternary-period deposits and artificial fill (Qoa, Qc, af), Briones Formation (Tbr), Tice Shale Formation (Tt), Oursan Sandstone Formation (To), Claremont Formation (Claremont Formation Chert and Shale Member [Tcc] and Sandstone Member [Tcs]), and the Cretaceous Sandstone and Shale (Ks). The geologic unit names are consistent with USGS geologic maps, but in some cases, the names do not match the bedrock lithology present within the tunnel. For example, the Ks unit includes extensive siltstone and sandstone but relatively little shale.

The NIT lies in the western ridges of the Diablo Range. This range has been subjected to significant faulting and folding, which has fractured and sheared the rocks along the tunnel alignment. Regional tectonic compression has uplifted the range and created folds that form at least one anticline and one large syncline (Niles Syncline) in the site area.

Fault-bounded blocks are formed by four mapped faults that cross the tunnel alignment: the Pirate Creek, Sheridan Creek, Unnamed, and Mill Creek faults. At least three additional faults and shear zones without surface expressions were observed during the EIT construction. Strata within the fault-bounded blocks tend to dip to the southwest between the AWP and Sheridan Creek fault, and are folded into a syncline between the Sheridan Creek fault and the Unnamed fault east of I-680. Strata between the Unnamed fault and the Mill Creek fault are folded to form a northwest-trending anticline. West of the Mill Creek fault, the strata dip to the northeast. Bedding dip inclinations range from moderate to vertical within these folds and are commonly steepest in the vicinity of faults. The largest fold feature is the Niles Syncline, with a near vertical axial plane mapped at approximately Sta. 140+50. The Claremont, Tice, and Briones formations will be encountered two to three times along the alignment because of the folding involved with the regional structure.

An extensive program of geotechnical investigation was completed during design. The investigation objectives were to characterize the soil and rock conditions, lithologic and fault contacts, and groundwater conditions along the tunnel alignment. The investigations included geologic mapping, exploratory borings, downhole testing and logging, surface geophysics, and laboratory testing.

Because of the complexity of the conditions encountered, the subsurface investigations program ultimately grew to include 38 exploratory borings, ranging from 15.2 to over 228.6 m deep (50 to over 750 ft). The total length drilled was over 2,652 m (8,700 ft), nearly double the originally anticipated footage. Twenty-five of the borings were drilled vertically, and 13 were inclined. Standpipe piezometers and/or vibrating wire piezometers were installed in 28 borings. The drilling was primarily performed using LF-70 and CS-1000 skid-mounted rotary rigs equipped with HQ-3 wireline core barrels. Sonic drilling equipment was also used in one area in an effort to improve sample recovery.

Water pressure (packer) testing was performed in a total of 17 core borings. Downhole geophysical surveys, including caliper logging and televiewer logging, were performed in 15 borings. Seismic velocity surveys (OYO suspension) were performed in 8 borings. The televiewer logging results were used to characterize the in situ frequency and orientation of rock discontinuities, fractures, bedding, and shear zones.

Surface seismic refraction surveys were completed at each portal area to assist in characterizing the overburden depths and bedrock properties. Surface wave geophysical surveys were performed to investigate the deeper overburden in the Vargas Road area. Aquifer pumping tests were carried out at the Vargas Road and the Sheridan Valley sites to investigate the hydrogeologic formation properties.

Laboratory testing was performed on selected rock core samples. The tests included uniaxial compressive strength, point load strength, indirect (Brazilian) tensile strength, punch-penetration, slake durability, Cerchar abrasivity index, thin-section petrographic analysis, specific gravity, modulus and uniaxial compression, triaxial compression, and multistage direct shear tests on joint samples.

GROUND CHARACTERIZATION

An extensive program of geotechnical investigations was completed as part of the design process (URS 2009). Based on the results, the tunnel alignment has been divided into eight reaches. The reach boundaries were established at the estimated contacts between the geologic formations (Figure 2). The locations of the geologic contacts were estimated based on the available geologic data, interpolation between borings, evaluation of surface geologic mapping data, and correlation with the EIT construction records. The reaches are shown in Table 1.

Table 1: Summary of the tunnel reaches

Reach (Stationing)	Length, m (ft)	Geologic Formations	Anticipated Fault Zones
1 (Sta. 41+40 to Sta. 75+90)	1,052 m (3,450 ft)	Cretaceous Sandstone and Shale (Ks)	None
2 (Sta.75+90 to Sta. 86+50)	323 m (1,060 ft)	Claremont Chert and Shale (Tcc); Oursan Sandstone (To)	Pirate Creek Fault; Sheridan Creek Fault
3 (Sta. 86+50 to Sta. 104+00)	533 m (1,750 ft)	Tice Shale (Tt)	Fault A
4 (Sta. 104+00 to Sta. 158+50)	1,661 m (5,450 ft)	Briones Formation (Tbr)	Fault B
5 (Sta. 158+50 to Sta. 173+00)	442 m (1,450 ft)	Tice Shale (Tt); Claremont Chert and Shale (Tcc)	Unnamed Fault
6 (Sta. 173+00 to Sta. 190+50)	533 m (1,750 ft)	Claremont Sandstone (Tcs)	None
7 (Sta.190+50 to Sta. 198+70)	250 m (820 ft)	Claremont Chert and Shale (Tcc)	Fault C; Mill Creek Fault
8 (Sta. 198+70 to Sta. 228+00)	893 m (2,930 ft)	Briones Formation (Tbr)	None

Ground conditions that will be encountered along the tunnel alignment have been divided into four ground classes to aid in the selection of tunnel excavation and support methods. Ground classes were defined based on the physical characteristics of the ground and its anticipated behavior during the tunnel excavation. The ground assigned to a particular class is expected to perform similarly in the tunnel excavation, and to require similar support methods.

Each ground class will be encountered multiple times throughout the tunnel in all tunnel reaches. The ground class definitions, predominant ground behaviors, and key characteristics associated with each ground class are described in Table 2. Potentially unstable ground conditions will be encountered throughout the tunnel, including but not limited to, raveling/caving, squeezing, swelling, running, and flowing conditions. The sheared nature of the rock mass, weak rocks, abundant clay infilling materials, intensely fractured rock mass, and high groundwater levels all will contribute to the instability of the tunnel excavation if not properly controlled. Control of groundwater inflows into the tunnel by pre-drainage and/or pre-excavation grouting will be necessary to minimize adverse effects on unstable ground and excavation progress.

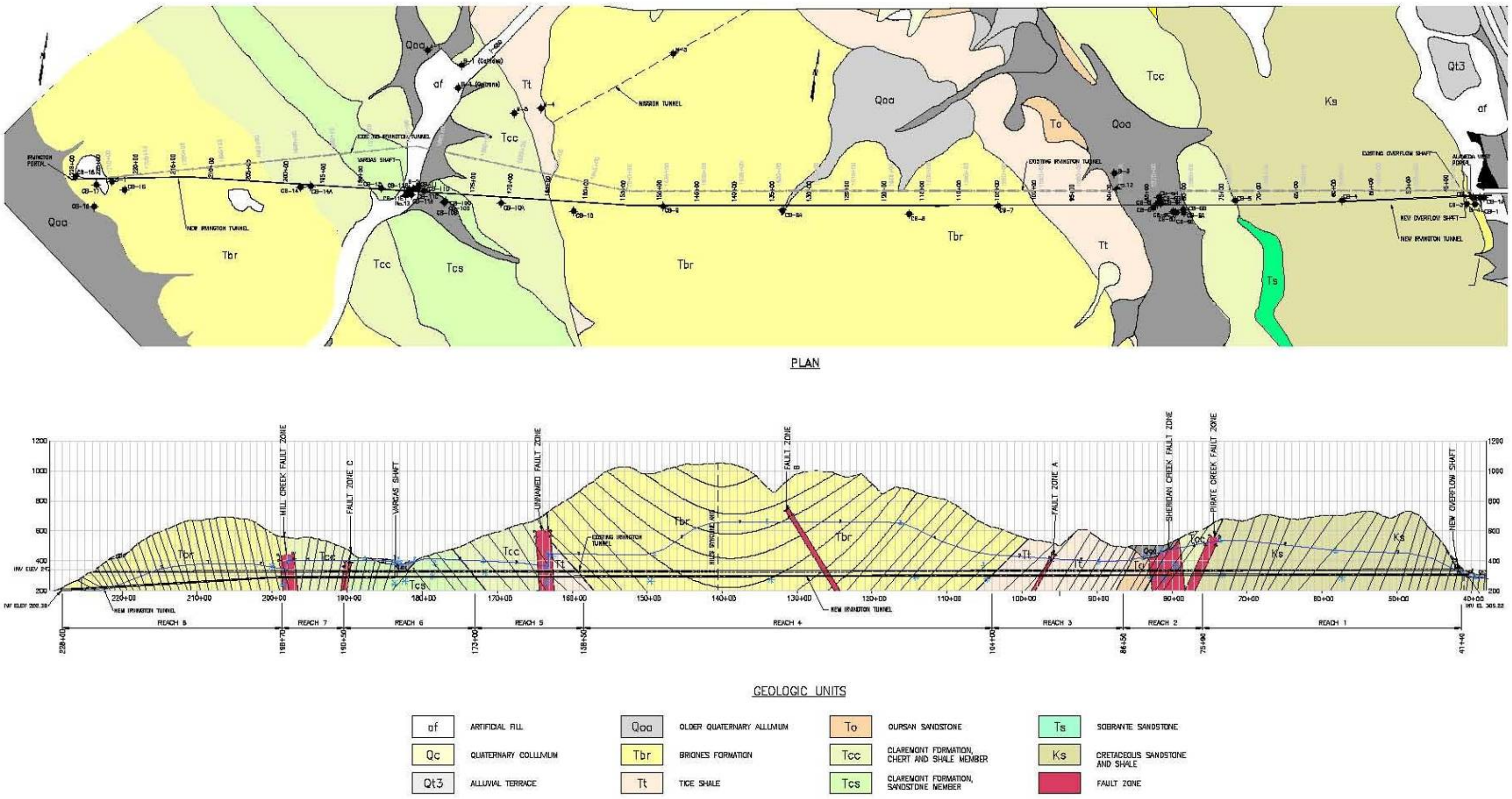

Figure 2. Generalized geologic profile of the New Irvington Tunnel project

Table 2: Definitions of ground classes

Ground Class and Definitions	Typical Rock Characteristics	Typical Discontinuity Characteristics	Ground Behavior
I: Massive to Moderately Fractured Rock	Sandstone, siltstone, and interbedded siltstone/sandstone; weak to strong rock; slightly weathered to fresh	Very rough to rough; fresh to slightly weathered surfaces	Structurally controlled block instability; spalling
II: Highly Fractured Rock	Sandstone, siltstone, interbedded siltstone/sandstone, and shale; weak to moderately strong rock; highly to slightly weathered	Rough, smooth, or slickensided surfaces or bedding planes; moderately to highly weathered/altered surfaces with infillings of clay and/or sand	Slow raveling; fast raveling where flowing groundwater is encountered
III Intensely Fractured Rock	Sandstone, siltstone, interbedded siltstone/sandstone, and shale; thinly bedded to laminated rock structure; very weak to moderately strong rock, may be friable, poorly cemented; highly to slightly weathered/altered	Smooth, slickensided surfaces; highly weathered/altered with occasional moderately wide clay/sand-filled joints, shears, and shear zones	Fast raveling/caving; potentially flowing ground
IV: Heavily Sheared/Faulted Rock with Clay Gouge/Infilling Materials	Heavily sheared rock including fault gouge, shattered rock, all with abundant clay; extremely weak to very weak rock; moderately to completely weathered/altered	Slickensided surfaces; highly weathered/altered with wide clay-filled joints, shears, and fault/shear zones	Squeezing; swelling; caving; fast raveling

The anticipated ground conditions along the tunnel were estimated based on evaluation of the geologic and geotechnical data collected for this project, along with review and correlation of the available EIT construction records. Rock mass quality evaluations were performed utilizing the Rock Quality Designation (RQD) (Deere and Deere 1988) and the Rock Mass Rating (RMR) system (Bieniawski 1988).

During the review process, we discovered that the EIT construction records contain some inconsistencies in their descriptions of the ground conditions and lithology encountered in the tunnel. In addition, the records provide no definitions of the descriptive or geologic terms used, and it appears that some of the terms used differ from current practice. Therefore, the EIT records were interpreted based on our understanding of site geology and the construction methods employed at the time of construction for each reach of the NIT. As an example, “running ground” identified in the EIT in areas of high groundwater inflow was concluded to be “flowing ground” in current tunneling terminology.

Significant shearing and many shear zones were observed in the geotechnical investigation borings completed for the NIT. Similar conditions are reported in the EIT construction records. The estimated amount of sheared/faulted rock in the NIT based on the EIT records ranges up to 90 percent over some reaches (in terms of the tunnel length impacted by sheared/faulted rock).

TUNNEL EXCAVATION AND SUPPORT

The NIT will be constructed in variable ground conditions, ranging from strong, massive rock to very weak and intensely fractured and sheared rock including fault gouge. The tunnel will encounter high groundwater inflows and difficult ground conditions including raveling, running, flowing, caving, and squeezing ground. Conventional tunneling methods, including the use of roadheaders, drill-and-blast techniques, and hydraulic excavators, are expected to be adaptable to the anticipated wide range of ground conditions, including running, caving, flowing, raveling, and squeezing ground in a number of areas along the alignment.

The potential for use of a tunnel boring machine (TBM) was evaluated and concluded to be unacceptable for several reasons. Given the anticipated highly variable and difficult ground conditions, high groundwater inflows, and high groundwater pressures present along portions of the alignment, the use of a TBM, especially a machine of the size required for this project, was concluded to present excessive risks in terms of the cost and schedule impacts to the project. Use of a TBM would also restrict access to the tunnel face and would hamper the use of ground improvement techniques needed on this project. In addition, the seismic reliability goal of the project requires a steel pipe lining across zones of potential sympathetic fault offset. The use of a TBM (with a continuous full-perimeter lining) would restrict the face inspection and mapping needed to detect and delineate the lengths and locations of these zones.

The tunnel excavation methods, initial support systems, and groundwater control measures for the tunnel will be determined by the contractor. The EIT was mainly excavated using drill-and-blast methods, although "hand spades" were used in two sections of the tunnel to excavate the Claremont Formation in the vicinity of Reach 5. Several excavation methods could be applied to the NIT, including roadheaders, drill-and-blast methods, and mechanical excavation. The selected approach will depend mainly on economics, equipment availability (for a tunnel of this size), and the skills and experience of the contractor's crew. Key geotechnical issues include rock types, strength and degree of fracturing of the rock mass, and groundwater conditions.

Roadheaders are expected to be capable of excavating most of the tunnel reaches where the rock mass is moderately weak or weaker and highly fractured. The performance of a roadheader will depend on the machine size and weight, and the rock strength (UCS), fracture spacing, and abrasivity.

Drill-and-blast excavation methods were used extensively for the EIT and are applicable to most of the NIT, the exception being zones of intensely fractured, sheared rock with significant clay content and the fault zones. The maximum round lengths for blasting will depend on face stability, rock mass quality, initial support requirements, and other factors. Wherever blasting is used, appropriate blast designs and vibration and noise monitoring will be required to control and minimize potential impacts of blasting on the existing tunnel, existing pipeline facilities, and residences adjacent to the portals and above the tunnel alignment.

The NIT will have a two-pass support system consisting of an initial support system and a final lining. The initial support requirements will vary along the tunnel due to the range of ground conditions that will be encountered during construction.

Presupport using spiling and/or forepoling will be required to control raveling, caving, and crown instability, primarily in tunnel reaches with Ground Class III and IV conditions. Presupport may also be required to prevent structurally controlled block instability, expected primarily in Ground Classes I and II. Face support in conjunction with pre-support is expected to be required to control block instability, overbreak, raveling, running/flowing, slaking and caving behaviors at the tunnel face in Ground Classes II, III, and IV.

GROUNDWATER CONTROL

The anticipated groundwater levels above the tunnel crown range from zero to 112.8 m (370 ft). Groundwater inflows are anticipated throughout a significant portion of the tunnel. The heaviest flows will occur where the ground is highly fractured and where fault and shear zones are encountered. Estimates of the maximum potential groundwater flush flows and sustained flows were made for each of the NIT reaches. The estimates were based on the results of the groundwater modeling for present-day conditions and on interpretation of the EIT construction records (Figure 3).

Implementation of effective groundwater control measures will be required to limit uncontrolled inflows into the tunnel and to reduce the impact of inflows on tunnel construction. Additional inflow control measures will be required where needed to protect groundwater wells and resources, as directed by SFPUC.

Predrainage of the rock mass ahead of the face is expected to be feasible from within the tunnel in many areas. At selected locations, predrainage from the ground surface is also feasible. The contractor must implement pre-excavation grouting, predrainage, and/or a combination of both measures, as necessary, to reduce sustained groundwater inflows to within workable limits. Predrainage from within the tunnel is considered feasible when the probe holes indicate the potential for significant groundwater inflows into the tunnel. Predrainage has the following objectives:

- Reduce high groundwater inflow potential,
- Improve the efficiency of the pre-excavation grouting, and, if necessary,
- Improve ground behavior.

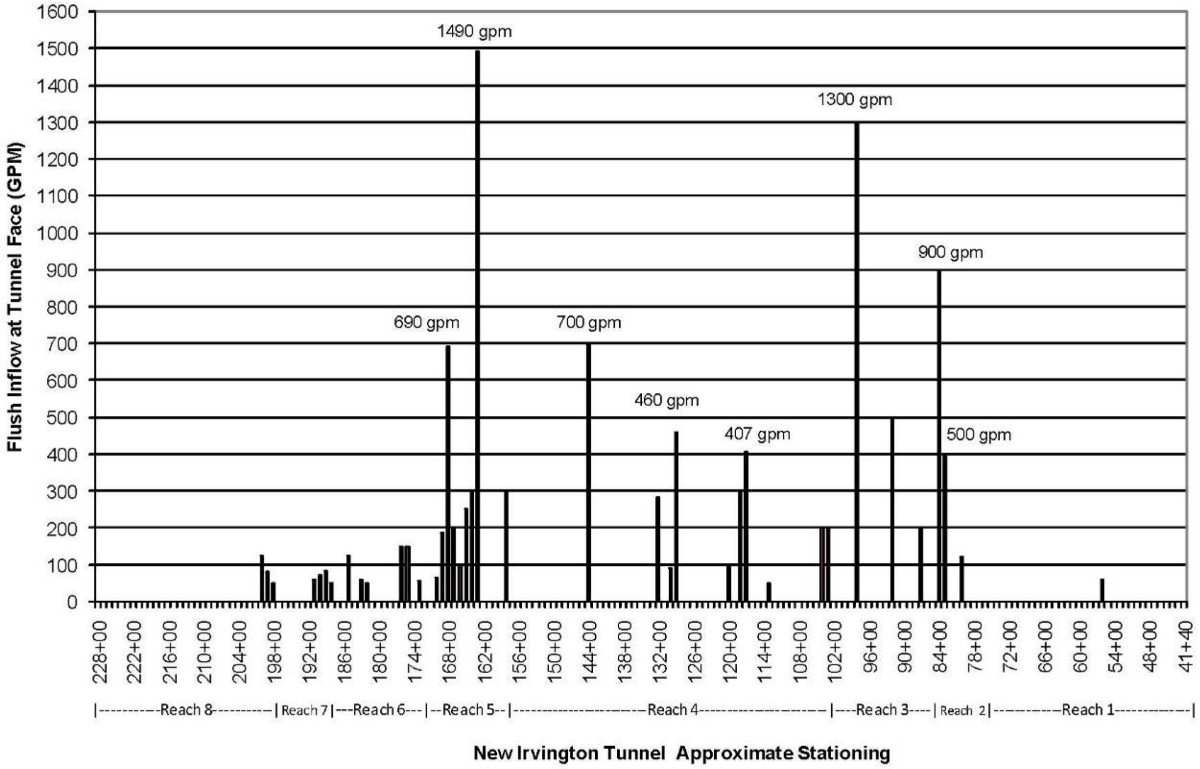

Figure 3. EIT flush flows interpreted from portal flow records and shown based on the NIT stationing and reaches

Drain holes drilled ahead of the tunnel face can reduce the groundwater head and aid in the control of heading inflows. Depending on the fracture openings, fracture spacings, and storativity of the rock mass, the effectiveness of drainage will vary. Typically, drainage would be done ahead of the advancing face to improve the ground behavior at the tunnel walls, roof, and face.

Predrainage from the ground surface using dewatering wells is planned to supplement dewatering from within the tunnel in two areas along the alignment: Sheridan Valley and the Vargas Road/I-680 corridor. At each site, the contractor will be required to design and install a dewatering system to lower the groundwater table in advance of tunnel excavation, in areas where problematic ground conditions and/or high water inflows are expected.

Pre-excavation grouting requires the injection of grout to fill open fractures in the rock mass. Typical pre-excavation grouting will not penetrate intact rock or joint infillings with low porosity. In zones of completely weathered/altered rock, clay-filled shears and clay fault gouge, or intensely fractured rock, grout penetration is expected to be limited because of low hydraulic conductivity, so the effectiveness of pre-excavation grouting for groundwater control and ground improvement will also be limited. In areas where the rock exhibits high hydraulic conductivity, treatment of the fractured rock mass through grouting is expected to be more effective. The performance objectives for pre-excavation grouting and drainage are as follows:

- Limit the groundwater inflows at the tunnel face to a rate compatible with the selected tunnel construction means and methods.
- Mitigate adverse ground behavior caused by heavy groundwater inflows as necessary to allow adequate installation of initial support measures.

The contractor’s drilling, casing, and grouting equipment and methods must be capable of staged grouting in unstable rock formations. The probe and grout holes are expected to encounter hole stability problems in weak rock that includes highly to intensely fractured rock and clayey shear and fault zones. Due to uncertainties associated with the characteristics of groundwater flows in a fractured rock mass, the required probe and verification holes are not expected to detect all potential inflows. The actual inflows encountered at the tunnel face may vary substantially from estimates based on probe hole flows. The contractor must be prepared to adjust pre-excavation grouting and drainage techniques, criteria, and procedures during construction to accommodate the expected rapidly varying ground conditions.

Inflows into the tunnel may negatively impact existing water wells and nearby springs. If monitoring data indicate unacceptable impacts are occurring, the owner may direct the contractor to implement additional control measures. Feasible additional measures include additional pre-excavation grouting, the installation of a built-up shotcrete lining for controlling water inflows in conjunction with controlling ground stability, and other effective measures proposed by the contractor.

SEISMIC DESIGN

The NIT is located in a seismically active region of California, dominated by the San Andreas fault system. The nearby Hayward fault on the west and the Calaveras fault on the east are capable of generating large (Magnitude ≥7.0) earthquakes. In addition to these two faults, numerous other, active faults are located within 48.3 km (30 mi) of the tunnel site. However, the tunnel alignment does not cross any faults that have been zoned as Alquist-Priolo Earthquake Fault Zones by the state of California. The Alquist-Priolo Earthquake Fault Zones are defined based on evidence of Holocene surface rupture (i.e., within the last 11,000 years). Surface mapping completed as part of the geotechnical investigations (URS 2009) found no evidence for Holocene surface rupture on any of the mapped faults crossing the tunnel alignment.

According to the SFPUC’s General Seismic Design Requirements (SFPUC 2006), the NIT is specified as a Seismic Performance Class III facility. The design earthquake for the Seismic Performance Class III facilities has a 5 percent probability of exceedance in 50 years (975-year approximate return period). The SFPUC’s objective for seismic performance of the tunnel is to deliver winter day demand of water within 24 hours of a major earthquake (SFPUC, 2006). In order for this objective to be achieved, catastrophic damage to the facilities during the design earthquake must be avoided. The seismic design parameters for the NIT were developed based on a seismic hazard and ground response analysis (URS 2007).

Depending on distance from the active faults, the estimated peak ground accelerations (PGA) along the tunnel alignment for the design earthquake vary from 0.85 g to 1.02 g at the ground surface level, and from 0.45 g to 0.58 g at the tunnel level. The peak ground velocities (PGV) range from 148 to 160 cm/s (58.3 to 62.9 in./sec) at the ground surface level, and from 47 to 58 cm/sec (18.5 to 22.8 in./sec) at the tunnel level.

Although the NIT alignment does not cross any seismically active faults, up to 150 mm (6 in.) of sympathetic displacement over a width of 1.5 m (5 ft) is considered possible on any or all of the four mapped secondary faults (Pirate Creek, Sheridan Creek, Unnamed, and Mill Creek faults). Sympathetic displacement could occur in response to a significant earthquake event on either the Hayward or Calaveras fault (WLA 2007).

In order to minimize the potential impact of sympathetic displacements on the NIT final lining, the design included steel pipe final lining in areas where the NIT crosses the four mapped fault zones. The steel pipe is more ductile than concrete and can tolerate much higher deformations or strains without rupture or collapse. However, because the design displacement occurs over a very short length, high shear strains in the steel pipe are expected. To investigate the effects of potential fault offset on the steel pipe final lining and finalize the pipe design, numerical analyses were performed using the three-dimensional finite-difference program FLAC3D (Fast Lagrangian Analysis of Continua in 3 Dimensions) Version 3.0 (Itasca, 2005). The FLAC3D analyses were used to calculate deformations and stresses induced in the steel pipe final lining by the design. A typical model used in the FLAC3D analyses is illustrated in Figure 4. In the analyses, two blocks of rock, one on either side of a 2.7-m (9-ft) wide fault zone were offset in the opposite directions along the fault plane. The resulting total relative displacement between these two blocks of rock masses was equal to the design displacement of 150 mm (6 in.) considered.

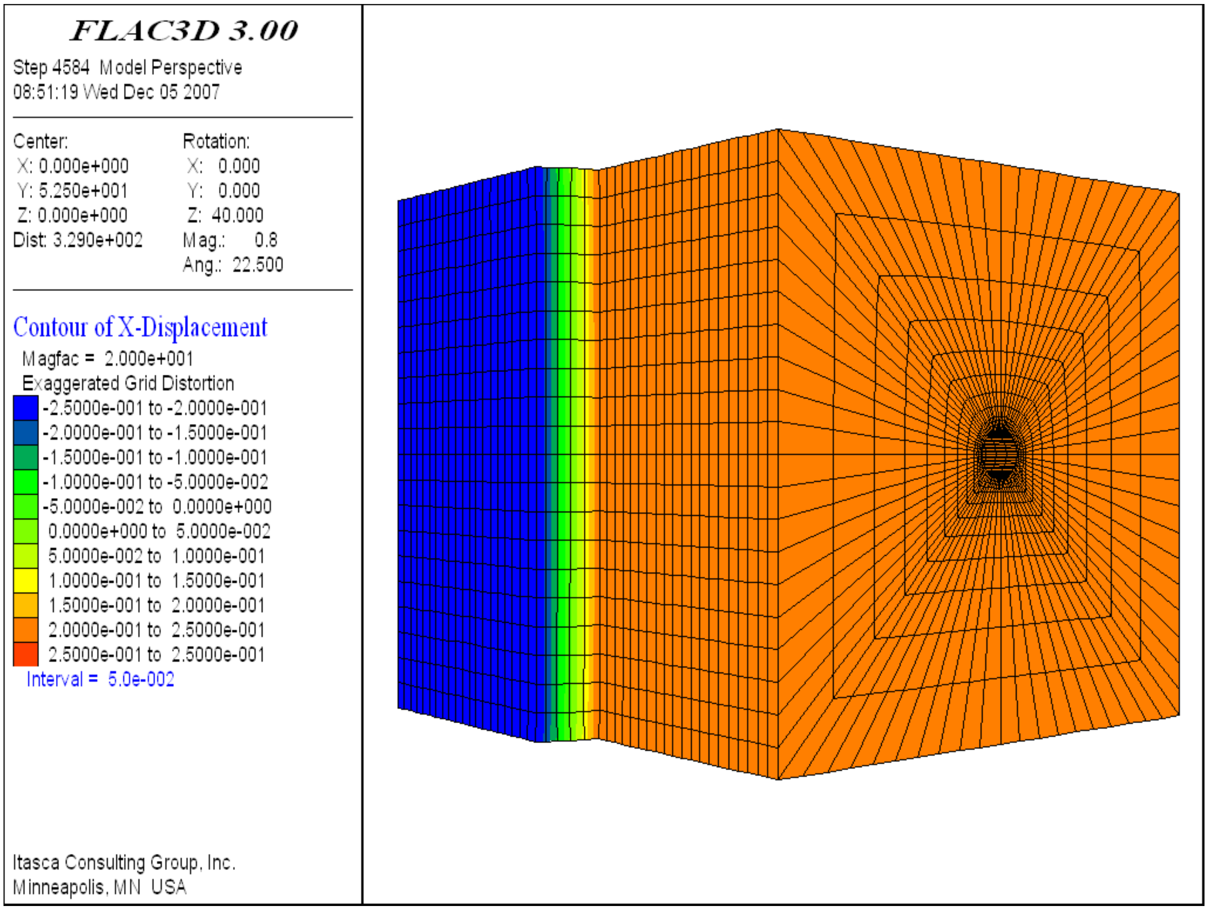

Figure 4. Typical FLAC3D model for fault offset analysis

Key parameters were varied in the analysis to optimize the design and assess the impact of potential uncertainties. The key parameters include the stiffness of backfill concrete, wall thickness of steel pipe, magnitude of fault offset, and deformation moduli of rock mass and fault zone. Figure 5 shows a typical deformed shape (magnified by 20 times) of the tunnel steel pipe final lining following a 150-mm (6-in.) fault offset. Results of the analyses indicated that the maximum stress in the steel pipe increases with stiffness of the backfill concrete. In order to control the maximum stresses in the steel pipe to within tolerable limits, use of a special low-density backfill concrete to fill the annular space between the initial support and the steel pipe will be required. This special backfill will consist of cellular concrete with an unconfined compressive strength between 1.37 and 2.07 MPa (200 and 300 psi).

PORTAL EXCAVATION AND SUPPORT

Development of the portals for tunnel construction and installation of pipeline connections will be a critical element of the project. The preparatory work will include access development, installation of erosion and sedimentation control measures, and protection of existing SFPUC pipelines and portal structures.

The two portal excavations will require both soil and rock excavation techniques. Rock near the portals is expected to be highly to intensely fractured, with an average fracture spacing of less than one foot. The use of appropriately sized excavating and earth-moving equipment will be suitable for most of the portal excavations. Drill-and-blast methods or impact hammer methods will be required to break up harder, more massive rock in localized areas. However, such excavation methods and related construction activities may disturb nearby residents and affect the stability of adjacent pipelines, slopes and shoring systems. The project includes very tight noise and vibration criteria and required measures to reduce and mitigate potential impacts.

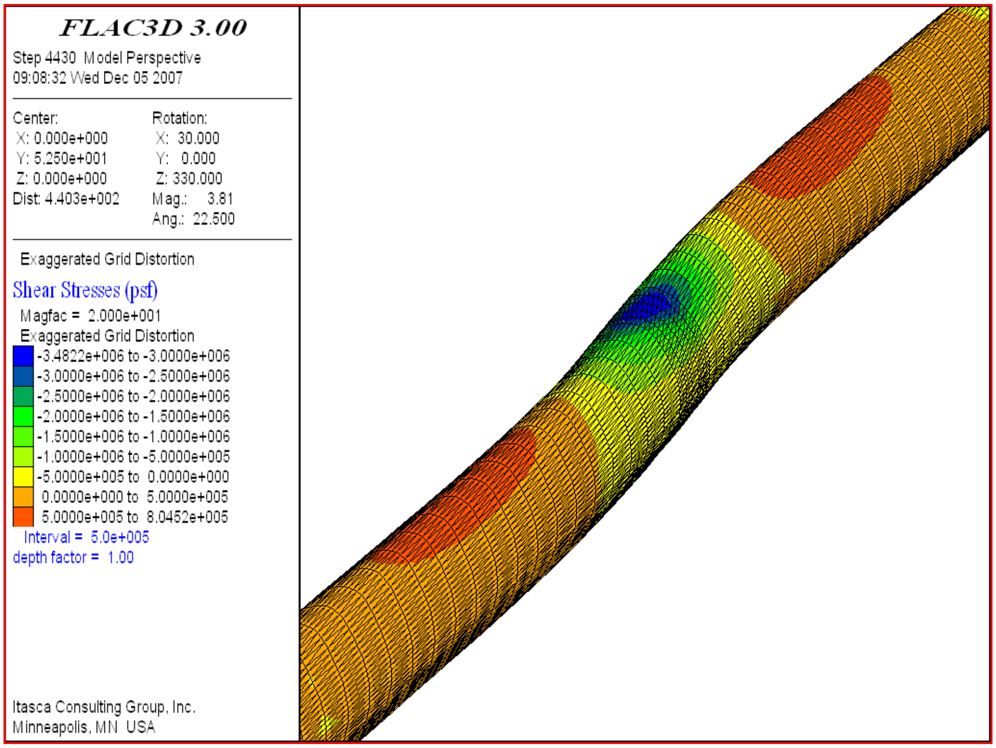

Figure 5. Contours of shear stresses and deformed shape of the steel pipe lining caused by fault sympathetic displacement

To achieve slope stability, all portal excavation cut slopes will require temporary support measures, such as fully grouted soil nails, shotcrete, rock dowels, rock bolts, and/or other measures as appropriate for the subsurface conditions encountered. In addition, to achieve a stable tunnel excavation at the portal, reinforcement of the rock mass above the tunnel crown will be required, by installation of portal spiles or forepoling before starting the tunnel. Typical minimum required ground support measures for the portal excavation sidewalls are shown in Figure 6.

PROJECT SCHEDULE

The design of the project was completed in December 2009 and advertised for bidding in January 2010. Bid opening was March 2010. Notice to proceed is anticipated in June 2010. The project has a 42-month construction schedule and completion is scheduled for November 2013.

CONCLUSION

The first tunnel was built with very little advanced information. The miners in 1928 drove the tunnel blindly, with no advanced exploration, no probe hole drilling, limited ability to grout, and used timber sets. Information collected during construction of the existing tunnel has been valuable in the design of the new Irvington Tunnel project to help limit some of the challenges to be faced. That information combined with today's tunneling equipment, technology, materials, increased exploration, advanced planning, and the ability to probe, drain, grout and dewater from the surface, should allow the New Irvington Tunnel to be completed with much fewer construction problems and much greater ability to survive the next major earthquake in the area.

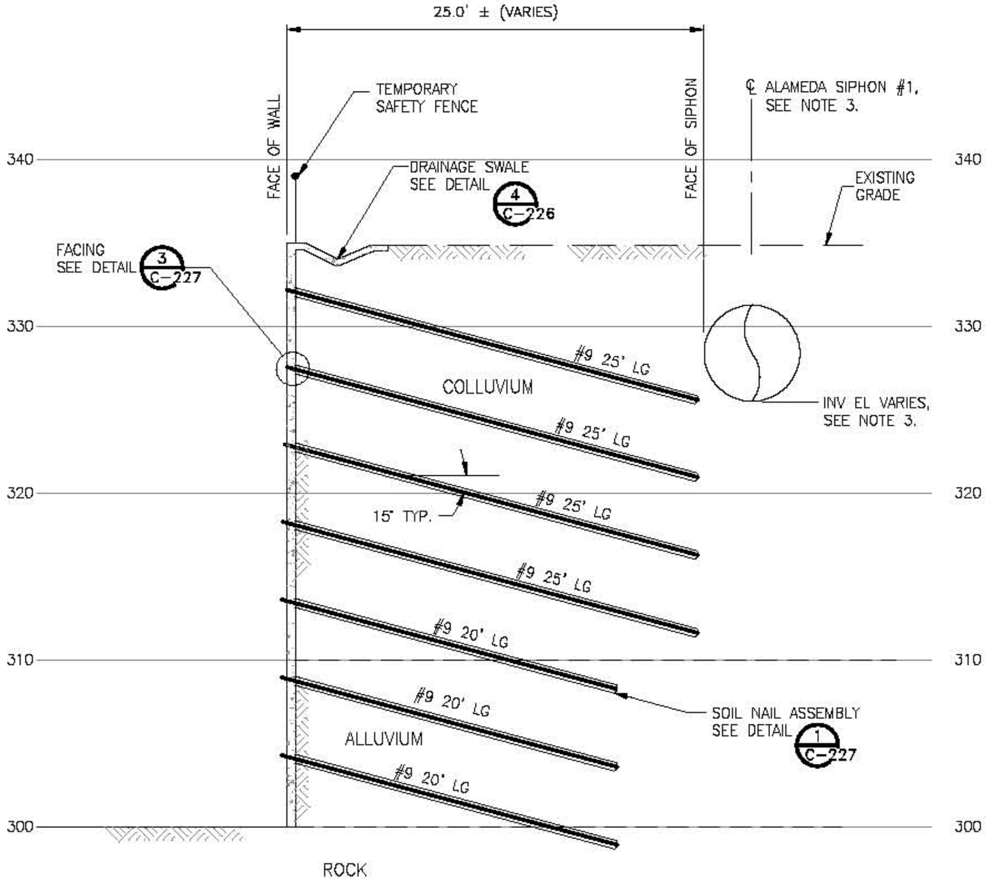

Figure 6. Typical portal excavation support

REFERENCES

Bieniawski, Z. T. 1988. The Rock Mass Rating (RMR) system (geomechanics classification) in engineering practice. In *Rock Mass Classification Systems for Engineering Purposes*. Edited by L. Kirkaldie, 17–101. Philadelphia, Penn.: American Society for Testing of Materials, ASTM STP 984.

Deere, D.U., and Deere, D.W. 1988. The Rock Quality Designation (RQD) index in practice. In: *Rock classification Systems for Engineering Purposes*, L. Kirkaldie, ed., 91–101. Philadelphia, Penn.: American Society for Testing of Materials, ASTM STP 984.

Itasca. 2005. *Fast Lagrangian Analysis of Continua in 3 Dimensions (FLAC3D),* Version 3.0, Itasca Consulting Group, Minneapolis, Minnesota.

SFPUC (San Francisco Public Utilities Commission). 2006. *General Seismic Requirements for Design of New Facilities and Upgrade of Existing Facilities*.

URS (URS Corporation). 2007. *Seismic Hazard Analysis, New Irvington Tunnel Project.* Draft Technical Memorandum 6-06, prepared for the San Francisco Public Utilities Commission, June.

URS. 2009. *Geotechnical Data Report, New Irvington Tunnel Project.* Final Report prepared for the San Francisco Public Utilities Commission, November.

URS and Jacobs Associates. 2009. *Geotechnical Baseline Report, New Irvington Tunnel Project.* Final Report prepared for the San Francisco Public Utilities Commission, November.

WLA (William Lettis & Associates, Inc.). 2007. *Geologic Investigation of Possible Deformation Expected on Secondary Faults Along the New Irvington Tunnel Corridor, Alameda County, California.* Technical Memorandum prepared for URS Corporation, October 26.