GEOTECHNICAL BASELINE REPORT
PSA 923

FOR

COWMIRE AND COLDWATER STORMWATER TUNNELS

Prepared for

LAMBERT-ST. LOUIS INTERNATIONAL AIRPORT
EXPANSION PROGRAM (AEP)
CITY OF ST. LOUIS – BOARD OF PUBIC SERVICE
c/o THE SPK TEAM

May 17, 2002

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LAMBERT-ST.LOUIS INTERNATIONAL AIRPORT EXPANSION PROGRAM (AEP)

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The St. Louis Lambert International Airport Expansion Program (AEP) includes the construction of two gravity stormwater sewer tunnels (one 102-inch inside diameter pipe and one 72-inch inside diameter pipe) that extend from the relocated Lindbergh Boulevard to the proposed Cowmire and Coldwater South Detention Basins. The tunnel alignments relative to current and future structures of the AEP are shown in Figure 1-1. This report only addresses design and construction considerations for the Cowmire and Coldwater Stormwater Tunnel projects.

The project design team for the AEP consists of Sverdrup, Parsons, and Kwame (SPK); a joint venture of Parsons Brinkerhoff and TSI Engineering (PBTS), a joint venture; and URS Corporation (URS). SPK is the program manager for the City of St. Louis. PBTS performed civil engineering, pipeline design, design of hydraulic structures, and produced the plans and specifications. URS is the project geotechnical engineer and prepared the Geotechnical Data Report and Geotechnical Baseline Report.

This Geotechnical Baseline Report (GBR) summarizes the geotechnical basis for design and construction of the Cowmire and Coldwater Stormwater Tunnels and is intended for use by bidders as an aid in bid preparation. This report includes:

- descriptions of the tunnels and other construction to be accomplished;
- interpretations of the geologic and geotechnical data collected for the project;
- a summary of anticipated subsurface conditions;
- a summary of how these anticipated conditions have been addressed in design; and
- a discussion of important construction considerations that the Contractor will need to address during bid preparation and construction.

The results of the geotechnical investigations carried out for the tunnels are presented in URS' report titled *Geotechnical Data Report – Cowmire and Coldwater Stormwater Tunnels* dated April 12, 2002, and hereafter referred to as the GDR. The Geotechnical Baseline and Geotechnical Data Reports are included as part of the Contract Documents.

This report presents the geotechnical engineer's best judgment of the subsurface and ground conditions anticipated to be encountered during construction. In order to develop the design, it was necessary to interpolate between exploratory boring data to estimate the conditions along the tunnel alignments. While the actual conditions encountered in the field are expected to be within the range of conditions discussed herein, the distribution of ground conditions encountered in the tunnels may be more complex than presented in this report. In addition, the project design is based on assumptions regarding construction methods likely to be used and on the level of workmanship that can reasonably be expected during the construction of a tunnel. The Contractor’s selected equipment, means, methods, and workmanship will influence the behavior of the geologic deposits encountered in the tunnels and portal/shaft excavations.

The Project Documents include a Differing Site Conditions clause. Descriptions and discussions of anticipated subsurface conditions contained in this report and the GDR are intended for use by the Airport and the Contractor, if necessary, in evaluating the merits of differing site conditions claims.

Some of the technical concepts, terms, and descriptions in this report may not be fully understood by bidders. The Contract documents require that bidders confer with a geotechnical engineer or engineering geologist who is familiar with all topics of this report and the GDR. This individual should have experience under conditions similar to those described herein, and should carefully review and explain this information so that a complete understanding of the information presented can be developed prior to submitting a bid.

The project consists of the construction of two stormwater tunnels and associated drop structures and access shafts that will convey surface water run-off from the proposed runways, taxiways, and relocated Lindbergh Boulevard to the proposed Cowmire and Coldwater South Detention Basins. The general layout of the tunnels and hydraulic structures is shown in Contract Drawing G3-01. The elements of each of the tunnel projects are described in more detail in the following paragraphs. References herein to structure locations associated with this work are approximate. The Contractor should refer to the Contract Drawings for precise information on project layout.

2.1 COWMIRE STORMWATER TUNNEL

The Cowmire Stormwater Tunnel will consist of a trunk sewer, four access/drop shafts, and an outlet structure. Appurtenant construction includes an access road and relocation of an 8-inch diameter sanitary sewer and associated manhole. The plan and profile of the Cowmire Stormwater Tunnel are shown on Contract Drawings CG1-01 through CG1-03.

2.1.1 Trunk Sewer

The trunk sewer will consist of a 102-inch inside diameter (ID) pipe installed in the tunnel as shown in the Contract Drawings. The pipe material is the choice of the Contractor, within the limitations of the Contract Specifications. The trunk sewer will be installed by tunneling and will extend from about Sta. 0+80 at the outlet structure (see Contract Drawing CG1-01) to drop shaft SP 20 near Sta. 32+00 (see Contract Drawing CG1-03). Invert elevations of the 102-inch pipe increase from about El. 512 (feet, mean sea level datum, at Sta. 0+80) to El. 520 (near Sta. 32+00), corresponding to a grade of about 0.24 percent.

2.1.2 Shafts

Four shafts are planned along the Cowmire Stormwater Tunnel alignment for personnel access and connection to sewer laterals. The shafts are located near Sta. 2+00 (SP 12), 12+00 (SP 14), 22+00 (SP 16), and 32+00 (SP 20) as shown on Contract Drawings CG1-01 through CG1-03. Access shafts SP 12, SP 14, and SP 16 will consist of 4-foot ID concrete pressure pipes connected to the trunk sewer as shown on Contract Drawing CB1-06.

Structure SP 20 is a drop and access shaft that will be located near Sta. 32+00. The shaft is detailed in Contract Drawings CB1-02 and CB1-03. The shaft will be a rectangular, reinforced concrete structure, with dimensions of 28 feet by 21.5 feet. The top of the shaft will be at approximately El. 573 along an 80-foot wide bench that is part of proposed grading for relocated Lindbergh Boulevard. The interior base of the shaft will be at El. 519. An 84-inch ID RCP will enter the shaft from the airfield to the south at about invert El. 561. As a part of future construction, a 48-inch ID sewer lateral will enter the shaft from the east at invert El. 530, as shown on Contract Drawing CG1-03. At the time of this Work, the Contractor will install one pipe length at the specified location and construct a temporary bulkhead.

2.1.3 Outlet Structure

The outlet for the trunk sewer will be located on the east side of the proposed Cowmire Detention Basin. The outlet structure will consist of a concrete headwall, concrete cantilever walls, and a concrete-lined channel. Details regarding the size, shape, and length of the outlet structure are shown on Contract Drawing CB1-01.

2.1.4 Appurtenant Construction

The contractor is responsible for appurtenant construction including excavation for the outlet structure, drop shafts, and manholes; access road construction; and relocation of an 8-inch PVC sanitary sewer line that will run below the outlet structure, including a 25-foot deep manhole. Details of the appurtenant construction elements are provided in the Contract Drawings.

2.2 COLDWATER STORMWATER TUNNEL

The Coldwater Stormwater Tunnel will consist of a trunk sewer, two access/drop shafts, and an outlet structure.

2.2.1 Trunk Sewer

The trunk sewer will consist of a minimum 72-inch ID pipe installed in the tunnel as shown in Contract Drawing CG1-04. The trunk sewer will be installed by tunneling and will extend from the shaft near Sta. 0+00 to the outlet structure near Sta. 14+30. Invert elevations of the sewer pipe decrease from about El. 535 (near Sta. 0+00) to El. 531 (near Sta. 14+30) corresponding to a grade of 0.24 percent.

2.2.2 Shafts

Two shafts will be constructed for the Coldwater Stormwater Tunnel, one near Sta. 0+00 and the other near Sta. 10+00. Drop shaft SP-840 near Sta. 0+00 will be a 5-foot ID concrete riser pipe, with connections to three existing sewer laterals as shown on Contract Drawing CB1-05. The personnel access shaft SP-860 near Sta. 10+00 will be a 5-foot ID concrete pressure pipe connected to the trunk sewer as indicated on Contract Drawing CB1-06.

2.2.3 Outlet Structure

The outlet for the trunk sewer will be located on the west side of the proposed Coldwater South Detention Basin. The outlet structure will consist of a concrete headwall, concrete cantilever walls, and a concrete-lined channel. Details regarding the size, shape, and length of the outlet structure are shown on Contract Drawing CB1-04.

2.3 CONSTRUCTION PHASING

The Airport Expansion Project involves a large number of construction elements and will employ a number of contractors to complete the work. Several of these construction elements impact the Stormwater Tunnels Project. These impacts are discussed below.

Grading by others for relocated Lindbergh Boulevard will occur prior to construction of the stormwater tunnels. The limits of the proposed grading are shown on the Contract Drawings. As shown on the Contract Drawings, a 50-foot high slope will be cut at the east end of the Cowmire Stormwater Tunnel alignment prior to tunnel construction. All proposed rough grading for relocated Lindbergh Boulevard will be completed prior to the tunneling contractor beginning his work.

Prior to construction of the Coldwater Stormwater Tunnel, others will cut a 70-foot slope and a 90-foot slope at the western and eastern ends of the tunnel alignment, respectively, as shown on

Contract Drawing CG1-04. Following or during tunnel construction, approximately 40 feet of overburden cover will be removed by others as shown on Contract Drawing CG1-04. Based on the current construction schedule, the tunneling contractor must plan to coordinate work areas, spoil placement, haul roads, erosion control, etc., with the runway grading contractor and the Coldwater South Detention Basin contractor. Construction of a roadway interchange (Lindbergh Boulevard at Natural Bridge Road) is currently underway near the western reaches of the Coldwater Tunnel. The tunneling contractor will also need to coordinate activities with the interchange contractor. The construction limits shown on the plans are general, and the tunneling, runway grading, and detention basin contractors' construction limits overlap. Coordination will need to be performed between all contractors and the Engineer prior to beginning of this work.

This section of the report describes the general site and subsurface conditions along the project alignments. Specifically, this section discusses existing surface topography and proposed grading that will affect this Work; environmental conditions based on review of current and past land use performed by others; and subsurface conditions including soil, rock, and groundwater.

3.1 SITE TOPOGRAPHY

Surface topography generally consists of gently rolling hills – typical of a landform resulting from erosion of loess and residual clay. The site topography of the completed airport expansion program is included as Figure 3-1. The topography along each of the tunnel alignments is described in the following sections.

3.1.1 Cowmire Stormwater Tunnel

Topography along the Cowmire Stormwater Tunnel alignment is characterized by a northwest to southeast trending ridge adjacent to a natural drainage channel. Existing grades along the project alignment gradually increase from about El. 520 at the proposed outlet to El. 625 near Sta. 23+50. East of Sta. 23+50, grades gently decline to about El. 580 near Sta. 32+00. Prior to tunnel construction, cuts east of Sta. 26+00 will result in grades ranging from El. 612 to 574, as shown on Contract Drawings CG1-02 and CG1-03.

Cover above the sewer tunnel crown ranges from less than 5 feet at the outlet structure to a maximum of about 100 feet near Sta. 23+50, and averages approximately 60 feet.

3.1.2 Coldwater Stormwater Tunnel

The Coldwater Stormwater Tunnel runs beneath an existing wide ridge. Prior to tunnel construction, regrading for the relocated Lindbergh Boulevard will involve an approximately 70-foot deep cut from about Sta. 0+00 to Sta. 3+40. The permanent cut slopes will be at 3H:1V, as shown in the Contract Drawings.

As indicated on Contract Drawing CG1-04, from about Sta. 10+50 to 12+50, regrading for the Lindbergh bypass will lower the grade approximately 20 feet to El. 590. From Sta. 12+50 to beyond 14+70, an approximately 75-foot cut leading to the Coldwater South Detention Basin will precede tunnel construction. Following tunnel construction, final grading for the proposed runways and taxiways will involve lowering the general grade from about El. 590 at Sta. 2+60 to El. 565 at Sta. 13+40.

At the time of tunnel construction, cover above the sewer tunnel crown will range from a minimum of five feet near the outfall structure to a maximum of about 75 feet from Sta. 3+40 to 9+60. Average cover is approximately 60 feet.

3.2 ENVIRONMENTAL CONDITIONS

Current land use in the Cowmire Stormwater Tunnel vicinity includes a golf course and undeveloped woodland and pastures. Portions of the project site were previously residential, but homes have since been demolished. The Cowmire Stormwater Tunnel will pass below existing Fee Fee Road. At the time of tunnel construction, Fee Fee Road will be open to traffic. The proposed Coldwater Stormwater Tunnel will pass below two active roadways – the existing 4-lane Lindbergh Boulevard (from approximately Sta. 6+00 to 8+00) and the Lindbergh bypass (from approximately Sta. 10+50 to Sta. 12+50) where traffic will be diverted during construction. Besides these roadways, current land use in the Coldwater Stormwater Tunnel vicinity includes several motels/hotels, a former gas station, and empty fields. All the lots adjacent to the tunnel alignment have been developed previously, although several structures are abandoned or have been removed.

Geotechnology, Inc. conducted an environmental site assessment for the AEP. Their report, *Final Environmental Impact Statement* dated December 1997, indicates that no soil or groundwater contamination are likely near the project site for the Cowmire Stormwater Tunnel. During the field investigation for the Coldwater Stormwater Tunnel, an unidentified odor was detected in Boring 1554 in the top 40 feet. The source of this odor is currently being investigated and will be included in a separate report to the Airport by the Airport's environmental consultant. We understand that the Airport plans to remediate any identified contamination prior to tunnel construction. Remediation will consist of removal of impacted soils. Therefore, for planning and bidding purposes, Contractors should assume that no soil or groundwater contamination will be encountered.

3.3 SOURCES OF GEOLOGIC DATA

Geologic data is available from several sources, including the geotechnical investigations performed for the Cowmire Stormwater Tunnel in 2000 and 2001 and the Coldwater Stormwater Tunnel in 2001. Other sources of geologic information include published maps and reports. A compilation of available information and data including results of the exploratory boring and laboratory test data obtained for the tunnel projects is presented in the GDR. Refer to the GDR for a more complete listing of sources of geologic data.

3.4 GEOTECHNICAL INVESTIGATIONS

Borehole explorations and laboratory testing were performed by Terracon under contract with the City of St. Louis. Several drill rig mobilizations were made to drill borings for the proposed tunnels. In September 2000, three borings (1501, 1502, and 1503) were drilled to assess the feasibility of tunneling along the Cowmire Stormwater Tunnel alignment. In November 2000, eight additional borings (1504 through 1508 and 1510 through 1512) were drilled near the proposed Cowmire Stormwater Tunnel centerline. In October 2001, three more borings (B1513 through 1515) were drilled to define the soil/rock contact near the proposed outlet structure.

Near the Coldwater Stormwater Tunnel alignment, five borings (LB-4, 504, 940, 1205, and 1206) were drilled as part of other AEP projects. Six initial borings (1550 and 1553 through 1557) were drilled in August and September 2001 along the Coldwater Stormwater Tunnel alignment for preliminary engineering design. In October 2001, two additional borings (1551 and 1552) were drilled to better define the limits and depth of residual clay from approximately Sta. 1+00 to 2+50.

The field exploration work also included in-situ hydrogeologic packer tests. Groundwater levels were measured in each borehole, typically 48 hours after completion of drilling. After the groundwater levels were measured, each boring was then grouted closed. Grout was installed by pouring grout into the borehole at the ground surface until the grout level rose to the existing grade. A laboratory testing program was conducted on soil and rock samples recovered from the borings. The field and laboratory programs are described in the GDR. Lastly, as indicated in the GDR, in-situ pressuremeter tests were conducted in three borings located approximately 200 feet south of the Coldwater Stormwater Tunnel western portal.

3.5 REGIONAL GEOLOGY

The project site is located in the Dissected Till Plains physiographic province (Lutzen and Rockaway 1971), which consists of gently rolling hills. The stratigraphy generally consists of weathered loess, residual clay (derived from weathering and decomposition of shale and limestone), and Pennsylvanian-age bedrock consisting of interbedded limestones and shales. Geologically recent stream channel deposits (alluvium) occur in natural drainage channels in the broad valleys between adjacent uplands. The following paragraphs provide brief geologic descriptions of the soil and rock formations that will be encountered during this work. The GDR provides more detailed descriptions of the deposits.

3.5.1 Soils

Soils in the project vicinity consist of fill, loess, alluvium, and residual clay. The fill consists primarily of stiff silty clay, and contains trace quantities (less than 5%) of organics, sand, and gravel. Loess is a wind-blown deposit of firm to stiff clayey silts and silty clays. Locally, the loess deposits are termed “modified loess” as a result of weathering and higher clay contents than typically found in younger loess deposits. The alluvium (stream deposits) consists of firm to stiff, low to high plasticity clay and soft to medium stiff, low plasticity clayey silt.

Residual clay overlies bedrock in the project vicinity. This material primarily was formed by the in-place weathering and decomposition of the Labette Formation shales; however, some of the residual clay is likely derived from weathered and decomposition of limestone. The residual clay generally has medium to high plasticity and is stiff to very stiff.

3.5.2 Bedrock

The bedrock formations include highly variable interbedded shale and limestone, with thin coal and chert seams of Pennsylvanian-age. The formations consist of (in descending order): Fort Scott Formation (comprised of the Little Osage and Blackjack Creek Members); the Excello Formation;

and the Lagonda Formation. A simplified stratigraphic column is provided in Figure 3-2. The Little Osage Member consists of a limestone/shale/limestone/ shale sequence, although the lower limestone unit is sometimes missing. The upper three units of the Little Osage Member comprise the Houx Limestone. The lowermost, unnamed shale bed contains the Summit Coal Streak. The Blackjack Creek Member consists of a limestone/shale/limestone sequence. Chert nodules and beds (up to 2 feet thick) of chert are present in the Blackjack Creek limestones. Solution cavities or karst are not anticipated to occur in the Pennsylvanian-age limestone units. The Excello and Lagonda Formations consist of shale beds.

Bedding planes are nearly horizontal, with the entire geologic section dipping to the northeast at a rate of about one degree or about 90 ft per mile. However, local dips of individual shale and limestone beds may approach 5 degrees as the result of differential consolidation of coal and underclay seams.

The only known outcrop of the Fort Scott Formation in the project vicinity occurs along Interstate 170 near the Interstate 70 overpass. URS mapped this outcrop, and the results of the mapping activities are presented in the GDR.

3.6 GROUNDWATER CONDITIONS

Groundwater is expected to occur at several locations within the geologic profile, including the:

- interface of loess and residual clay;
- interface of residual clay and bedrock;
- open joints in the shale and limestone bedrock (particularly in the limestone beds);
- open bedding planes within the individual limestone beds;
- local bedding planes within shale beds;
- interfaces of limestone and shale beds; and
- along coal seams.

Groundwater level readings were obtained as part of the field investigations and indicate that, in general, groundwater was encountered about 10 to 20 feet (ranged from 1 to 45 feet) below existing grade, and most often encountered between El. 570 and 590. The groundwater levels in the borings are above the proposed tunnel crown elevations.

This section discusses previous construction experience relevant to tunnel construction and excavations for the portals and shafts.

4.1 TUNNELS

There is no local tunneling experience in the Pennsylvanian-age bedrock strata. Rock tunnels in the St. Louis area have been excavated predominantly in the Mississippian-age St. Louis Limestone Formation. The Mississippian-age deposits differ significantly from the Pennsylvanian strata in that the Mississippian deposits are primarily limestone, whereas the Pennsylvanian deposits are primarily shale with limestone and coal beds.

URS is aware of only one road cut excavated through Pennsylvanian-age bedrock. This road cut occurs near the intersection of Interstate 170 and Interstate 70, approximately 4 miles from the project site. Blasting was required to remove some areas of limestone, as evidenced by the blasthole traces in the rock. However, it is not known whether blasting was used for the entire excavation. Observations made at this road cut are discussed in detail in the GDR.

Several deep excavations in the Pennsylvanian Lagonda Formation have been made in the city of Clayton, Missouri (approximately 12 miles from the project site). Several excavations experienced significant magnitudes of bottom heave during construction as a result of swelling of the Lagonda shales. Furthermore, several structures experienced distress as a result of swelling pressures that were not countered by foundation loads.

4.2 EXCAVATIONS

At the time of this report, excavations for relocated Natural Bridge are underway. Of importance to this work is slope instability that occurred in April 2002 near the proposed western portal of the Coldwater Stormwater Tunnel. The instability occurred in the western slope for the cut for Lindbergh Boulevard. A backhoe was mobilized to assist with the investigation. Two test pits were made. A thin clean sand zone less than 18 inches thick was observed in one test pit between the residual clay and shale. In the other test pit a thin sandy clay zone was observed. This sandy layer was observed at about the same elevation, El 560, which is the approximate bottom of the movement. It is unknown as to the lateral extent of this sandy zone.

URS developed interpretive subsurface profiles along the tunnel alignments based on boring and laboratory data and our understanding of the site geology. Figures 5-1 and 5-2 provide the interpretive subsurface profiles along the Cowmire and Coldwater Stormwater Tunnel alignments, respectively. The following discussion describes the stratigraphy and engineering properties of the soil overburden and bedrock deposits. The results of laboratory tests are available in the GDR.

5.1 SOIL OVERBURDEN

The soil overburden consists of fill, loess, recent alluvium, and residual clay. Soil materials will be encountered in the cuts required for temporary portals, outlet structures, and access/drop shafts. Classification of soil materials was made in accordance with the Unified Soil Classification System (USCS) based on physical testing and/or visual examination. The following paragraphs describe soil deposits that will be encountered during this work.

5.1.1 Fill

Along the Cowmire Stormwater Tunnel alignment, fill will be encountered in shaft excavations, as shown in Figure 5-1. The fill thickness ranges from 0 to 25 feet, as indicated in Borings 1505, 1238, 1504, 1239, and 450. Along the Coldwater Stormwater Tunnel alignment, fill will be removed prior to tunnel construction and will not be encountered.

The fill encountered in the borings is primarily brown, reddish-brown, and gray, stiff, silty clay to clayey silt. In contrast to the natural soils, the fill contains trace quantities (less than 5%) of organics, sand, and gravel. The in-situ moisture content of the fill ranges from 10 to 29%. Local fill soils consist of loess borrow, and therefore, the plasticity characteristics of the fill soils would be similar to the loess described below. No laboratory strength tests were conducted on fill samples during this study.

5.1.2 Loess

Loess will be encountered in all open cuts made for the Cowmire Stormwater Tunnel, but will be encountered only in the shaft (near Sta. 10+00) excavation for the Coldwater Stormwater Tunnel. The loess encountered in the borings generally consists of brown and grayish-brown, medium stiff to stiff, low plasticity clayey silt and silty clay. The loess in the St. Louis area is typically termed "modified" loess to indicate that it has weathered from its original state. The weathering process has made the loess considerably more clayey, more plastic, and less sensitive than typical loess deposits. Soft zones, accounting for 13% of the loess sampled, also were encountered in the borings. Moisture contents in the loess averaged 25% and ranged from 9 to 52%. The liquid limit and plasticity index of the loess averaged 39 and 18, respectively. The undrained shear strength averaged 720 psf and ranged from 250 to 1280 psf.

5.1.3 Alluvium

Alluvium was encountered in Borings 1512 and 1205 and will be encountered in the excavation for the outlet structure of the Cowmire Stormwater Tunnel. The alluvium encountered in this area consists of stiff, high plasticity clay overlying soft to medium stiff, low plasticity clayey silt. The single set of water content, liquid limit, and plasticity index measured in the alluvial clay was 23, 50, and 34, respectively. The single set of water content, liquid limit, and plasticity index measured in the alluvial silt was 29, 32, and 8, respectively. No laboratory strength tests were conducted on alluvial soil samples.

5.1.4 Residual Clay

Residual clay was encountered in most borings and will be encountered in all open cuts made for this work, with the exception of the excavation for the outlet structure of the Cowmire Stormwater Tunnel. In addition, a reach of the Coldwater Stormwater Tunnel (from approximately Sta. 6+00 to 7+50) will be constructed through residual clay.

The residual clay consists of brown, reddish-brown, and gray, stiff to very stiff, low to high plasticity clay. The median water content of the residual clay is 25% and ranges from 9 to 54%. The median liquid limit and plasticity indices are 53 and 28, respectively. The residual clay contains remnant fragments of the parent rock ranging in size from gravel to boulder. The maximum size boulder based on observations of current AEP excavations and other regional excavations is not greater than 24 inches in maximum diameter. Only three unconfined compression tests were conducted on samples of residual clay for this project. The undrained shear strengths from these three tests were 1090, 1900, and 3900 psf.

For the AEP, laboratory "free swell" tests were conducted to evaluate the swell potential and "swell pressure" of selected residual clay and shale samples due to the presence of illite and smectite clay minerals in the shale. No swell tests were conducted on residual clay samples for the Stormwater Tunnel projects; therefore, the results of swell tests conducted on residual clay samples from other AEP borings were evaluated. Figure 5-3(a) and (b) present histograms of free swell and swell pressure for AEP residual soil samples, respectively. The results of these swell tests are presented in the GDR. These data indicate that about 75% of the residual clays tested have a free swell of less than 2% and a swell pressure of less than 20 psi. However, two residual clay samples (of the eight samples tested) had free swells of about 5% and swell pressures over 50 psi.

5.1.5 Summary of Soil Properties

Table 5-1 summarizes index and engineering properties for the overburden soils. Because the tunnels are primarily in bedrock, only a limited number of laboratory tests were conducted on soil samples. As a result, many parameters have no data or too few data to reasonably estimate statistical values such as mean, median, and standard deviation.

5.2 BEDROCK

Bedrock will be encountered in all elements of this work. The two main rock types encountered in the explorations were limestone and shale. In addition, a thin coal unit and chert zones were encountered in some of the borings. The chert is present within the limestone units, particularly in the Blackjack Creek Member of the Fort Scott Formation, and is up to 2 feet thick. The bedrock that will be encountered in the Cowmire Stormwater Tunnel outlet and access/drop shafts will consist of interbedded limestone and shale of the Little Osage and Blackjack Creek Members of the Fort Scott Formation. The Cowmire Stormwater Tunnel itself will encounter primarily interbedded limestone and shale (and coal and chert seams) of the Blackjack Creek Member. Bedrock encountered in the Coldwater Stormwater Tunnel, shafts, and outlet will be interbedded limestone and shale of the Little Osage Member.

5.2.1 Limestone

Limestones of the Fort Scott Formation (Little Osage and Blackjack Creek Members) will be encountered in excavations for the Cowmire and Coldwater Stormwater Tunnels. The dry unit weight of the limestone averages 165 pcf. Thirteen compressive strength tests were conducted on limestone samples. The median unconfined compressive strength (UCS) of the limestone is about 10,000 psi, with values ranging from about 4,500 to 26,600 psi. Figure 5-4 presents a histogram of the UCS results for limestone samples. It should be noted that none of the limestone samples contained significant amounts of chert. The presence of chert, as thin beds or nodules, will increase the compressive strength of the limestone. Fourteen point load index (PLI) tests were conducted on limestone samples. At six locations, UCS values also were available to calculate a toughness index (UCS/PLI). The average toughness index for the limestone samples is 34. The presence of chert, as thin beds or nodules, also will increase the toughness of the limestone.

Petrographic analyses were conducted on three limestone samples – one sample from the lower limestone unit of the Little Osage Member and two samples from the upper limestone unit of the Blackjack Creek Member. The samples were characterized broadly as fossiliferous limestone, and are composed primarily of calcite (89-99%), with minor (less than 8%) amounts of aragonite, dolomite, and illite. The Moh's Hardness (H) of calcite is 3 to 4. Limestone samples with chert were not tested. As previously indicated, chert can occur as nodules within the limestone matrix or as distinct beds up to 2 feet thick. The Moh's Hardness of chert is approximately 7.

Cerchar Abrasivity Index was determined for eight limestone samples – two samples were from the lower limestone bed of the Little Osage Member and six from the upper limestone bed of the Blackjack Creek Member. One, and possibly two, limestone samples contained chert nodules and did not exhibit significantly larger values of Cerchar Abrasivity Index. The Cerchar Abrasivity Index averaged 1.9 with a range of 1.6 to 2.4.

5.2.2 Shale

Shales of the Fort Scott Formation exhibited different water contents and strength properties than those of the Excello and Lagonda Formations. Therefore, the following discussion of shale properties is separated into properties representative of the Fort Scott Formation and those representative of the Excello and Lagonda Formations.

Fort Scott Formation Shales. The Fort Scott Formation shales generally are very weak, weathered to highly weathered, and gray. Water contents in the Fort Scott shales averaged 18% and ranged from 5 to 50%. The water content varied with depth and degree of weathering. The total unit weight of the shale averaged about 132 pcf. Fourteen UCS tests were conducted on Fort Scott shales. The median UCS is 28 psi, with values ranging from 11 to 115 psi. Figure 5-5 shows the distribution of UCS values for the Fort Scott Formation shales. It should be noted that coal beds encountered within shale units tend to be weaker than the surrounding shale.

X-ray diffraction analyses were conducted on three shale samples – two samples from the lower shale unit of the Little Osage Member and one sample from the Blackjack Creek Member. These analyses indicated that the Fort Scott shales are composed primarily of the following minerals: mixed layer illite/smectite, discrete illite, kaolinite, quartz, and chlorite. Illite and smectite have moderate and high swelling potential, respectively. Therefore, the mixed layer illite/smectite is anticipated to exhibit moderate to high swelling potential. The presence of quartz in the shale is minor (less than 2%).

Swell potential tests were conducted on shale samples along the Cowmire and Coldwater Tunnel alignments. The presence of chert, as thin beds or nodules, will increase the compressive strength of the limestone. As discussed above, due to the limited number of swell tests conducted on borings drilled for this project, the results of swell tests conducted on shale from other AEP borings are incorporated in this discussion. The results of these additional tests are included in the GDR. Figure 5-6(a) and (b) presents histograms of free swell and swell pressure, respectively, for all Fort Scott shale samples collected for the AEP. Approximately 70% of the shale samples have a free swell of greater than 2%, and free swells as high as 15% were measured. Approximately 70% of the shale samples have a swell pressure of less than 30 psi, but swell pressures as large as 167 psi were measured.

Slake durability index was evaluated for seven Fort Scott shale samples – one upper shale bed of the Little Osage Member, and six from the lower shale bed of the Little Osage. Table 5-2 provides a summary of slake durability index values. Based on Gamble's (1971) durability classification, the Fort Scott shales have "very low" durability.

Excello and Lagonda Formation Shales. The Excello and Lagonda shales are greenish-gray and maroon, very weak, and slightly to moderately weathered. Water contents in these formations average approximately 6% and range from 4 to 12%. The total unit weight of the shales averages about 155 pcf. Four UCS tests were conducted on Excello and Lagonda shales. Understanding that there are limited data for statistical analysis, the median UCS is about 1080 psi, with values ranging from 516 to 1570 psi. Figure 5-7 shows the distribution of UCS values for the Excello and Lagonda Formation shales.

No swell tests were conducted on Excello or Lagonda Formation shales for this project. However, based on our local experience with the mineralogy and behavior of deep excavations in these formations, we would anticipate similar swell characteristics to the measurements reported for the Fort Scott Formation.

Slake durability indices were determined for two samples of Excello or Lagonda Formation shale. These test results are provided in Table 5-2. Based on Gamble's (1971) durability classification and our experience, the Excello and Lagonda shales typically have "low" durability.

5.2.3 Summary of Rock Properties

The results of all index and engineering property tests as well as descriptions of the testing procedures are provided in the GDR and were briefly discussed in the preceding paragraphs. Table 5-3 presents a summary of the tests conducted on rock samples.

5.3 ROCK STRUCTURE

As used herein, bedrock structure refers to discontinuities including joints, faults/shears, and bedding plane separations, as well as in-situ stresses. Joints and bedding plane separations also are referred to collectively as fractures. Discontinuities of the Fort Scott Formation were evaluated by mapping a single outcrop, detailed examination of recovered rock core, and review of pertinent geologic literature. The GDR provides additional details on the evaluation of discontinuities.

5.3.1 Joint Sets

Two high angle joint sets occur in the Fort Scott Formation. (For reference, the Cowmire Stormwater Tunnel follows a bearing of N40°W and the Coldwater Stormwater Tunnel bears at N65°W.) Details of joint characteristics are provided in the GDR.

- Joint Set 1 strikes at approximately N70°W (mean value, range of N50°W to N85°W) and dips at approximately 80°N (mean value, range of 45°N to 45°S). A stereoplot of joint orientation is included in the GDR. Joint Set 1 is vertically persistent through the limestone and shale beds. The horizontal persistence of this joint set is greater than 50 feet and the median spacing (measured on a horizontal plane perpendicular to strike) is approximately 17 feet.
- Joint Set 2 strikes at approximately N50°E (mean value, range of N20°E to N70°E) and dips at approximately 85°N (mean value, range of 35°N to 80°S). A stereoplot of joint orientation is included in the GDR. Joint Set 2 also is vertically persistent through the limestone and shale beds. The horizontal persistence of this joint set is greater than 50 feet and the median spacing (measured on a horizontal plane perpendicular to strike) is approximately 11 feet.

5.3.2 Joint Characteristics

Joint characteristics include aperture, infilling, shape, and roughness. Joints near the ground surface through limestone are expected to have apertures of approximately 1 inch (ranging from ¼-inch to over 12 inches), to have firm infilling consisting of weathered limestone, and to be undulating or irregular and slightly rough. Joints through shale are expected to have similar apertures, to have firm infilling consisting of clay and shale, and to be planar and slickensided. At tunnel depths, we anticipate that the joint characteristics will be similar to those observed near the surface, except that joint apertures should be smaller. (Refer to ISRM 1972 for definitions of all geologic terms.)

5.3.3 Faults and Shear Zones

No faults or shear zones were observed in the outcrops, and geologic literature does not indicate that faults and large shear zones exist in the project vicinity. Furthermore, no local folding or bedding plane shears are anticipated in the project vicinity.

5.3.4 Bedding Conditions

Bedding thickness in the limestone averages approximately 3 to 4 inches and ranges from 0.1 inches to about 20 inches. Bedding plane separations in the limestone are typically tight (aperature less than 1/16-inch) and slightly wavy. The regional dip of the bedding is nearly horizontal. Bedding plane separations (or fractures) control the fracture frequency of the core. The fracture frequency in the limestone core is shown on Figures 5-1 and 5-2.

Due to the severe deterioration of the shale core, no observations of shale bedding thickness could be made. Geologic literature indicates that bed thickness of the shale is on the order of a few inches to a few feet.

5.3.5 In-Situ Stresses

URS evaluated in-situ stresses in the Pennsylvanian-age shales as part of our *Final Geotechnical and Foundation Engineering Report for Lindbergh Tunnel*, dated October 2001. This report is available upon request. Based on the pressuremeter test results and empirical correlations with overconsolidation ratio and shear strength, the horizontal stress in the shale bedrock is approximately 1.3 times greater than the vertical stress. This corresponds to a K_o value of 1.3.

5.4 GROUNDWATER AND HYDROGEOLOGIC CONDITIONS

Hydrostatic groundwater levels are expected to range generally from about 5 to 75 feet above the tunnel inverts as indicated in Figures 5-1 and 5-2. In-situ hydraulic conductivity tests (packer tests) that were completed in borings indicate hydraulic conductivity values representative of the rock mass at tunnel depth from $3x10^{-7}$ to $1x10^{-5}$ cm/sec, with an average value of approximately $1x10^{-6}$ cm/sec. It should be noted that of the twenty-four tests where total inflow was reported (see test results in the GDR), seventeen tests had no water take. These tests were not included in the average value. We anticipate that the average value of $1x10^{-6}$ cm/sec is representative of the intact limestone beds near the tunnel depth. The intact shale beds are less permeable than indicated by this average value.

Two sites near the Airport Expansion Program contain excavations through similar materials. Actual seepage rates were observed in a quarry (3 miles west) and a new roadway cut for Interstate

70 (5 miles east). Seepage was observed at two locations in the quarry. Seepage estimates were 5 gpm over a 300 ft face and 25 to 50 gpm over a 1000 ft face. Seepage was observed at one area of the roadway cut of 5 gpm over an area of 30,000 ft^2. Observations were made after the excavations had been made, therefore initial flush flow quantities were not observed.

5.5 GAS CONDITIONS

Coal formations will be encountered in the tunnel and shaft excavations. No obvious signs of flammable gas (such as an oily sheen on drilling mud, or bubbles or odors from the borehole) were observed during the field explorations; however gas detectors were not used during the drilling operations. The coal is at least 290 million years old. It is likely that all of the volatile organics (CH_4 material) have been dissipated through volatilization. This process of volatilization leaves only carbonaceous material and potentially sulfides. However, no indications of sulfur-bearing compounds were observed during studies made for this work.

This section of the report presents the methods used to evaluate ground behavior and evaluate ground loads for design of primary and permanent tunnel linings. In addition, design recommendations are presented for various elements of the permanent construction. Considerations for design of temporary construction elements are provided in Section 7.

6.1 METHODS TO EVALUATE GROUND BEHAVIOR

Several methods were used to evaluate ground behavior and loads for design of the primary and permanent tunnel linings. These methods include correlations based on ground behavior classifications and ground loads from calculated modified overload factors and measured swell pressures. The anticipated ground loads predicted by these various methods were compared to develop recommendations for the permanent lining system.

6.1.1 Ground Behavior Classification

Terzaghi (in Proctor and White 1946) proposed ground behavior terms to describe rock mass behavior during tunneling and to characterize ground loads. The Terzaghi classification system is defined in Table 6-1. Based on these definitions, the shale formations are classified as swelling and squeezing ground. Where the shales are not swelling or squeezing, raveling behavior is anticipated. The limestones are classified as hard, stratified rock. Terzaghi and Deere et al. (1970) provided recommendations for permanent or final ground loads on tunnels based on ground classification. These suggested final ground loads are included in Table 6-1.

6.1.2 Modified Overload Factors

Deere et al. (1969) indicated that overstressed rock conditions will result in the development of a plastic zone around the tunnel perimeter when the modified overload factor is greater than unity. The modified overload factor (OFM) is defined as:

$$OFM = \frac{s_q}{UCS}$$

where σ_θ is the average tangential stress at the tunnel perimeter and UCS is the unconfined compressive strength of the rock. For the horizontal to vertical stress ratio (K_o) of approximately

1.3 expected at the project site, the average tangential stress for an assumed circular opening is approximately $2.3\sigma_v$ (where σ_v is vertical total stress). Jethwa et al. (1984) indicated that OFM values of 1 to 2.5 correspond to mildly squeezing conditions, 2.5 to 5 to moderately squeezing conditions, and 5 or greater to highly squeezing conditions in clayey materials. Cording (1984) indicated that an OFM value of greater than 0.5 can lead to minor stress slabbing in harder, sedimentary rocks, such as limestone. Stress slabbing refers to the formation of new fractures and spalling of slabs or blocks of rock from the tunnel crown or walls. Table 6-2 summarizes the modified overload factors for an average cover of 60 feet and a maximum cover of 100 feet. Based on this evaluation, the Fort Scott shales are expected to behave as moderately to highly squeezing ground. The Excello and Lagonda shales are not expected to exhibit squeezing or stress slabbing. The limestone is not expected to exhibit stress slabbing. Final rock loads for squeezing ground are included in Table 6-1.

6.1.3 Swell Potential

Several investigators have developed correlations between tunnel behavior or ground load and the swell potential of overconsolidated, clayey materials. Terzaghi (in Proctor and White 1946) indicates that rock with a free swell greater than 2% will behave as swelling ground. Kormornik and David (1969) developed empirical correlations between swell pressure, liquid limit, and dry density. Heuer (1974) indicated that heavily preconsolidated clay (or shale) with a plasticity index greater than 30 and a significant percentage of smectite will behave as swelling ground. Howard et al. (1975) indicated that clays or shales with a swell pressure greater than 36 psi are likely to present significant swelling problems, while materials with a swell pressure less than 22 psi are very unlikely to present significant swelling problems. Mesri et al. (1994) indicated that illite has a moderate swelling potential while smectite has a high swelling potential. They also indicated that the swell pressure measured in a free swell/reload test (as used on AEP samples) is typically somewhat larger than the pressure mobilized against a rigid support in a tunnel.

Figure 6-1 presents correlations of free swell and swell pressure with Atterberg limits for residual clay and Fort Scott shale samples. As noted earlier, the x-ray diffraction tests indicated that the shales are composed primarily of the clay minerals illite and mixed layer illite/smectite. The liquid limits of the samples tested are generally between 40 and 50 and the plasticity indices are between 20 and 30, indicating moderate swelling potential. As illustrated in Figure 6-1, the swelling pressure is generally less than 30 psi, but pressures as large as 167 psi were measured in the laboratory. Furthermore, the measured swelling pressures generally are larger than that predicted by Kormornik and David (1969).

Based on the measured data for the Fort Scott shales and local experience in the Lagonda Formation shale, we anticipate that final ground loads due to swelling will approach 150 psi for the Fort Scott, Excello, and Lagonda Formation shales.

6.2 TRUNK SEWER TUNNELS

The final tunnel linings (i.e., the trunk sewers) for the Cowmire and Coldwater Stormwater Tunnels are to be 102-inch and 72-inch (minimum) ID pipes, respectively, conforming to Contract Specifications. Pipe material is the choice of the Contractor, within the limitations set in the Contract Specifications. This section discusses the ground loads that will act on the final tunnel lining and requirements for sewer pipe backfill.

6.2.1 Ground Loads

Ground loads were evaluated using the methods described in Section 6.1. The predicted ground loads were compared and a reasonable upper bound load, based on the distribution and extent of very poor quality shales and engineering judgment, was selected for the purpose of final lining design.

The permanent ground loads on the tunnel are controlled by the swelling behavior of the shale. Load will tend to build over time as the shale swells. The actual swelling pressure on the liner will depend on factors such as the time between removal of soil and placement of primary support, the contact between the primary support and the ground, and the contact between the final lining and the ground. The final lining system for both tunnels should be designed to withstand a combined ground pressure of 180 psi (150 psi ground pressure and 30 psi hydrostatic pressure above the tunnel springlines). The value of 150 psi ground pressure incorporates nearly all of the swell test data, and corresponds to a value slightly larger than the average load estimated using the Q-system.

6.2.2 Sewer Pipe Backfill

Tunnel Backfill. After installing the pipes in the tunnels and securing them in place, the annular space outside the pipes is to be filled with low density cellular concrete, flowable fly ash-cement mixture, or another material conforming to requirements of the Contract Specifications. The method of installing the pipes and blocking and securing them in place to avoid flotation during backfilling is to be determined by the Contractor within the requirements of the Contract Specifications, subject to review by the Engineer.

Trench Backfill. Bedding material is placed at the bottom of the trench excavations to provide uniform support of the sewer pipe. The pipe bedding is defined as that portion of the pipe zone backfill between the base of the excavation and the springline of the pipe. The pipe zone extends from the pipe bedding material to 6 inches above the pipe crown. At least 4 to 6 inches of bedding material should be placed below the pipe invert, as indicated in the Contract Drawings and Specifications. Bedding material should consist of sound, durable sand or gravel (such as crushed limestone and screenings) that can be easily placed and compacted. The gradation of the bedding material should meet the requirements in the Contract Specifications. If wide bell and spigot pipe joints are used, bell holes should be excavated in the bedding material at required locations to allow uniform bearing of the pipe on the bedding material.

In cut and cover segments, the contractor may elect to use granular backfill or a controlled low strength material (CLSM) for trench backfill. If granular backfill is selected, the same bedding material as the pipe zone backfill should be used. The material should be placed in lifts not exceeding 8 inches in uncompacted thickness and compacted with hand-operated compaction equipment to the requirements indicated in the Contract Specifications. Compaction of the bedding or the pipe zone backfill by jetting with water is not permitted.

6.3 GENERAL BACKFILL AND COMPACTION REQUIREMENTS

Significant quantities of shale and limestone bedrock will be excavated as a part of this Work. If used as fill to re-establish final grade, the shale will need to be pulverized by discing and tracking upon excavation in order to be used as a structural fill. The natural water content is anticipated to be less than the optimum water content for compaction. Therefore, the addition of water is anticipated for compaction if used as fill. The shale fill should be placed in non-structural areas because of its swell potential and the difficulty in working it. The size of limestone fragments used in structural fill should be limited to a maximum of 6 inches in size in any direction and large fragments should be surrounded by soil fill, not “nested.” Nesting of large diameter fragments creates voids and potential for settlement due to migration of soils into the voids. Detailed backfill requirements are provided in the Contract Specifications.

Fill soils should be placed in thin lifts (8- to 10-inches thick prior to compaction) and compacted by multiple passes of kneading-type vibratory rollers per the Contract Specifications. Assuming a 90 percent ASTM D-1557 (modified Proctor) compaction criteria, the estimated volumetric shrinkage (from excavated to re-compacted volume) is provided in Table 6-3. These recommendations are based on testing described in URS’ *Final Geotechnical and Foundation Engineering Report for Runway and Taxiways*, dated Feb. 2002.

6.4 PERMANENT CUT SLOPES

Permanent cut slopes will be required at the outlets for the Cowmire and Coldwater Stormwater Tunnels. These cut slopes will be made through a combination of loess, residual clay, and highly weathered shale and limestone. We recommend the following criteria for the design of permanent cut slopes.

Slope Geometry. Permanent cuts in soil or rock should be sloped at 3H:1V or flatter. Permanent slopes greater than 35 feet in height should have a 70-foot wide bench for every 35-foot change in elevation, as illustrated in Figure 6-2.

Permanent Erosion Control. Requirements for permanent erosion control are described in the Contract Specifications.

Surface Drainage. Paved ditches are required along permanent slopes for collection and discharge of surface water, as shown in the Contract Drawings.

6.5 PERMANENT RETAINING STRUCTURES

Permanent retaining structures are required at the outlets. These structures will consist of reinforced concrete headwalls and concrete cantilever walls. These structures should be designed for active earth pressures. Table 6-4 presents equivalent fluid pressures applicable for various cases. Table 6-4 also presents allowable bearing capacities for retaining structures founded on natural soil deposits. Backfill for the structures may consist of natural soils or imported granular fill. Backfill may be sloped away from the wall at 3H:1V or flatter. If backfill is sloped at 3H:1V from the wall, active earth pressures should be increased by a factor of 1.35.

The drop shafts and manholes required for the tunnels should be designed for at-rest earth pressures. Table 6-4 also presents equivalent fluid pressures applicable to the drop shafts and manholes for various cases.

The loads outlined in Table 6-4 do not include permanent surcharges or seismic loads. Surcharge loads can be estimated using elastic theory (i.e., Boussinesq). Loads calculated by elastic theory should be doubled for non-yielding structures.

6.6 INSTRUMENTATION AND MONITORING

Instrumentation will be required to: (1) monitor deformations of the tunnel excavation; (2) monitor performance of the portal/shaft excavations; (3) measure groundwater discharge volumes; and (4) monitor for potentially hazardous gases during tunnel and shaft construction. The Contractor will be responsible to furnish, install, and maintain the instrumentation except for the inclinometers. The Contractor will locate the instrumentation within the guidelines of the Contract Specifications. The Contractor will be responsible to monitor tunnel convergence instrumentation, and the Owner’s representative will obtain readings for the instruments monitoring portal/shaft excavations. In addition, the Contractor will be responsible for obtaining readings for water flow and monitoring for gases. The following paragraphs describe the basic elements of the instrumentation program.

Tunnel Instrumentation. The purpose of the tunnel instrumentation is to verify that convergence is controlled and the tunnel has stabilized prior to installing the carrier pipe. Instrumentation will consist of convergence reference points for tape extensometer measurements or borehole extensometers, at the Contractor's option. Instrument locations and quantities are indicated on Contract Drawing CT1-01. The contractor will monitor the convergence instruments according to the schedule provided in Table 6-5. Results of convergence monitoring should be submitted to the Owner's Representative within 24 hours of measurements. If the tunnel convergence exceeds 1 percent of the tunnel diameter, the Contractor is responsible to evaluate the source of the convergence and to submit a corrective action plan to the Owner's Representative.

Portal/Shaft Excavation Instrumentation. The stability of the cut slopes at the all portal excavations will be monitored for slope movements throughout construction. Monitoring instrumentation will consist of inclinometers. The inclinometer locations and quantities are indicated on Contract Drawing CT1-01. Sheet CT1-01 also provides a schematic for inclinometer installation. The Contractor shall provide access to the instrumentation by the Owner's representative.

Groundwater Discharge. Groundwater inflows into the tunnel excavations will need to be removed using pumps and properly disposed. Flowmeters are required on water supply lines to measure the amount of water being pumped into the tunnels for construction purposes (i.e., construction make-up water), and at the downstream ends of the discharge lines to monitor the amount of groundwater and construction water being pumped from the tunnels.

Air Monitoring. The mere presence of coal in the borings will require the contractor to monitor for methane, hydrogen sulfide, and carbon monoxide during tunnel and shaft construction.

The Cowmire and Coldwater Stormwater Tunnels incorporate several construction elements. These include: (1) portal and/or work shaft construction at the ends of the alignments; (2) trunk sewer tunnel excavation; and (3) drop/access shaft excavations. This section of the report describes the anticipated sequence of construction. In addition, anticipated ground conditions and behavior; excavation methods; primary lining requirements and construction issues; and groundwater inflows are addressed for each construction element. In addition, other considerations such as water and muck disposal and monitoring requirements are discussed.

7.1 CONSTRUCTION SEQUENCING

At the time of tunnel construction, grades will be as shown on the Contract Drawings. The direction of tunneling for both tunnels is left to the contractor, although we anticipate that tunneling uphill is preferred to promote gravity drainage. In this case, portals at the west end of the Cowmire Stormwater Tunnel alignment (near Sta. 0+70) and at the east end of the Coldwater Stormwater Tunnel alignment (near Sta. 14+40) will be required. We anticipate that the contractor will elect to excavate large diameter shafts to facilitate egress from the trunk tunnel excavations and construction of the drop shaft (SP 20 along the Cowmire Stormwater Tunnel alignment) and access shaft (SP 840 along the Coldwater Stormwater Tunnel alignment). Work shafts for removal of a tunneling machine should be completed prior to launch due to potential stability problems related to the residual clay/shale interface.

Timing of construction for the smaller diameter access shafts (SP 12, SP 14, SP 16, and SP 860) and the manholes for the 8-inch PVC line are at the Contractor’s option. Regrading for the Cowmire Detention Basin will start following the start of the Cowmire Stormwater Tunnel construction. Final grading for the proposed runway is not scheduled for completion until after Coldwater Stormwater Tunnel construction. Therefore, the final top of shaft SP 860 will be approximately 37 feet lower (or less if runway grading is underway when the tunnel contractor completes the tunnel) than the grade at the time of construction.

7.2 PORTAL/SHAFT DEVELOPMENT

Temporary portals and/or work shafts will be required at the east and west ends of both tunnels. In addition, a number of access shafts are required along the tunnel lengths, as indicated in the Contract Drawings. Excavations for the portals and shafts may involve cut slopes and/or earth retention systems. However, due to the recent slope instability adjacent to the western portal of the Coldwater Stormwater Tunnel, the Contractor is required to use earth and rock retention systems for this excavation, rather than an open cut. This section presents criteria for the design and construction of temporary elements of the work.

7.2.1 Anticipated Ground Conditions and Behavior

Excavations for the portals/shafts for the Stormwater Tunnels will encounter a wide range of materials, including loess, residual clay, alluvium, shale and limestone. The interpretive geologic profiles for the Cowmire and Coldwater Stormwater Tunnel alignments in Figures 5-1 and 5-2, respectively, provide distributions of material types that will be encountered at the portals/shafts. Table 7-1 provides approximate volumetric material percentages that can be expected for the shafts. The method to determine these percentages is described in the table. Volumetric material percentages are not provided for the ingress portals because these values will depend on the selected excavation method (i.e., open cut, earth retention, and/or rock retention).

Soil. The soil overburden consistency is generally stiff. The residual clays are weathered from shale and limestone, and as such, will contain decomposed fragments of the parent rock of various sizes – up to approximately 24 inches in diameter. Excavations made in soils will present the following difficulties:

- The loess is erodible and will be removed by surface run-off. Therefore, surface water should be diverted from any cuts or slopes.
- The residual clays (and shales) will soften and swell where exposed to ponded water, e.g., in the bottom of a cut. On cut faces or sideslopes, the residual clay will tend to ravel and slough as a result of secondary structure, e.g., fissures. This problem will be exacerbated by exposure to water and slopes steeper than recommended herein. Therefore, surface water should be diverted from excavations and runoff and seepage water should not be allowed to accumulate at excavation bottoms.

- Based on local landslide experience, the contact between the residual clay and the weathered shale is a known plane of weakness. Over time, the shear strength along this contact will decrease, potentially leading to slope instability. Therefore, cuts that are to be open for more than one year should be considered permanent and laid back at a 3H:1V slope, as recommended in Section 6.
- Excavation bottoms in the high plasticity residual clay and shales will present trafficability problems. As such, excavation floors in these materials should be undercut a minimum of 12 inches, the base proof-rolled, and the removed soil replaced with a geotextile separation layer (e.g., Mirafi 180N or equivalent) and crushed rock.

Limestone. The limestone is slightly to moderately weathered, strong, and contains chert nodules, as well as distinct beds of chert up to two feet thick. For baseline purposes, the chert content of the limestone is assumed to be 5% of the total limestone volume. During open excavation, the limestone will break along bedding planes to create relatively planar surfaces. Slopes in limestone will likely exhibit a stair-step shape.

Shale. The Fort Scott Formation shales are moderately to highly weathered, very weak and soil-like, and contain slickensides. The Excello and Lagonda Formation shales are slightly to moderately weathered and very weak. Exposed shale in excavation bottoms will soften and swell over time. This problem will be exacerbated by exposure to ponded water. At excavation bottoms, softening of shales will present trafficability problems. Therefore, the portal floors should be undercut a minimum of 12 inches, and the removed shale should be replaced with a geotextile separation layer (e.g., Mirafi 180N or equivalent) and crushed rock. Because swelling of the shale occurs over a period of time, occasional maintenance or regrading of the floor should be expected. Alternately, cast-in-place concrete working slabs can be constructed near the portal faces during tunneling to prevent deterioration of the shale due to construction traffic. Shale in sideslopes will ravel and slough, presenting overhead hazards to workers in the excavation.

7.2.2 Excavation Methods

Soils, as shown in Figures 5-1 and 5-2, can be excavated using typical soil excavation equipment. This includes bucket excavators, scrapers, bulldozers, earth augers (on drilled shaft rigs), and similar equipment.

We anticipate that the shale will require a D-10 (or equivalent) equipped with a ripper for removal. Generally, the limestone is too strong to excavate using a ripper. It will require a hoeram or splitter to remove. Blasting also may be used to remove areas of strong limestone. However, typical soil excavation equipment (e.g., bulldozers) may be able to “pop out” thin limestone beds encountered in hillside excavations. Such thin beds will be encountered near the western portal area of the Cowmire alignment. For drilled shaft excavation, we anticipate the much of the shale can be removed with an earth auger. The limestone will require a core barrel or rock auger with carbide teeth for removal.

7.2.3 Slope Stability and Earth Retention

Portal and shaft excavations can be accomplished by laying slopes back to a stable configuration and/or using temporary shoring, such as sheet pile walls, soldier pile and lagging walls, ring beams and lagging, liner plate, or steel casing to maintain near vertical side slopes.

Soil Excavations. Temporary soil slopes should be no steeper than 2H:1V. If the slope will be open for more than one year, it should be considered permanent and cut at 3H:1V. Furthermore, if the cut slope height is greater than 35 feet, it should be considered permanent and designed with intervening benches as recommended in Section 6. Earth pressures for design of temporary shoring for excavations in soils are presented in Figure 7-1. Soil nails, if used, should meet the criteria described in the following paragraph regarding rock anchors. Excavation bottoms should be should be undercut a minimum of 12 inches, the base proof-rolled, and the removed soil replaced with a geotextile separation layer (e.g., Mirafi 180N or equivalent) and crushed rock.

Rock Excavations. Temporary rock slopes should be no steeper than 1.5H:1V. Alternately, the Contractor may elect to excavate near vertical slopes in the bedrock (1H:8V) and provide rock reinforcement or anchors. If excavations in rock are to be open for more than one year, the excavation should be considered permanent and designed in accordance with the recommendations provided in Section 6. Temporary rock anchors should be designed in accordance with the following criteria:

- Anchors should be fully grouted, 60 or 75 ksi deformed steel bars.

- Anchors should be on a center-on-center spacing (horizontally and vertically) of not greater than 6 feet.
- Allowable bond stresses for anchors are presented in Table 7-2.
- Anchors should be sufficient to support a rock wedge that daylights at the toe of the excavation and extends upward with a 1.5H:1V dip, as shown in Figure 7-2. The wedge should be assumed to extend over the length of the cut.
- Shotcrete should be reinforced using a welded wire fabric or steel fibers (or other approved alternative) and should be at least 3 inches thick.

If shoring is used to retain soils in conjunction with rock slopes, a minimum 25-foot wide bench should be maintained between the base of the shoring and the cut face of the bedrock. The configuration of the excavation and design of any support systems are left to the Contractor, subject to review by the Engineer.

7.2.4 Groundwater

The base of the excavations for the portals/shafts are expected to be below the groundwater level during construction. Groundwater levels were measured in the test borings are shown in Figures 5-1 and 5-2. The groundwater level varies with the surface topography and reaches maximum levels of approximately 50 feet above the excavation bases in the portal areas. Bidders should anticipate and be prepared to handle groundwater inflows into the excavations of approximately ¾ gpm per linear foot of open cut.

Surface water resulting from rainfall will flow into the excavations unless measures are taken to divert surface flow. Sandbagging, berms, or diversion channels should be used to divert surface water from the excavations. The residual clays and shales will soften and rapidly deteriorate if exposed to standing water, so it will be important to have an effective drainage and sump system at the base of the cuts to maintain a stable surface that can support construction equipment. The drainage system should be designed to efficiently collect groundwater seepage and surface water drainage into the excavation so it can be pumped out. The bottom of the excavation should be sloped to drain away from the portals and be provided with a gravel drain blanket and a geotextile filter fabric in addition to a system of drainage trenches (i.e., french drains) to control and collect seepage and stormwater in the excavation. The drains should direct the seepage and stormwater to a sump with a pump to remove the water and maintain a firm, dry working surface. Drainage trenches or horizontal drains should be constructed to intercept pervious sand and gravel layers exposed in the excavation, collect the water, and divert it to the sump. Water pumped from the excavations is to be treated and disposed of as described in Section 7.4.

7.3 TRUNK SEWER TUNNELS

The trunk sewers between Sta. 0+80 and 31+92 for the Cowmire Stormwater Tunnel and Sta. 0+00 and 14+28 for the Coldwater Stormwater Tunnel are required to be installed by tunneling. The tunnels may be constructed using a tunnel boring machine (TBM) or by drill-and-blast. Primary supports will be designed by the Contractor in accordance with the criteria given herein.

This section of the report discusses anticipated ground conditions and behavior for tunneled reaches of the stormwater sewer alignments based on the Contractor’s selected means and methods. Design criteria and ground loads to be considered for primary supports are provided and baseline ground water inflows are quantified.

7.3.1 Anticipated Ground Conditions and Behavior

The trunk sewer tunnels will be excavated primarily in interbedded shales and limestones of the Fort Scott Formation, as indicated in Figures 5-1 and 5-2. The western portion of the Cowmire Stormwater Tunnel alignment will be mined in part through Excello and Lagonda Formation shales. In addition, a reach of the Coldwater Stormwater Tunnel (from approximately Sta. 1+00 to 2+50) and possibly a portion of the Cowmire Stormwater Tunnel (near Sta. 10+00) will be mined through residual clay. Based on the interpretive distribution of rock shown in Figures 5-1 and 5-2, URS determined volumetric percentages of materials encountered in the Cowmire and Coldwater Stormwater Tunnels. Table 7-1 presents the distribution of materials in the tunnels.

Borings also encountered highly weathered coal seams (smut) in the shale beds and chert nodules in the limestone beds as well as distinct chert seams up to 2 feet thick. For baseline purposes, the chert content of the limestone is assumed to be 5% of the total limestone volume. This value is based on the frequency of chert seams encountered in the borings.

Face conditions in the Cowmire Stormwater Tunnel will range from 100% shale to 100% limestone, and can be any combination of shale and limestone. Face conditions in the Coldwater Stormwater Tunnel will range from 100% shale to approximately 50% shale and 50% limestone. In addition, a reach of the Coldwater Stormwater Tunnel (from approximately Sta. 1+00 to 2+50) will encounter 100% residual clay face conditions and mixed face conditions between soil and rock.

Fort Scott Formation Shale. When mined, the Fort Scott Formation shales will behave as swelling ground. Swelling behavior will be exacerbated by seasonal variations in precipitation or introduction of construction make-up water. Where the overburden thickness is greater than about 50 feet, the Fort Scott shales also will behave as squeezing ground. In the tunnel heading, these shales will exhibit raveling behavior prior to primary support placement. Lastly, high-angle slickensided joints in the Fort Scott shales may form unstable wedges in the roof and sidewalls.

The behavior of the ground that is most relevant to tunnel construction will depend on the selected means and methods of construction, as well as workmanship. If left unsupported in the heading, the shales will ravel, slab, and slake. Overbreak should be anticipated by bidders during excavation of the shale, regardless of the methods used to excavate the shale. Overbreak will occur due to the low tensile strength of the shale, the lack of cementation or bonding between bedding planes, and slaking and slabbing. If left unsupported, the maximum overbreak will approach a wide corbelled, or stair-stepped, arch with a height of approximately equal to ½ the tunnel diameter above the crown. However, the swelling behavior of these shales generally will control the ground load on the primary supports. Ground loads on the primary supports are discussed subsequently.

Fort Scott Formation Limestone. The Fort Scott Formation limestone will behave as hard, stratified rock. The RQD of the limestone along the tunnel profiles is less than 60% and often less than 40%. This suggests that significant overbreak will occur in the limestone if the rock mass is left unsupported. Overbeak will vary depending on contractor means, methods, workmanship, amount and effectiveness of pre-support, and local conditions. Maximum overbreak will approach a narrow corbelled arch with a height of approximately equal to ½ the tunnel diameter above the crown. Stress-slabbing is not anticipated in the limestone.

Excello and Lagonda Formation Shale. The shales of the Excello and Lagonda Formations will behave as swelling ground. The tendency for swelling will be exacerbated by seasonal variations in precipitation and introduction of construction make-up water. Because of their higher strength, squeezing behavior is not anticipated. In the tunnel heading, the Excello and Lagonda shales will exhibit raveling behavior prior to placement of primary support.

Residual Clay. The residual clay will behave as swelling, moderately squeezing, and raveling ground. Similar to the Fort Scott shales, the residual clay behavior that is most relevant to tunnel construction will depend on the selected means and methods of construction, as well as workmanship. In the tunnel heading, the residual clay will ravel, slab, and slake prior to placement of primary support.

The relatively low strength of the residual soils and reaches of the shale may not provide a stable heading without additional support. Therefore, the Contractor should be prepared to furnish material, equipment, and labor to install spiling or another means of pre-support approved by the Engineer to provide a stable heading over these reaches. For baseline purposes, the Contractor should assume that pre-support will be required over a total length of 5% of each tunnel.

7.3.2 Excavation Methods

The size and shape of the main tunnel excavation are to be determined by the Contractor based on the selected tunneling methods and equipment, subject to the minimum requirements shown on the Contract Drawings and the clearances required for installation of the sewer pipes. The Contractor may enlarge the tunnel if necessary to suit the tunneling means and methods selected for excavation and the methods selected to install the sewer pipes. The minimum requirements shown on the Contract Drawings include an allowance along the tunnel perimeter for construction tolerances and convergence due to squeezing and swelling ground.

The tunnel can be excavated using either a tunnel boring machine (TBM) or drill-and-blast techniques. The Specifications preclude methods such as road headers or shield diggers for tunnel mining, primarily because of the difficulty of excavating very strong limestone and chert beds (with maximum unconfined compressive strengths of over 26,000 psi).

Tunnel Boring Machine. If the contractor elects to use a TBM to excavate the tunnels, the machine could be launched from starter tunnels excavated by hand-mining or blasting, or reaction for the launch could be developed from frames constructed within the portal/work shaft excavations. Tunneling could proceed uphill or downhill, although tunneling uphill would facilitate gravity drainage.

The TBM selected for this Work should meet the following minimum requirements.

- Considering the weak and potentially unstable ground conditions expected along the tunnel alignment, the TBM is required to be fully shielded. Either single- or double-shielded machines may be selected.
- The shales and residual clay are too weak to provide suitable reaction for TBM side grippers. Therefore, a thrust propulsion system that advances the machine by pushing off the tunnel primary supports is required.
- The tunnel excavations will encounter strong limestone and very weak shale (as shown in Figures 5-1 and 5-2). Therefore, the TBM cutting head should accommodate both disc cutters and drag teeth (or picks/spades) to remove the limestone and shale, respectively. Furthermore, the TBM must withstand extreme hydraulic pulse loadings resulting from the variability in hardness and toughness between the limestone and shale.
- Back-loaded cutting tools are recommended for safety of personnel working near the tunnel face.
- To avoid clogging of the muck removal system, the TBM should be equipped with an injection system that can inject water, foam, or another lubricant at the face.
- The TBM shield is required to be capable of withstanding ground pressures equivalent to 30 psi acting on the shield as a uniform pressure. The propulsion system must be able to advance the TBM with this ground pressure acting on the shield.
- In addition to being able to handle the estimated ground pressure, the TBM is required to have other features to avoid being trapped in difficult ground. At a minimum, adjustable gage cutters are required to provide at least 2 inches of overcut beyond the outside of the shield. Additional provisions, such as a tapered or conical shield, a retractable shield, or ports to inject a lubricant along the shield are recommended.
- In order to overcome cutterhead resistance and to assist in freeing the cutterhead following downtime periods, the cutterhead is required to have either two rotational speeds or a variable speed drive system and to deliver high torque at start-up. The cutterhead also should be capable of reverse rotation (i.e., turning in either direction).

Drill-and-Blast. Drill and blast excavation offers a greater degree of flexibility compared to TBM excavation and can be adapted to a wider range of ground conditions during mining. Furthermore, drill-and-blast can be used in combination with other mining methods (clay spades, etc.) in the soft ground tunneling reaches. The primary disadvantage of drill and blast methods in the poor quality/very weak rock that will be encountered on this project is that uncontrolled blasting can result in undesirable disturbance and loosening of the rock mass, safety problems due to rock fall, and increased ground support requirements.

If drill-and-blast is selected, the following should be considered.

- The tunnel heading can be advanced as a full-face excavation.
- The length of advance should be no more than 4 feet due to the generally poor ground quality (as indicated by RMR and Q ratings) and presence of very weak clay shales and/or residual clay.
- In some reaches where residual clay is encountered, it may be necessary to install spiling or another means of pre-support to advance the tunnel. For baseline purposes, the Contractor should assume that pre-support will be required over a total length of 5% of each tunnel.
- Due to the small tunnel size, it will not be possible to muck and place primary supports concurrently. However, it is important to install primary tunnel supports close to the face as quickly as possible following each round of excavation.
- Primary support should be installed immediately following mucking and scaling of the crown.
- Controlled, smooth-wall blasting techniques will be required to minimize disturbance to the rock mass. Perimeter line-drilling (closely spaced holes) and cushion-blasting techniques are acceptable methods, subject to review by the Engineer.

7.3.3 Primary Support Requirements

Primary support for the tunnels is to be selected and designed by the Contractor in accordance with minimum design criteria and requirements indicated in the Specifications, and as discussed below. Primary support is required along the entire tunnel length of both the Cowmire and Coldwater Stormwater Tunnels and should be compatible with the anticipated ground conditions as described previously, and with the Contractor's selected means and methods of tunnel excavation.

Initial Ground Loads. The Cowmire Stormwater Tunnel will be excavated at depths below ground surface that range from about 25 to 100 feet and averages about 60 feet. The Coldwater Stormwater Tunnel will be excavated at depths that range from 5 to 75 feet and averages about 60 feet. As discussed previously, swelling and squeezing ground loads will control the design of primary support. Based on the empirical methods described in Section 6.1, the primary tunnel support should be designed for a ground load of 30 psi. Lagging, if used, should be designed for a ground load of 15 psi.

In addition to the recommended ground loads, construction loads must also be addressed in the design of the support systems. For example, if a TBM is selected, these loads will include the jacking loads induced by a TBM thrusting against the primary support.

Primary Support. Steel ribs and wood lagging or precast concrete segments may be selected to provide primary support if a TBM is used. The supports should be prepared under the TBM tail shield and installed or expanded immediately following passage of the TBM. The Contractor should not rely on complete expansion of primary support to obtain the required clearances shown on the Drawings. Various sizes of spacers and shims will be needed to obtain maximum possible expansion. If drill-and-blast is used, primary support should consist of steel sets with invert struts. Wood lagging, steel channels, or reinforced shotcrete may be used to span the steel sets.

Pre-support, such as spiling, may be required to provide adequate heading stability over limited reaches of the tunnels. As mentioned previously, for baseline purposes, the Contractor should assume that pre-support will be required over a total length of 5% of each tunnel.

Construction tolerances and support deformations also are important in the design of the primary support system. Primary supports that fail structurally or deform and encroach into the tunnel to the extent that the final lining cannot be constructed to the specified tolerances will have to be removed, the ground re-mined (if necessary), and the supports replaced.

The Contractor is responsible to ensure that the primary lining is in intimate contact with the ground. In some areas, this may require the installation of cribbing or backpacking. Contact grouting between the rock and the primary support should occur following installation of the final lining. Grouting will ensure uniform contact between the rock and the supports and distribute any non-uniform or point ground loads on the final lining system.

7.3.4 Groundwater

Groundwater inflows generally are expected to be small as a result of the low permeability of the overburden soils and the low permeability of the intact rock. Most inflow likely will occur along open joints and bedding planes, along coal or smut seams, and at limestone/shale interfaces. For baseline purposes, localized peak inflow (flush flow) of 30 gpm at the heading should be assumed. For this project, the heading inflows are defined as inflows entering the tunnel within 100 feet of the tunnel face. Also for baseline purposes, sustained groundwater inflow of up to ¾ gpm per linear foot of tunnel should be assumed. In order to maintain safe working conditions in the tunnel, and to prevent softening and instability from developing in the invert in the shales, the Specifications preclude water from accumulating anywhere in the tunnel excavations.

7.3.5 Hazardous Gases

The potential for encountering methane gas or hydrogen sulfide gas is an important construction consideration and safety issue. Thin beds of smutty coal were encountered during the investigations. The drilling crews did not observe evidence of gas in the coal seems (e.g., air bubbles in water used for coring) during the drilling operations. The probability of encountering methane or hydrogen sulfide gas in the tunnel appears to be low; however, the potential can not be ruled out. The occurrence of potentially hazardous gases in the tunnel should be monitored during construction in accordance with OSHA requirements. The mere presence of coal in the borings will require the contractor to monitor for methane, hydrogen sulfide, and carbon monoxide during tunnel and shaft construction. Because the potential to encounter hazardous gases is present; any equipment used in the tunnel, e.g., the TBM, should be configured for “gassy” ground conditions.

7.4 OTHER CONSIDERATIONS

7.4.1 Water Disposal

Construction water will result from groundwater inflows into the tunnel, surface water and groundwater inflows into the east and west portal excavations, and construction make-water. This water must be handled, treated, and disposed of in accordance with the Specifications.

It is likely that contaminants will be introduced into construction water due to construction activities. These contaminants may include suspended and colloidal soil particles, hydraulic fluid, oil, grease and diesel fuel. In addition, the use of cement-based products and various chemical additives during shotcreting and concrete backfill placement operations will alter the pH of the water.

The contractor is responsible for treating and disposing of all water pumped from the excavation. As a minimum, Baker tanks or settlement ponds and oil skimmers will be required to remove sediment and oil and grease from the water, respectively. The facilities should be sized also to handle the estimated groundwater inflow volume from the tunnels and portal excavations discussed above. After treatment to remove sediment, oil, and grease, water can be pumped into a storage tank and used for dust control. During the winter months when dust control is not required, the water must be disposed of off site in accordance with the Specifications.

7.4.2 Muck Disposal

Excavations for the cut and cover segments, portals, and tunnels will generate excess earth and rock spoil materials (muck) that need to be properly disposed. Excavated spoils not re-used as backfill as part of this Work will be disposed of in the grading operations for other parts of the Airport Expansion Project. These materials are to be loaded and hauled to stockpile locations to be determined by the Owner’s Representative.

7.4.3 Monitoring Requirements

Recommended instrumentation for the tunnels, portals, and shafts were described in Section 6. As indicated in Section 6, the Contractor will monitor tunnel convergence instruments and the Owner’s Representative will monitor slope inclinometers. Remedial action will be at the Contractor’s expense, to be proposed by the Contractor and accepted by the Engineer.

The Contractor will measure water flow at time intervals indicated in the Contract Specifications to document the volume of water leaving the tunnels and excavations. In addition, the Contractor is required to constantly monitor air quality in the tunnel and shafts during this work.

The interpretations and assessments contained in this report are based upon the available information on the site, on limited surface and subsurface exploration data, and our experience in the St. Louis area. As such, the interpretations and assessments are based upon professional judgment and experience. Anticipated conditions were estimated with the standard of care commonly applied as state of the practice in the profession. No warranty is included, either expressed or implied, that the actual conditions encountered will conform with the anticipated conditions described herein.

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TABLES

Table 5-1. Summary of Index and Engineering Properties of Soil Overburden Deposits

	Water Content (%)	Liquid Limit (%)	Plasticity Index (%)	Total Unit Weight (pcf)	Undrained Strengths s_u (psf)
FILL					
No. of Tests	26	4	4	1	0
Mean	20	35	13	113	--
Median	19	33	10	--	--
Std. Dev.	5	8	10	--	--
Maximum	29	46	26	--	--
Minimum	10	28	5	--	--
LOESS					
No. of Tests	124	22	22	16	16
Mean	25	39	18	119	720
Median	25	39	17	119	685
Std. Dev.	5	6	7	2	300
Maximum	52	55	33	122	1280
Minimum	9	31	8	112	250
RESIDUAL CLAY					
No. of Tests	73	14	14	0	3
Mean	27	48	28	--	2320
Median	25	53	28	--	--
Std. Dev.	7	13	14	--	--
Maximum	54	62	45	--	3900
Minimum	9	22	1	--	1090

Note: The Alluvium is not included in this table because only two sets of water content and limits were obtained in these soils. No unit weight or strength tests were conducted on samples of Alluvium. Available test results are presented in the GDR and discussed in the text.

Table 5-2. Summary of Slake Durability Indices of Shale Samples

Formation/Member	Measured Slake Durability Indices	Average	Remarks[1]
Fort Scott/Little Osage (upper bed)	0.34	6.2	Very low durability
Fort Scott/Little Osage (lower bed)	4.24, 0.82, 8.88, 3.13, 20.84, 5.42		
Excello and Lagonda	88.84, 16.67	52.8	Low durability

[1]Based on Gamble's (1971) Slake Durability Classification

Table 5-3. Summary of Bedrock Properties

	Water Content (%)	Liquid Limit (%)	Plasticity Index (%)	Total Unit Weight (pcf)	Compressive Strength (psi)	Point Load Index (psi)	Free Swell[1,2] (%)	Swell Pressure[1,2] (psi)	Slake Durability Index	Cerchar Abrasivity Index
FORT SCOTT FORMATION SHALES										
No. of Tests	172	58	58	26	14	0	14	14	7	0
Mean	18	47	25	132	40	--	5.0	52	6.2	--
Median	18	48	25	134	28	--	4.1	23	4.2	--
Std. Dev.	6	8	7	10	36	--	4.0	58	7.1	--
Maximum	50	66	43	145	115	--	15.0	167	20.8	--
Minimum	5	31	12	98	11	--	0.4	2	0.3	--
BASELINE	**--**	**--**	**--**	**--**	**28**	**--**	**--**	**--**	**--**	**--**
FORT SCOTT FORMATION LIMESTONES										
No. of Tests	--	--	--	11	13	13	0	0	0	8
Mean	--	--	--	165	11337	497	--	--	--	1.9
Median	--	--	--	166	9984	463	--	--	--	1.8
Std. Dev.	--	--	--	5	6016	221	--	--	--	0.3
Maximum	--	--	--	168	26616	945	--	--	--	2.4
Minimum	--	--	--	150	4534	169	--	--	--	1.6
BASELINE	**--**	**--**	**--**	**--**	**10000**	**475**	**--**	**--**	**--**	**1.9**
EXCELLO AND LAGONDA FORMATION SHALES										
No. of Tests	18	8	8	5	4	0	0	0	2	0
Mean	6	32	15	155	1064	--	--	--	52.8	--
Median	5	31	12	157	1084	--	--	--	--	--
Std. Dev.	2	6	7	4	445	--	--	--	--	--
Maximum	12	44	28	158	1570	--	--	--	88.8	--
Minimum	4	26	8	148	516	--	--	--	16.7	--
BASELINE	**--**	**--**	**--**	**--**	**1070**	**--**	**--**	**--**	**--**	**--**

[1]Free swell and swell pressure are described in the text.

[2]Statistical values for free swell and swell pressure include tests on shale samples retrieved from boreholes outside Cowmire and Coldwater Stormwater Tunnel corridors. These additional data were presented in the GDR.

Table 6-1. Definitions of Tunnel Ground Condition Behavior Terms and Corresponding Ground Loads

(After Terzaghi, in Proctor and White, 1946; Deere et al. 1970; and Heuer, 1974)

Ground Classification	Description	Final Rock Load: Empirical (in ft of rock)[a]	Final Rock Load: Estimated[c] (psi)	Remarks on Load
Raveling Ground	Raveling ground is a material that gradually breaks up into chunks, blocks, slabs, or angular fragments. This process is time-dependent and materials may be classified by the rate of disintegration as fast or slow raveling. In fast raveling ground, the process starts within a few minutes, otherwise the ground is slow raveling. For a material to be raveling it must be moderately coherent and frable, or discontinuous. Examples of raveling ground include fine moist sand, sands and gravels with clay binder, stiff fissured clays, weak and friable rocks, and jointed rocks. It does not include material considered to be swelling or squeezing.	0.25B to 1.1C[b]	2.5 to 22	Little or no side pressure.
Swelling Ground	Swelling ground is a material that absorbs water, increases in volume, and expands slowly into the tunnel. This may occur in high plasticity overconsolidated clays, claystones, clay shales, and shales that exhibit large volume change characteristics upon wetting.	up to 250 ft	up to 250	Very heavy side pressure. Use circular support. In extreme cases, use yielding support.
Squeezing Ground	Squeezing ground is a material that advances slowing into a tunnel without any perceptible volume change or visible signs of fracturing. A prerequisite for squeezing is overstress of the material close to the tunnel opening, which is governed by the material strength in comparison to the overburden stress. Squeezing ground may consist of soft to medium stiff clays, and weak rocks (claystones, clay shales, and shales).	1.0C to 2.1C	20 to 42	Heavy side pressure, continuous support required.
Hard, Stratified Rock	Hard, stratified rock consists of individual strata with little or no resistance to separation along the strata boundaries. The strata may or may not be weakened by transverse joints. In such rock, slabbing and spalling conditions are common.	0.25B	2.5	Spalling common.

[a]One foot of rock is approximately equal to 1 psi of pressure

[b]B equals tunnel width; C = tunnel width plus tunnel height

[c]Load estimated using tunnel width and height of 10 feet

Table 6-2. Estimated Modified Overload Factors for Average and Maximum Tunnel Crown Covers

Formation and Rock Type	Unconfined Compressive Strength (UCS) (psi)			Cowmire Tunnel Estimated Overload Factor[1] (for Median UCS)		Coldwater Tunnel Estimated Overload Factor[1] (for Median UCS)	
	Low Value	High Value	Median Value	Average Cover (60 ft)	Maximum Cover (100 ft)	Average Cover (60 ft)	Maximum Cover (75 ft)
Fort Scott shales	11	115	28	4.8	8.0	4.8	6.0
Fort Scott limestones	4,534	26,616	9,984	0.01	0.02	0.01	0.02
Excello and Lagonda shales	516	1,570	1,084	0.1	0.2	0.1	0.15

[1]Overload factors calculated using median compressive strength, elastic stress distribution, and average total unit weight of 140 pcf for soil and rock.

Table 6-3. Estimated Shrinkage for Various Material Types[1]

Material Type	Estimated Shrinkage (%)
Loess	11
Residual Clay	9
Shale	-3

[1]Recommendations from URS' *Final Geotechnical and Foundation Engineering Report for Runway and Taxiways,* dated Feb. 2002.

Table 6-4. Earth Pressures and Allowable Bearing Capacities for Permanent Retaining Structures[1,2]

Geologic Unit	Active Earth Pressure			At-Rest Earth Pressure			Allowable Passive Resistance[3]			Allowable Bearing Capacity[4] (psf)
	K_A	Drained (pcf)	Saturated (pcf)	K_o	Drained (pcf)	Saturated (pcf)	K_P	Drained (pcf)	Saturated (pcf)	
Loess	0.5	60	95	0.67	80	105	2.5	200	110	1500
Residual Clay	0.5	60	95	0.67	80	105	2.5	200	110	3000
Alluvium	0.6	70	100	0.7	85	110	2.2	175	90	1500
Shale	0.6	70	100	1.1	145	140	2.2	175	90	3000
Free draining granular fill, if used	0.3	35	80	0.46	55	90	4.0	300[5]	250[5]	n/a

(1) Earth pressures in table are reported as equivalent fluid pressures and assumes level backfill as indicated in sketch below.

(2) Does not include dynamic loads such as earthquake shaking.

(3) Includes a factor of safety of 1.5.

(4) Includes a factor of safety of 3.

(5) Assumes the average width of the backfill is 1.5 times the height of the cut.

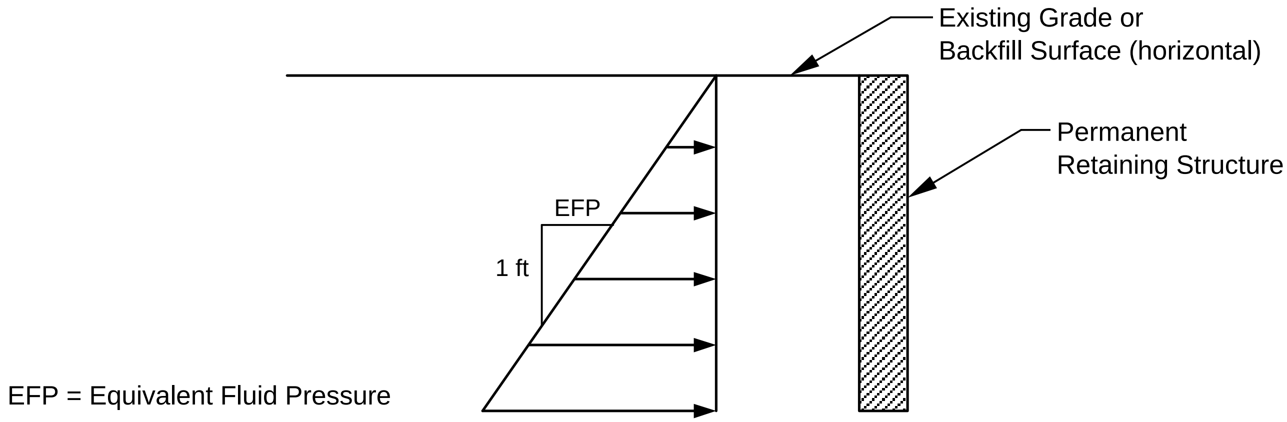


Table 6-5. Monitoring Schedule for Tunnel Convergence Instrumentation

Convergence (% of Tunnel Diameter, D)	Frequency of Measurement	Contractor Action
0 to 0.5%	weekly	None
0.5 to 1%	bi-weekly	Engineer to notify Contractor of possible need for corrective action
> 1%	bi-weekly	Contractor to evaluate cause of convergence and submit corrective action plan to Owner

Table 7-1. Volumetric Percentages of Various Materials Encountered in Excavations[1]

Tunnel Alignment	Struture	Volumetric Percentage of Material Type: Loess	Residual Clay	Fort Scott Shale	Excello or Lagonda Shale	Fort Scott Limestone
Cowmire	East Portal/Drop Shaft	10	25	30	--	35
	West Portal[2]	--	--	--	--	--
	Sta. 11+92 Shaft	55	15	10	--	20
	Sta. 21+92 Shaft	30	20	40	--	10
	Trunk Sewer Tunnel	--	--	35	5	60
Coldwater	West Portal/Drop Shaft	--	65	20	--	15
	East Portal[2]	--	--	--	--	--
	Sta. 10+00 Shaft	45	25	30	--	--
	Trunk Sewer Tunnel	--	10	75	--	15

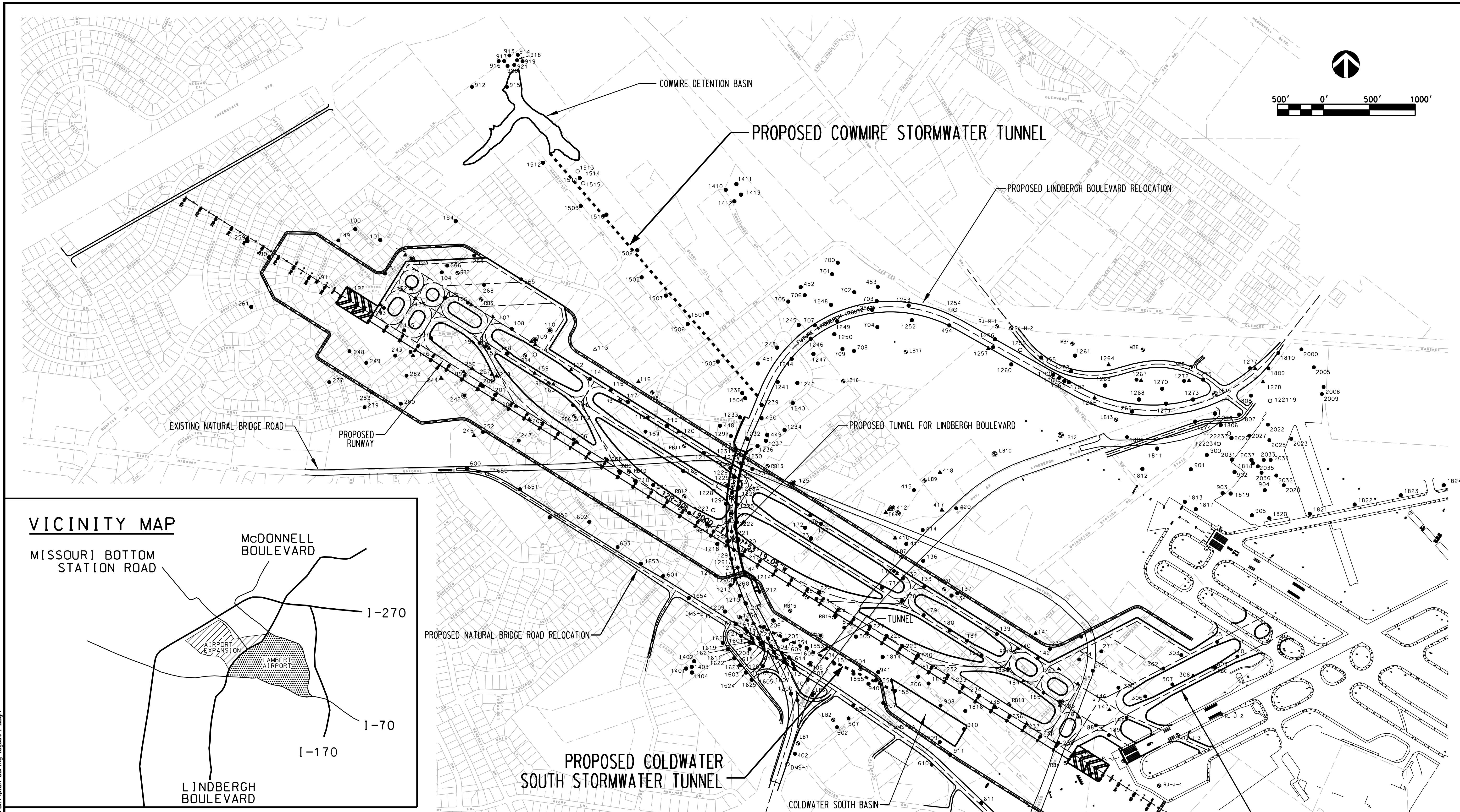
[1] Volumetric percentages were determined as follows. The subsurface stratigraphy shown in Figures 5-1 and 5-2 were developed based on boring log results. The shaded areas represent the longitudinal cross-sections of the various structures along the tunnel alignment centerlines. Two-dimensional (2-D) areas of each geologic unit encountered along a given structure were measured using a planimeter. The volumetric percentage was calculated as the 2-D geologic unit area divided by the total 2-D area of the structure. No correction for the shapes of the excavations were made in part because the excavation shapes will be determined by the Contractor. The volumetric percentages were rounded to the nearest 5 percent.

[2] Material percentages encountered in the West and East Portal excavations for the Cowmire and Coldwater Tunnels, respectively, will depend on the Contractor's selected means and methods for excavation and/or ground retention and the locations that the Contractor selects to establish the portals.

Table 7-2. Allowable Bond Stresses for Design of Rock Anchors and Soil Nails

Material Type	Allowable Bond Stress[1] (psi)
Loess and Residual Clay	5
Shale	10
Limestone	120

[1]Values assume that tieback, anchor, bolt, or nail is pressure grouted (FHWA 1999).



LEGEND

- PRELIMINARY ENGINEERING BORINGS 1998
- BORINGS FROM 1999 TO 2001 INVESTIGATIONS
- CPT SOUNDINGS FROM 1999-2000 INVESTIGATIONS

NO.	DATE	REVISION DESCRIPTION	APPROVED

PREPARED BY:

URS

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SEAL

DATE:	12/7/01
SCALE:	AS SHOWN
DESIGNED:	DJD
DRAWN:	DJD
CHECKED:	
APPROVED:	
SUBMITTED:	URS/TSI

LAMBERT - ST. LOUIS INTERNATIONAL AIRPORT

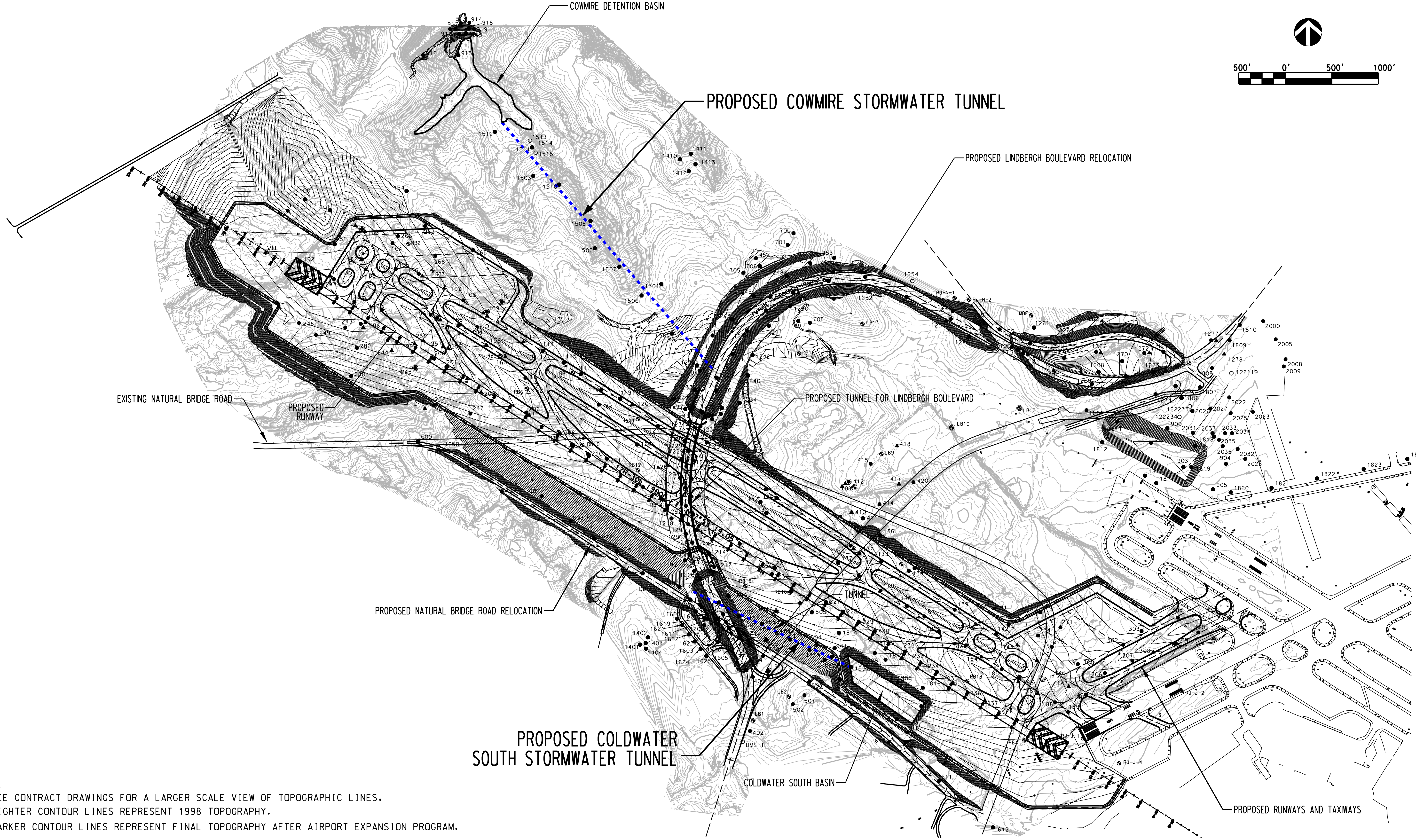
THE SPK TEAM
A SVERDRUP, PARSONS, KWAME JOINT VENTURE PROGRAM MANAGERS

AIRPORT EXPANSION PROGRAM

AIRPORT EXPANSION PLAN AND BORING LAYOUT

WBS NO.

SHEET NO. 1-1



NOTES:

1.) SEE CONTRACT DRAWINGS FOR A LARGER SCALE VIEW OF TOPOGRAPHIC LINES.
2.) LIGHTER CONTOUR LINES REPRESENT 1998 TOPOGRAPHY.
3.) DARKER CONTOUR LINES REPRESENT FINAL TOPOGRAPHY AFTER AIRPORT EXPANSION PROGRAM.

NO.	DATE	REVISION DESCRIPTION	APPROVED

PREPARED BY:

URS

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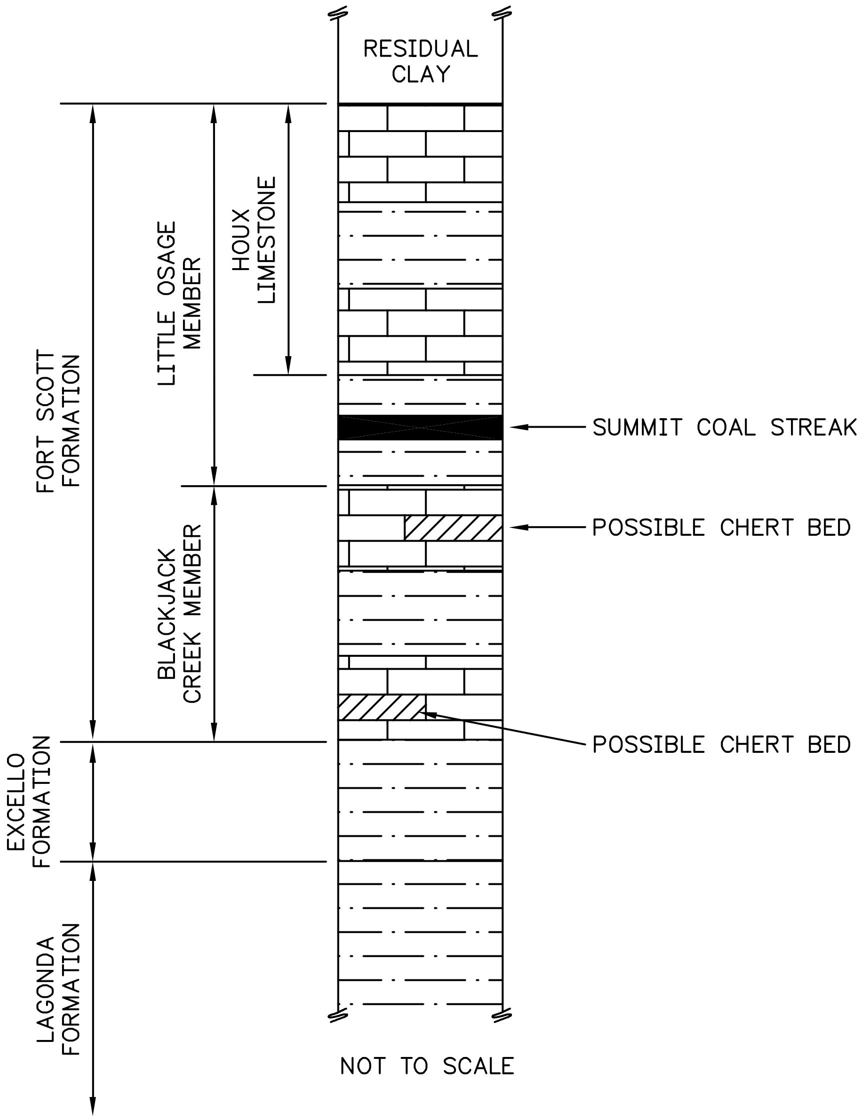
SURFACE TOPOGRAPHY AND PROPOSED TUNNEL ALIGNMENTS

WBS NO.

SHEET NO. 3-1



KEY

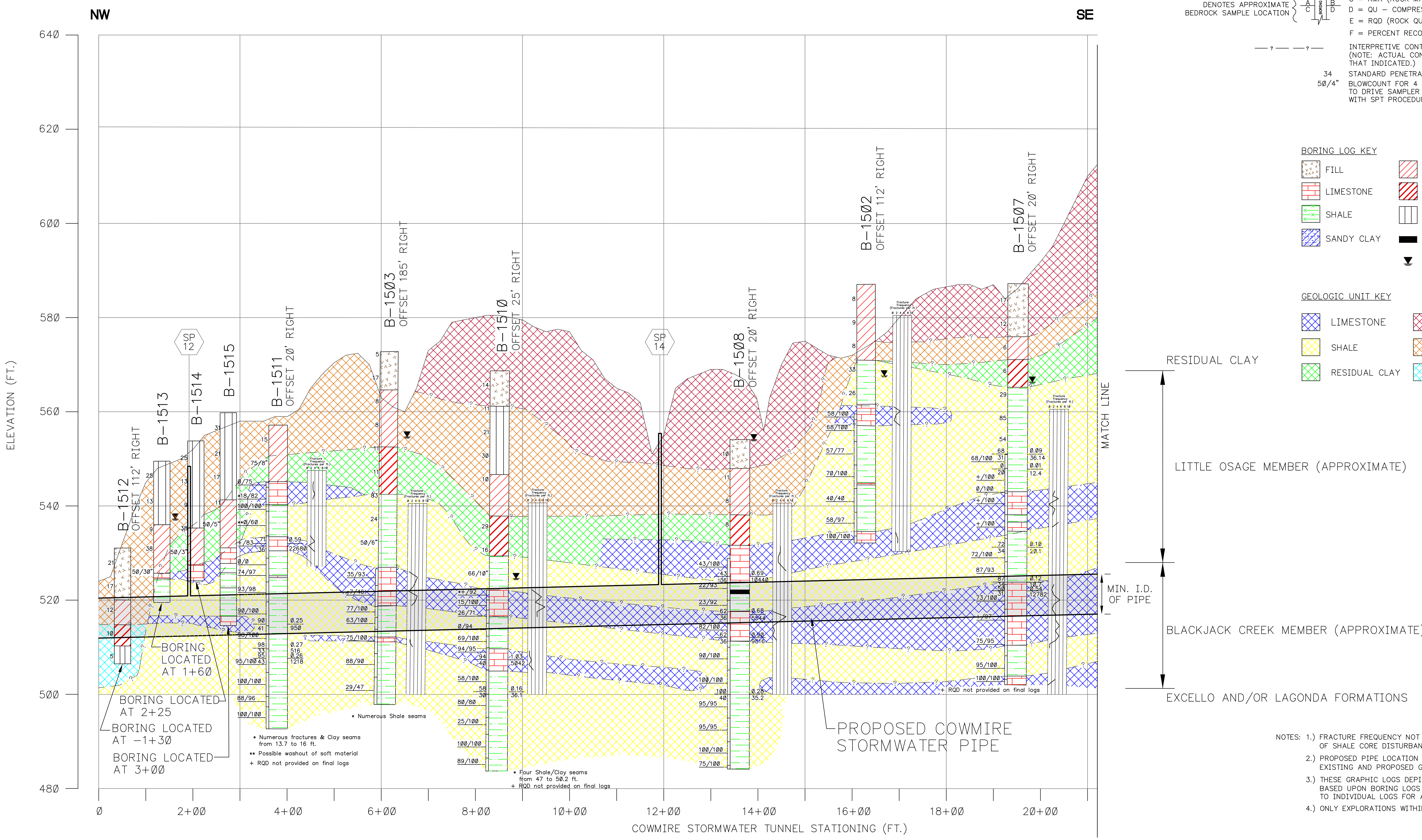
LIMESTONE

CHERT

COAL (SMUT)

SHALE

LAMBERT–ST. LOUIS INTERNATIONAL AIRPORT EXPANSION ST. LOUIS MISSOURI	PROJECT NO. 2399000042.08
URS	
DRN. BY:DS 5/16/02 DSGN. BY:SO CHKD. BY:	Simplified Stratigraphic Column
FIG. NO. 3–2	



LEGEND

DENOTES CORE RUN LOCATION — E/F

DENOTES APPROXIMATE BEDROCK SAMPLE LOCATION — A/C, B/D

A = RQD (ROCK QUALITY DESIGNATION) %
B = Q-SYSTEM VALUE
C = RMR (ROCK MASS RATING) SYSTEM VALUE
D = QU – COMPRESSIVE STRENGTH IN PSI
E = RQD (ROCK QUALITY DESIGNATION) %
F = PERCENT RECOVERY FROM CORE RUN

— ? — — ? — INTERPRETIVE CONTACT BETWEEN GEOLOGIC UNITS. (NOTE: ACTUAL CONTACT MAY DIFFER FROM THAT INDICATED.)

34 STANDARD PENETRATION TEST BLOWCOUNT (N-VALUE)

50/4" BLOWCOUNT FOR 4 INCHES OF PENETRATION; UNABLE TO DRIVE SAMPLER FULL 18 INCHES FOR COMPLIANCE WITH SPT PROCEDURES

BORING LOG KEY

- FILL
- LIMESTONE
- SHALE
- SANDY CLAY
- CL; LOW PLASTIC CLAY
- CH; HIGH PLASTIC CLAY
- ML; LOW PLASTIC SILT
- COAL OR SMUT SEAM
- DELAYED GROUNDWATER READING

GEOLOGIC UNIT KEY

- LIMESTONE
- SHALE
- RESIDUAL CLAY
- FILL
- LOESS
- ALLUVIUM

NOTES:
1.) FRACTURE FREQUENCY NOT CALCULATED FOR SHALE BECAUSE OF SHALE CORE DISTURBANCE (DRYING).
2.) PROPOSED PIPE LOCATION WAS PROVIDED BY PBTS. EXISTING AND PROPOSED GRADES WERE ALSO PROVIDED BY PBTS.
3.) THESE GRAPHIC LOGS DEPICT GENERALIZED SOIL CONDITIONS BASED UPON BORING LOGS FURNISHED BY TERRACON. REFER TO INDIVIDUAL LOGS FOR ADDITIONAL DETAILS AND DESCRIPTIONS.
4.) ONLY EXPLORATIONS WITHIN 200 FT OF CENTERLINE ARE SHOWN.

VERTICAL SCALE: 1"=10' (FT.) 10 0 10 20

HORIZONTAL SCALE: 1"=100' (FT.) 100 0 100 200

NO.	DATE	REVISION DESCRIPTION

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DRAWN: DS
CHECKED:
PROJECT NO.: 2399000042.08
SUBMITTED: TSI/URS 5/10/02

LAMBERT – ST. LOUIS INTERNATIONAL AIRPORT

THE SPK TEAM
A Sverdrup, Parsons, Kwame Joint Venture
PROGRAM MANAGERS

AIRPORT EXPANSION PROGRAM

GEOLOGIC PROFILE ALONG COWMIRE STORMWATER TUNNEL

WBS NO.

SHEET NO.
5-1a

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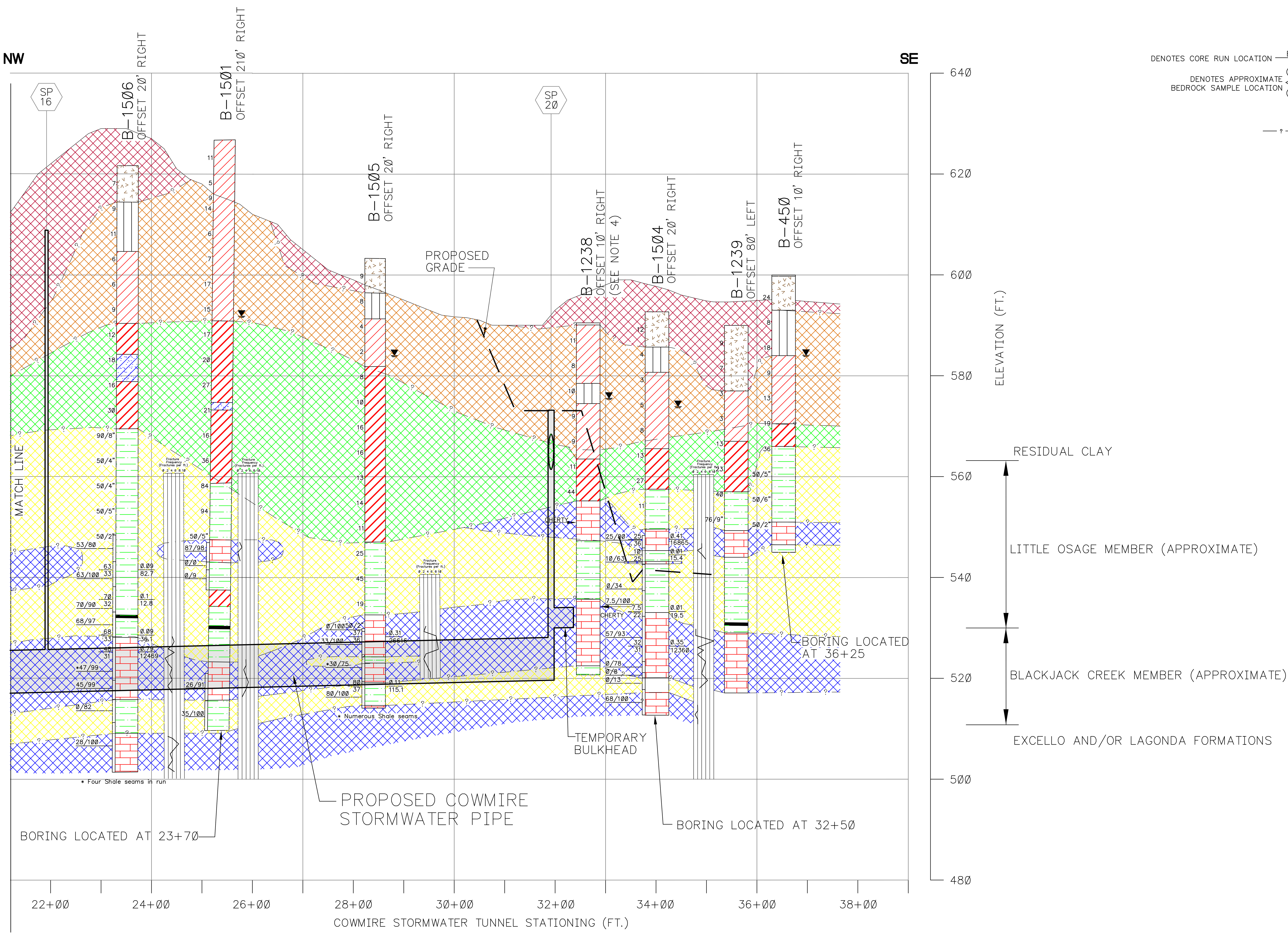

LEGEND

DENOTES CORE RUN LOCATION — E/F

DENOTES APPROXIMATE BEDROCK SAMPLE LOCATION — A/C, B/D (BORING)

A = RQD (ROCK QUALITY DESIGNATION) %
B = Q-SYSTEM VALUE
C = RMR (ROCK MASS RATING) SYSTEM VALUE
D = QU - COMPRESSIVE STRENGTH IN PSI
E = RQD (ROCK QUALITY DESIGNATION) %
F = PERCENT RECOVERY FROM CORE RUN

— ? — — ? — INTERPRETIVE CONTACT BETWEEN GEOLOGIC UNITS. (NOTE: ACTUAL CONTACT MAY DIFFER FROM THAT INDICATED.)

34 STANDARD PENETRATION TEST BLOWCOUNT (N-VALUE)

50/4" BLOWCOUNT FOR 4 INCHES OF PENETRATION; UNABLE TO DRIVE SAMPLER FULL 18 INCHES FOR COMPLIANCE WITH SPT PROCEDURES

BORING LOG KEY

- FILL
- LIMESTONE
- SHALE
- SANDY CLAY
- CL; LOW PLASTIC CLAY
- CH; HIGH PLASTIC CLAY
- ML; LOW PLASTIC SILT
- COAL OR SMUT SEAM
- DELAYED GROUNDWATER READING

GEOLOGIC UNIT KEY

- LIMESTONE
- SHALE
- RESIDUAL CLAY
- FILL
- LOESS
- ALLUVIUM

NOTES:
1.) FRACTURE FREQUENCY NOT CALCULATED FOR SHALE BECAUSE OF SHALE CORE DISTURBANCE (DRYING).
2.) PROPOSED PIPE LOCATION WAS PROVIDED BY PBTS. EXISTING AND PROPOSED GRADES WERE ALSO PROVIDED BY PBTS.
3.) THESE GRAPHIC LOGS DEPICT GENERALIZED SOIL CONDITIONS BASED UPON BORING LOGS FURNISHED BY TERRACON. REFER TO INDIVIDUAL LOGS FOR ADDITIONAL DETAILS AND DESCRIPTIONS.
4.) BORING B-1238 SHIFTED VERTICALLY DOWN 9FT. TO BE CONSISTENT WITH SURFACE CONTOURS BASED ON TOPOGRAPHIC PLAN DATED NOVEMBER 15, 2001 PREPARED BY PBTS.
5.) ONLY EXPLORATIONS WITHIN 200 FT OF CENTERLINE ARE SHOWN.

VERTICAL SCALE: 1"=10' (FT.) 10 0 10 20

HORIZONTAL SCALE: 1"=100' (FT.) 100 0 100 200

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DRAWN: DS
CHECKED:
PROJECT NO.: 2399000042.08
SUBMITTED: TSI/URS 5/10/02

LAMBERT - ST. LOUIS INTERNATIONAL AIRPORT

THE SPK TEAM
A Sverdrup, Parsons, Kwame Joint Venture
PROGRAM MANAGERS

AIRPORT EXPANSION PROGRAM

GEOLOGIC PROFILE ALONG COWMIRE STORMWATER TUNNEL

WBS NO.

SHEET NO. 5-1b

LEGEND

DENOTES CORE RUN LOCATION — E/F

DENOTES APPROXIMATE BEDROCK SAMPLE LOCATION — A/C, B/D

A = RQD (ROCK QUALITY DESIGNATION) %
B = Q-SYSTEM VALUE
C = RMR (ROCK MASS RATING) SYSTEM VALUE
D = QU - COMPRESSIVE STRENGTH IN PSI
E = RQD (ROCK QUALITY DESIGNATION) %
F = PERCENT RECOVERY FROM CORE RUN

DELAYED WATER LEVEL AFTER DRILLING

— ? —?— INTERPRETIVE CONTACT BETWEEN GEOLOGIC UNITS (NOTE: ACTUAL CONTACT MAY DIFFER FROM THAT INDICATED.)

34 STANDARD PENETRATION TEST BLOWCOUNT (N-VALUE)

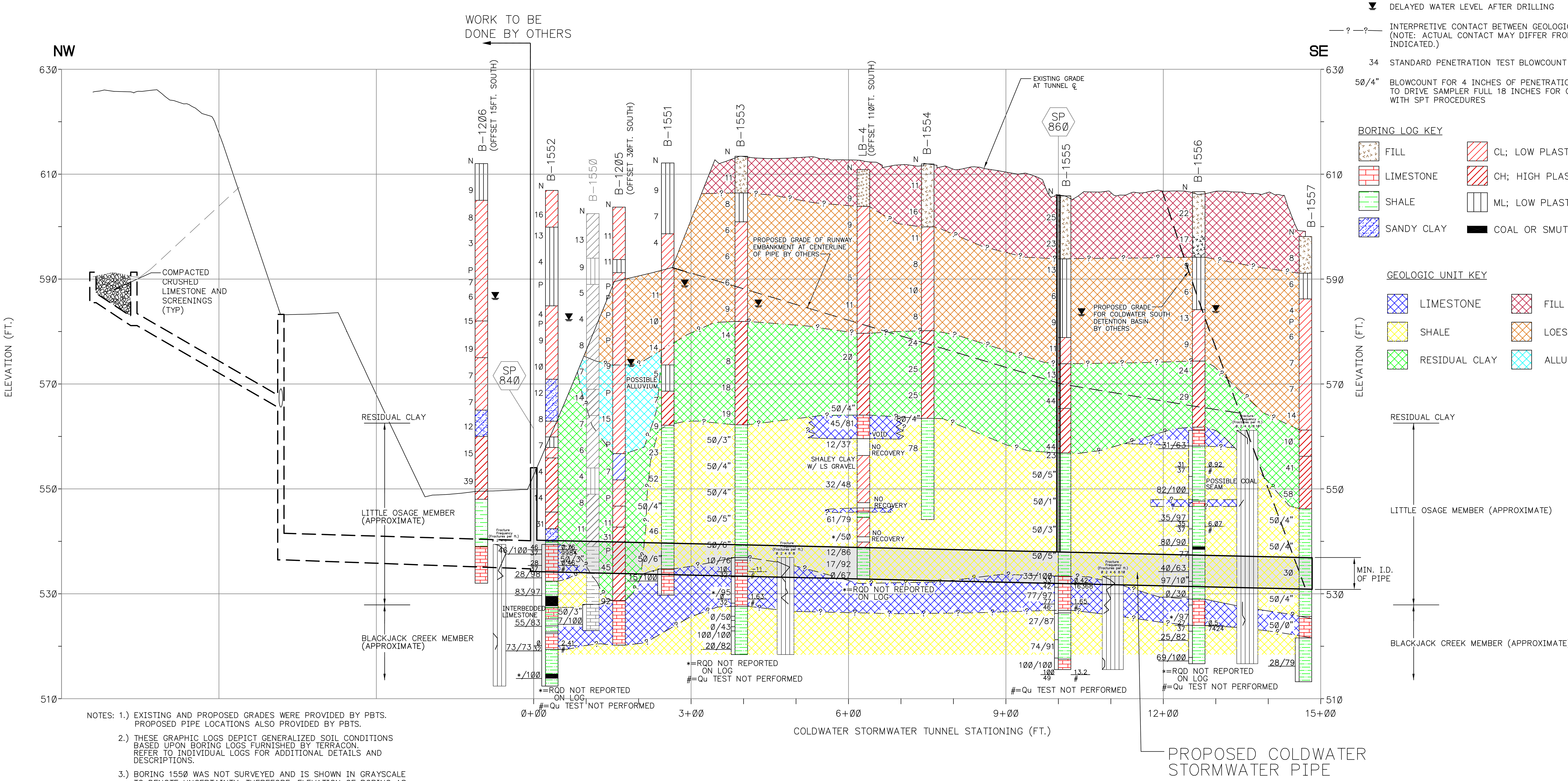
50/4" BLOWCOUNT FOR 4 INCHES OF PENETRATION; UNABLE TO DRIVE SAMPLER FULL 18 INCHES FOR COMPLIANCE WITH SPT PROCEDURES

BORING LOG KEY

FILL
LIMESTONE
SHALE
SANDY CLAY
CL; LOW PLASTIC CLAY
CH; HIGH PLASTIC CLAY
ML; LOW PLASTIC SILT
COAL OR SMUT SEAM

GEOLOGIC UNIT KEY

LIMESTONE
SHALE
RESIDUAL CLAY
FILL
LOESS
ALLUVIUM



Horizontal Scale: (FT.) 0 100 200

Vertical Scale: (FT.) 0 10 20

NOTES:
1.) EXISTING AND PROPOSED GRADES WERE PROVIDED BY PBTS. PROPOSED PIPE LOCATIONS ALSO PROVIDED BY PBTS.
2.) THESE GRAPHIC LOGS DEPICT GENERALIZED SOIL CONDITIONS BASED UPON BORING LOGS FURNISHED BY TERRACON. REFER TO INDIVIDUAL LOGS FOR ADDITIONAL DETAILS AND DESCRIPTIONS.
3.) BORING 1550 WAS NOT SURVEYED AND IS SHOWN IN GRAYSCALE TO DENOTE UNCERTAINTY. THEREFORE, ELEVATION OF BORING AS SHOWN IS APPROXIMATE BASED ON TOPOGRAPHIC PLAN DATED NOVEMBER 15, 2001 AND PREPARED BY PBTS.
4.) ONLY EXPLORATIONS WITHIN 50FT. OF CENTERLINE ARE SHOWN.
5.) FRACTURE FREQUENCY NOT CALCULATED FOR SHALE BECAUSE OF SHALE CORE DISTURBANCE (DRYING).

NO.	DATE	REVISION DESCRIPTION

PREPARED BY:

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PROJECT NO.: 23990000042.08
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THE SPK TEAM
A Sverdrup, Parsons, Kwame Joint Venture
PROGRAM MANAGERS

AIRPORT EXPANSION PROGRAM

GEOLOGIC PROFILE ALONG COLDWATER STORMWATER TUNNEL

WBS NO.

SHEET NO. 5-2

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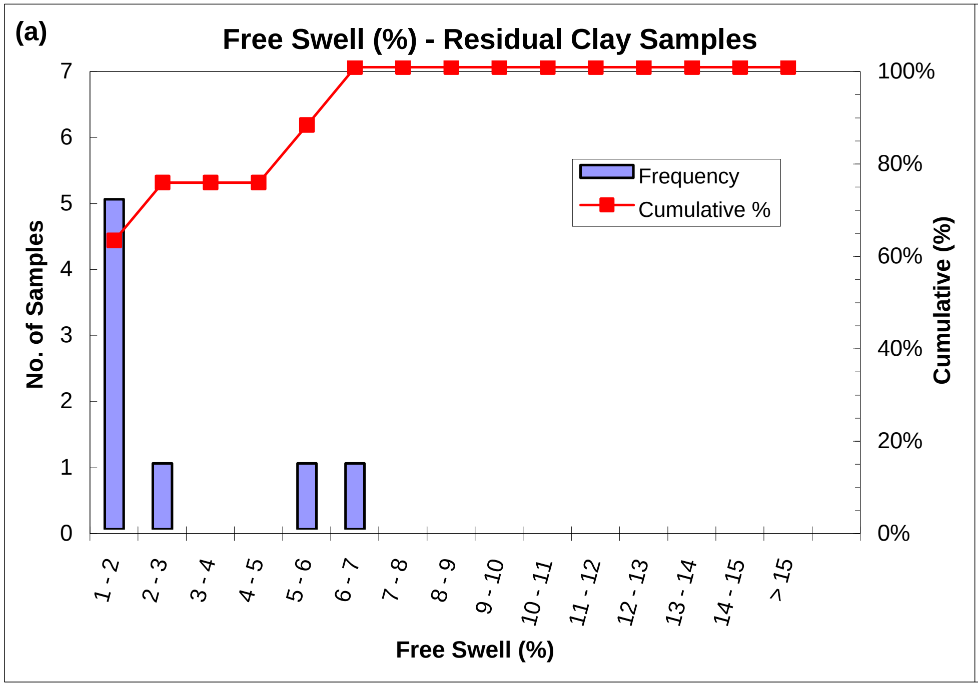

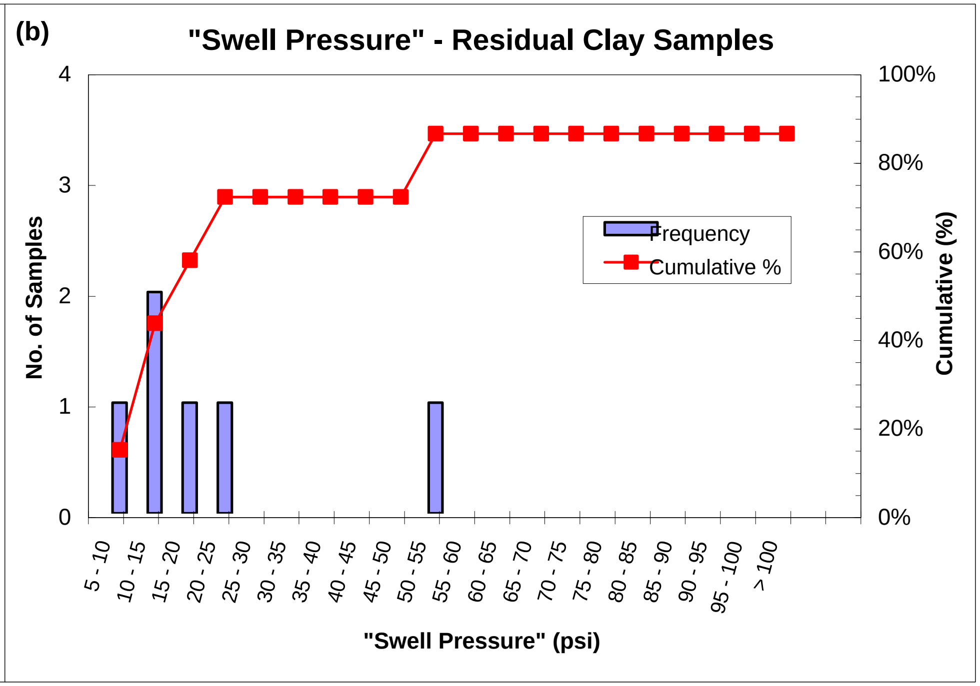

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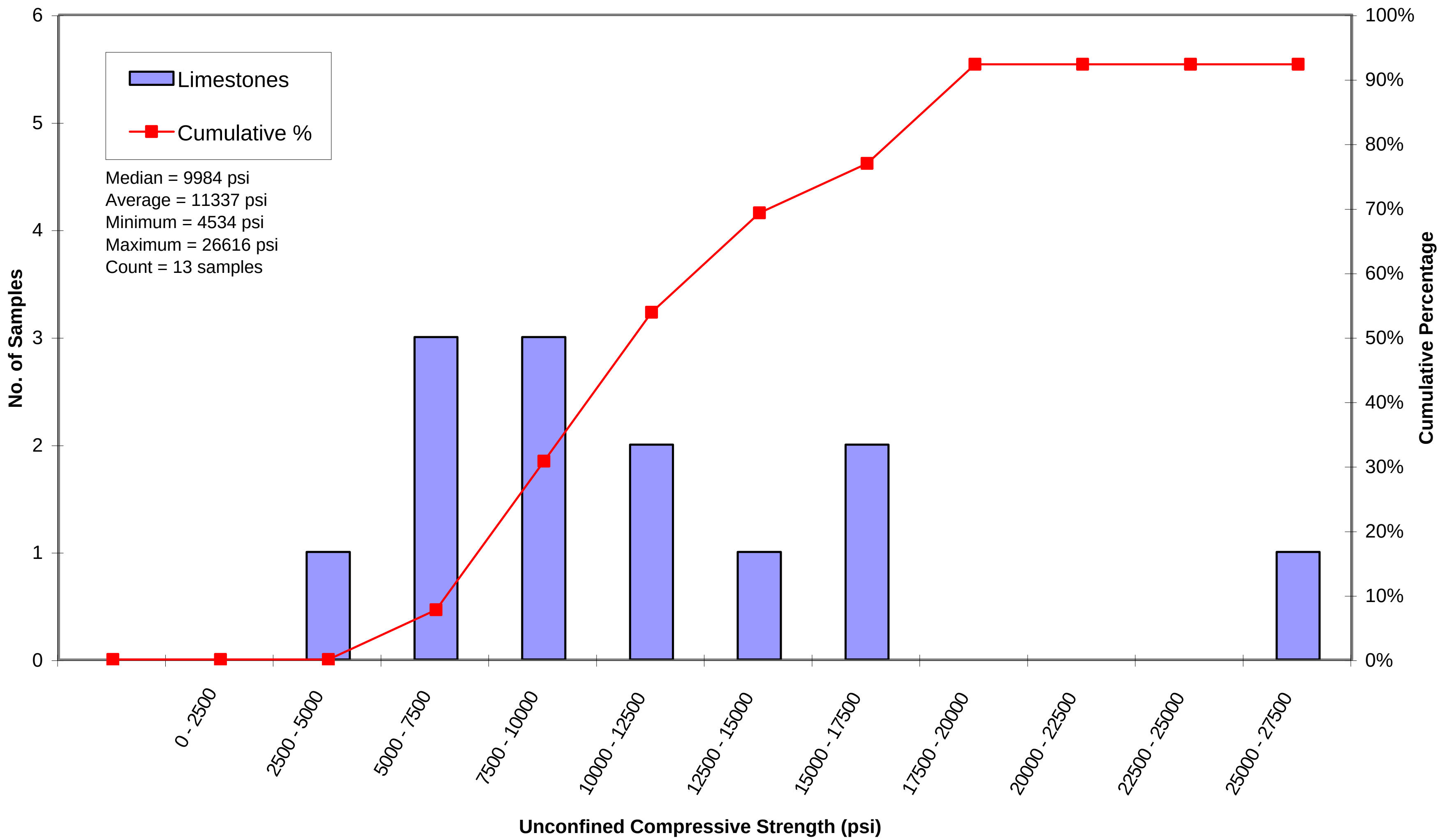
St. Louis Lambert Airport Expansion Program St. Louis, Missouri		PROJECT NO. 23-99000042.08
URS		
DRN. BY: DSGN. BY: smo CHKD. BY:	Free Swell and Swell Pressure of AEP Residual Clay Samples	FIG. NO. 5-3



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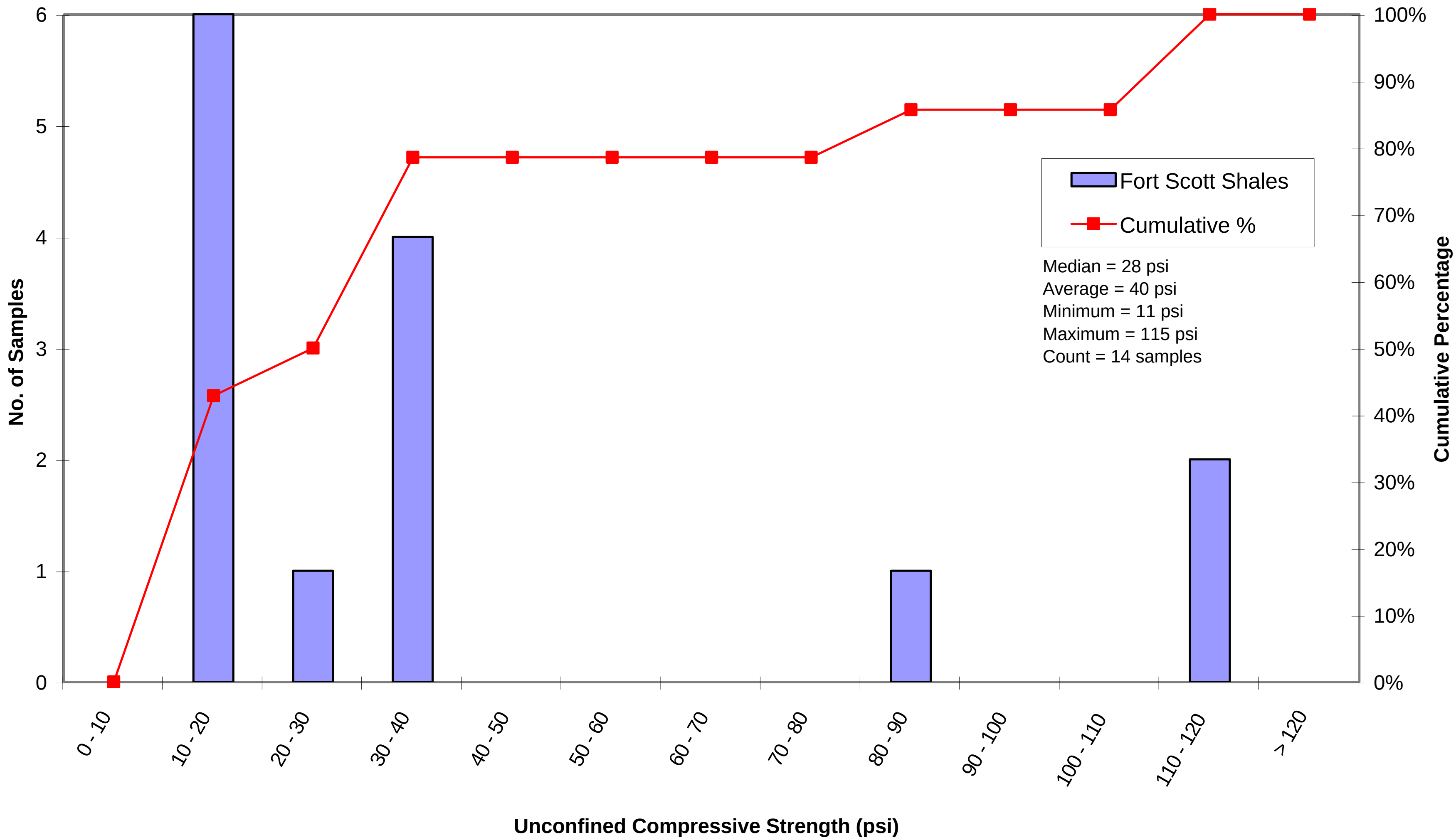
St. Louis Lambert Airport Expansion Program St. Louis, Missouri	PROJECT NO. 23-99000042.08

URS

DRN. BY: DSGN. BY: smo CHKD. BY:	Unconfined Compressive Strengths of Fort Scott Formation Limestone Samples	FIG. NO. 5-4



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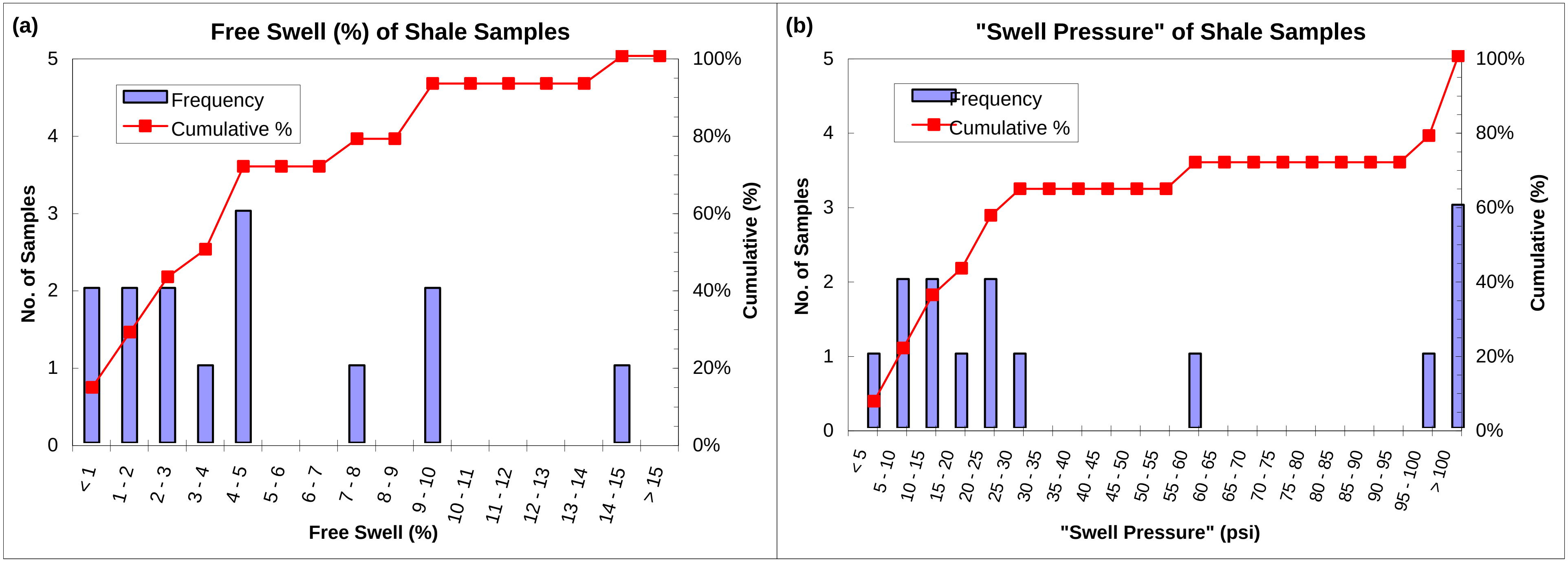
St. Louis Lambert Airport Expansion Program St. Louis, Missouri	PROJECT NO. 23-99000042.08

URS

DRN. BY: DSGN. BY: smo CHKD. BY:	Unconfined Compressive Strengths of Fort Scott Formation Shale Samples	FIG. NO. 5-5

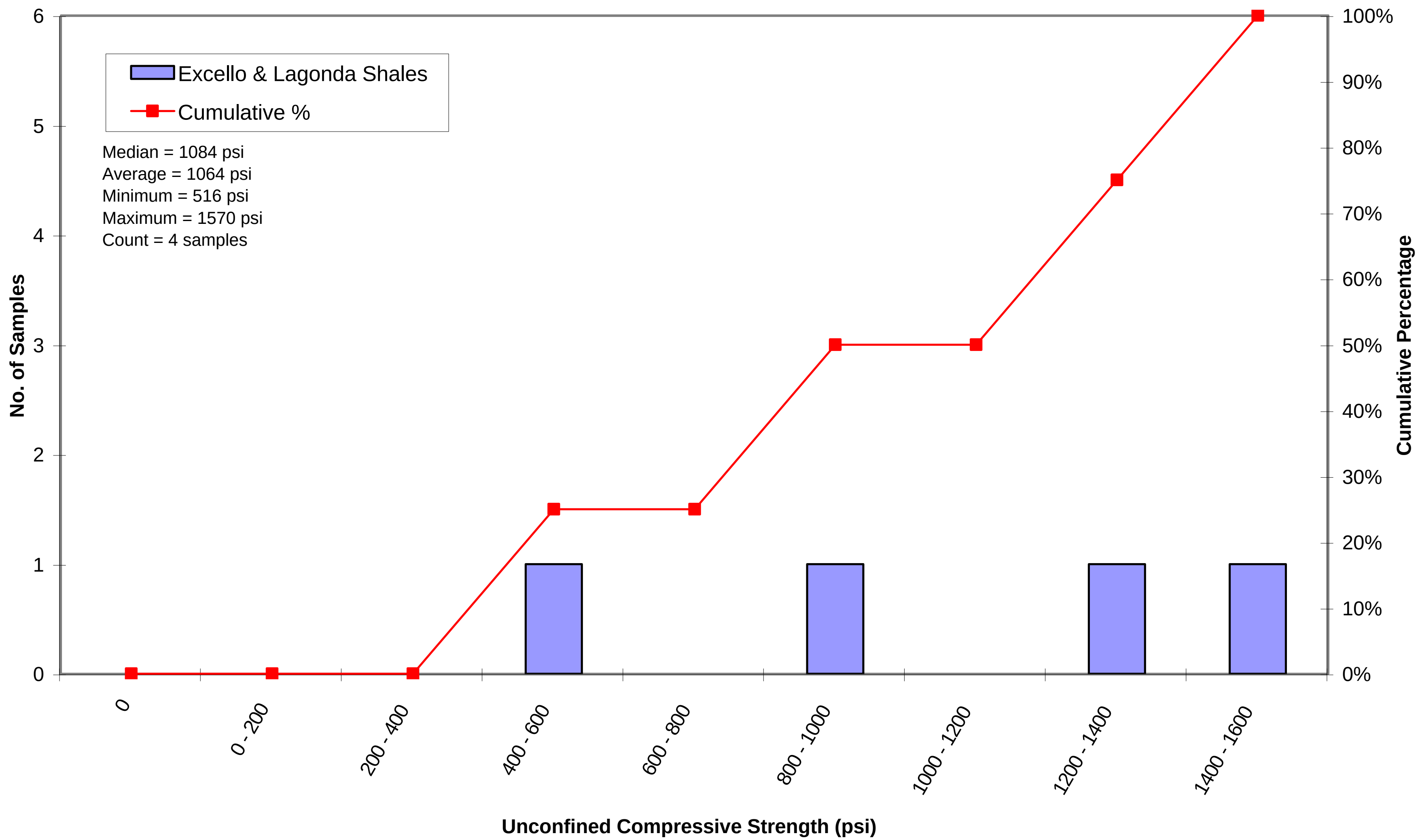


k:\geo\airport\cowmire\baseline report\shale swell 03/28/2002

St. Louis Lambert Airport Expansion Program St. Louis, Missouri	PROJECT NO. 23-99000042.08

URS

DRN. BY: DSGN. BY: smo CHKD. BY:	Free Swell and Swell Pressure of Fort Scott Formation Shale Samples	FIG. NO. 5-6



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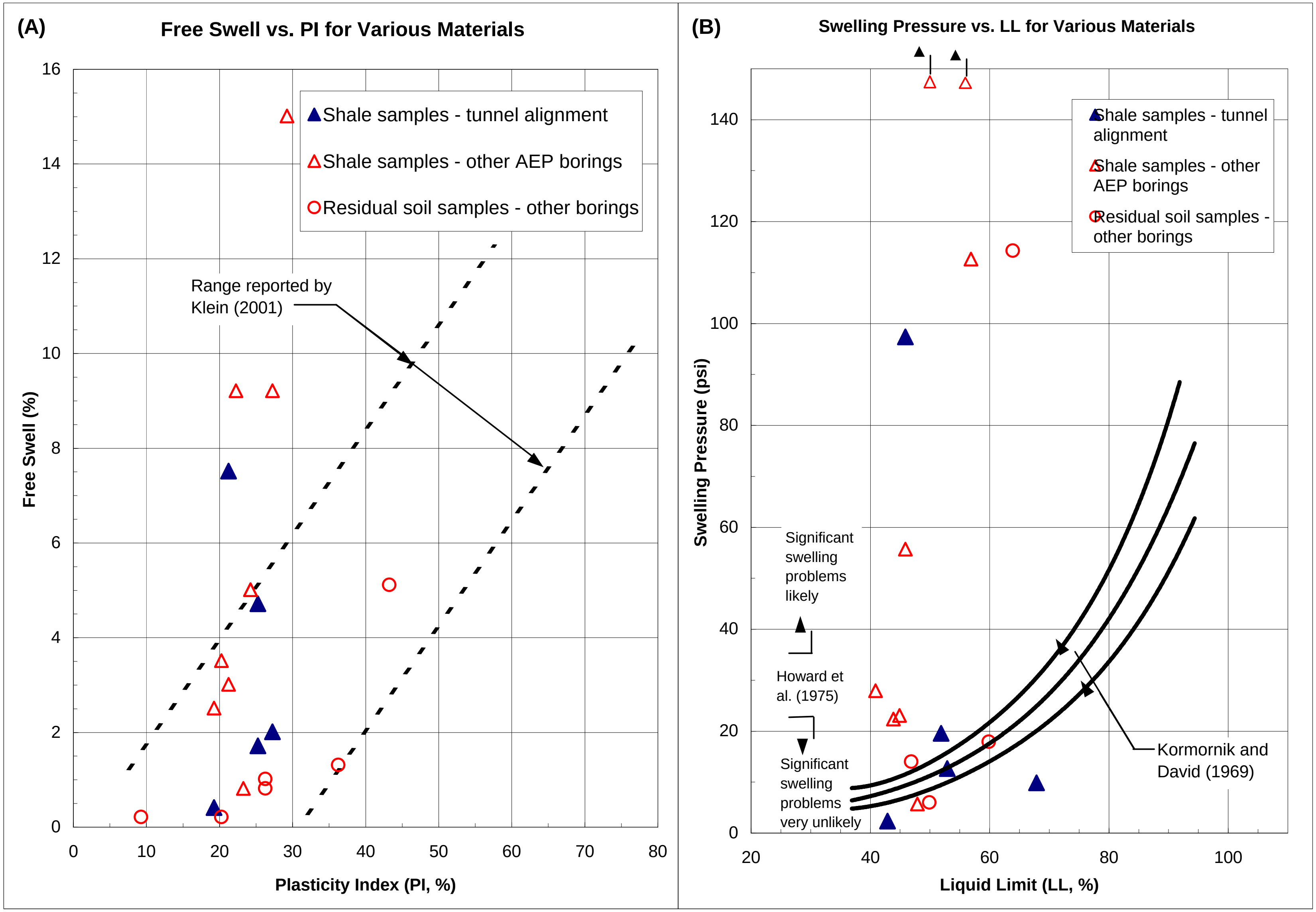
St. Louis Lambert Airport Expansion Program St. Louis, Missouri	PROJECT NO. 23-99000042.08

URS

DRN. BY: DSGN. BY: smo CHKD. BY:	Unconfined Compressive Strengths of Excello and Lagonda Formation Shale Samples	FIG. NO. 5-7



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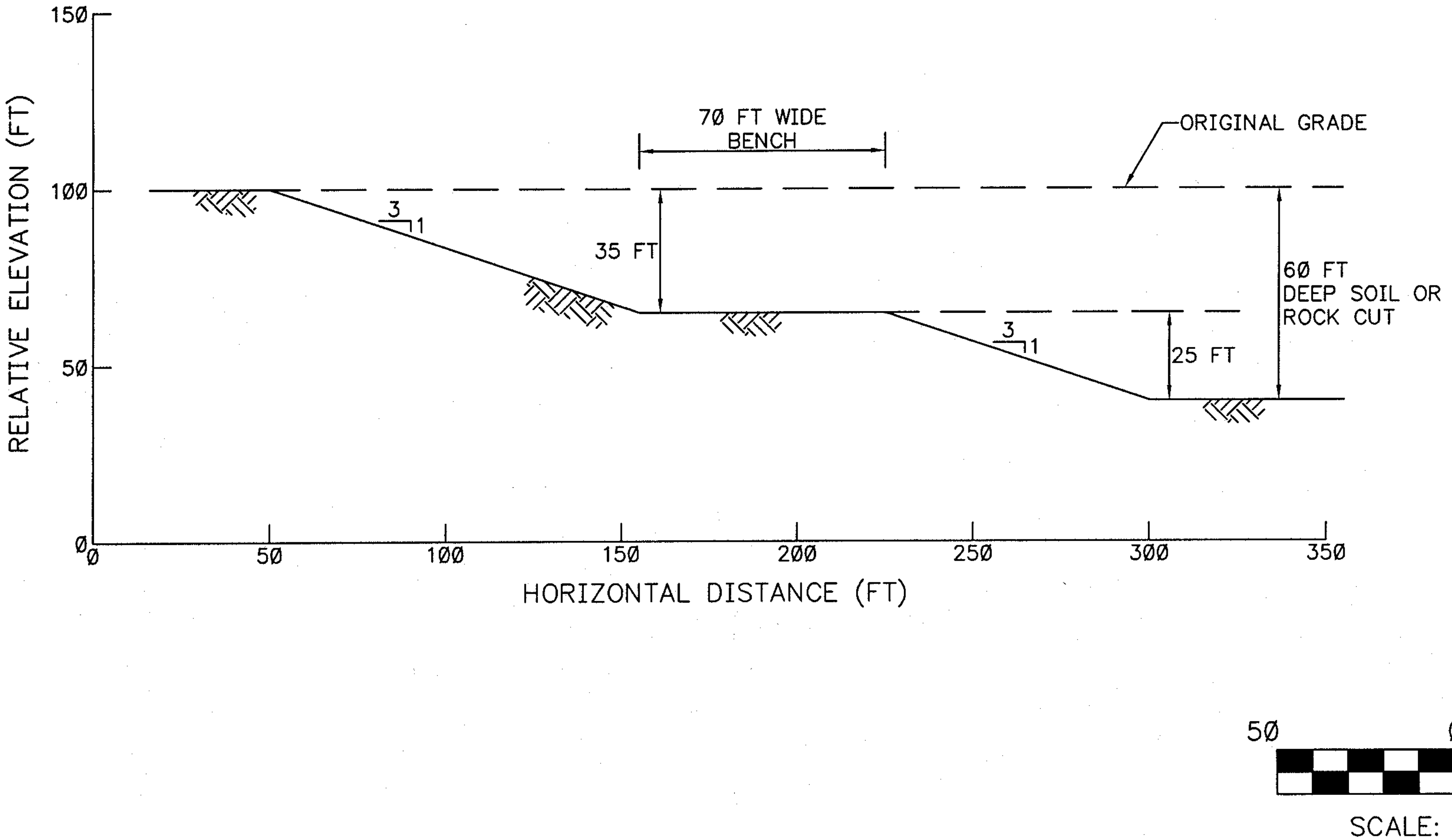
03/28/2002

St. Louis Lambert Airport Expansion Program St. Louis, Missouri	PROJECT NO. 23-99000042.08

URS

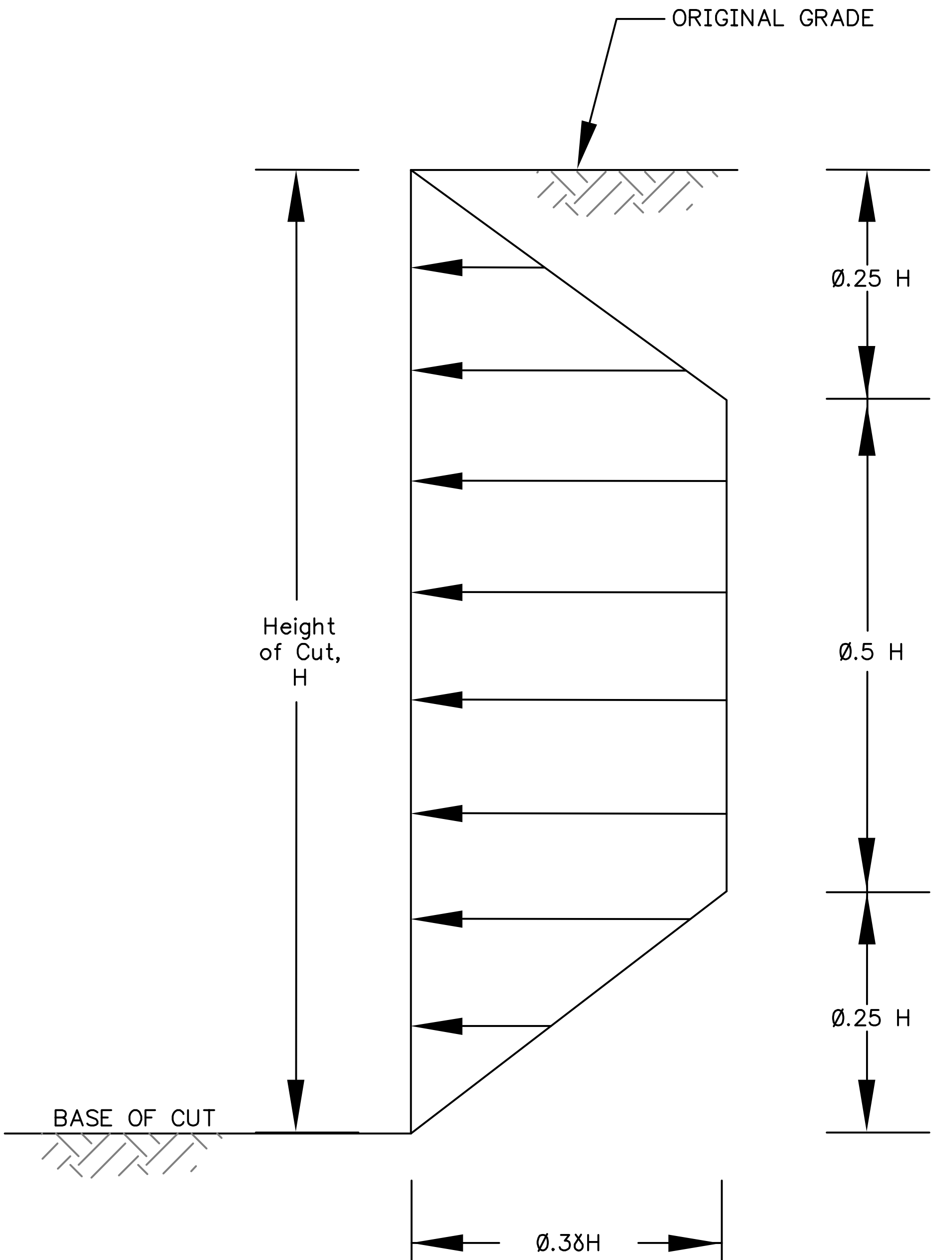
DRN. BY: DSGN. BY: smo CHKD. BY:	Correlations of Swell Parameters with Index Properties	FIG. NO. 6-1



LAMBERT–ST. LOUIS INTERNATIONAL AIRPORT EXPANSION ST. LOUIS MISSOURI	PROJECT NO. 2399000042.08

URS

DRN. BY: DS 5/10/02 DSGN. BY: SO CHKD. BY: KMB	Example of 60 FT Deep Permanent Cut in Soil & Rock	FIG. NO. 6–2



NOT TO SCALE

NOTE:
γ IS THE TOTAL UNIT WEIGHT OF SOIL

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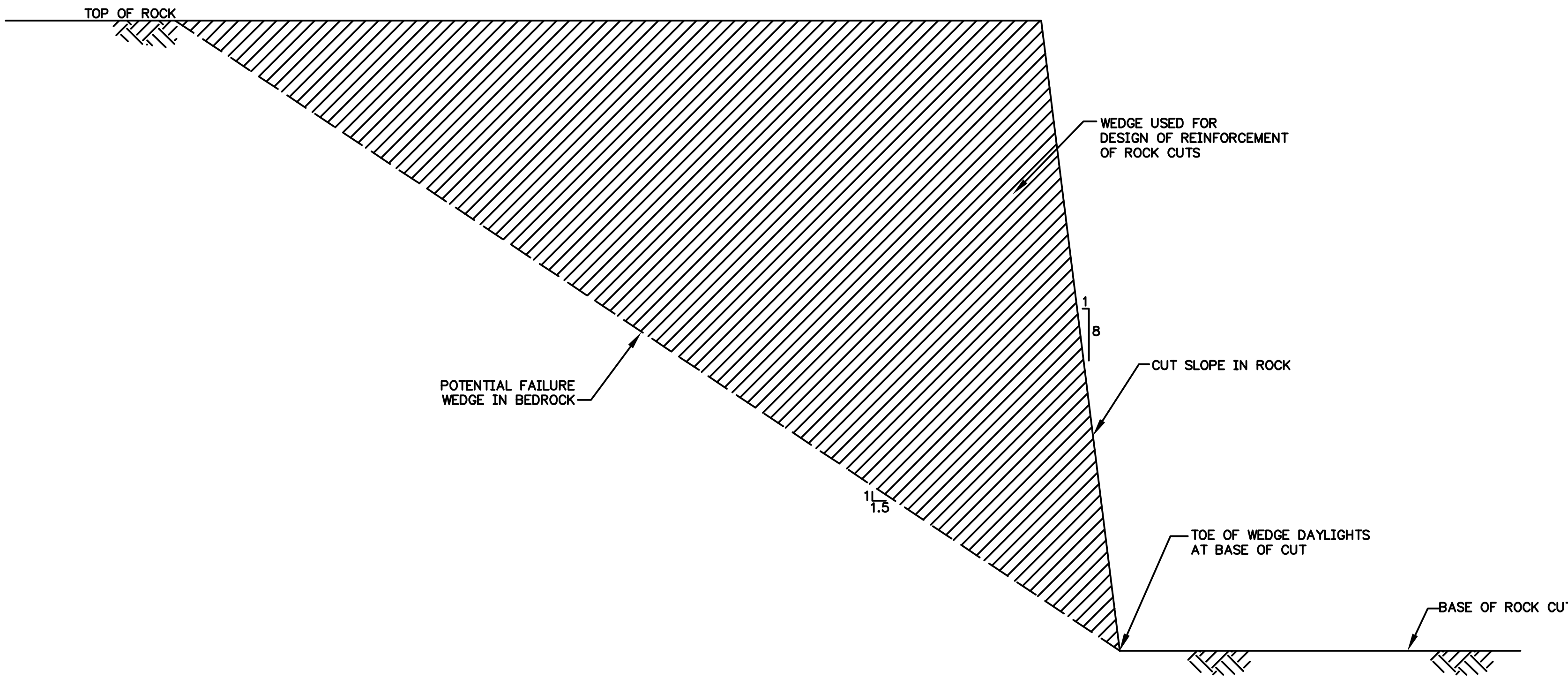
URS	
DRN. BY: djd 5/1Ø/Ø2 DSGN. BY: smo CHKD. BY:	Earth Loads for Design of Temporary Retaining Structures
FIG. NO. 7–1	



1Ø Ø 1Ø

SCALE: 1"=1Ø'

LAMBERT–ST. LOUIS INTERNATIONAL AIRPORT EXPANSION ST. LOUIS MISSOURI	PROJECT NO. 2399ØØØ42.Ø8
URS	
DRN. BY:DS 5/1Ø/Ø2 DSGN. BY:SO CHKD. BY:	Example Rock Wedge for use in Design of Temporary Supports — FIG. NO. 7–2