SUBSURFACE INVESTIGATION
PROPOSED CENTRAL TUNNEL REPLACEMENT
COLLEGE OF EASTERN UTAH
PRICE, UTAH

STATE OF UTAH
DEPARTMENT OF ADMINISTRATION SERVICES
DIVISION OF FACILITIES CONSTRUCTION MANAGEMENT
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CHAPTER 1

INTRODUCTION

The College of Eastern Utah ("CEU") in Price, Utah, proposes to construct a new 500-foot long central utility tunnel system that will replace an existing utility tunnel (see Plate 1). According to Mr. Dennis Geary (CEU facilities maintenance), the existing concrete tunnel was built in the 1960s with an overall height of about 6 feet and an inside height of about 5 feet. The tunnel is cast-in-place concrete with the walls supported on footings that are about 18 inches wide by 10 inches deep. The floor of the tunnel is a slab that was cast between the two walls, and the reinforced tunnel lid serves as a campus sidewalk. The excavation for the existing tunnel was about 7 feet deep. It is our understanding that the existing tunnel is being replaced because the lid is not strong enough to support campus fire-fighting trucks, the inside height is inadequate for maintenance personnel, and parts of the concrete are beginning to deteriorate. Following removal of the old utilities from the existing tunnel, CEU plans to backfill the existing tunnel and reconstruct the sidewalk.

The exact position of the proposed tunnel was unknown as of the writing of this report. The north-south section of the proposed tunnel will be built between the existing utility tunnel and the Library. It will extend southward from an area northwest of the Library to an area southwest of the Library, where it will branch east toward the new Reeves Classroom Building and west toward the Art Building. The new tunnel will contain high pressure steam lines, condensate return lines, data and communication cable trays, and other miscellaneous utilities. The tunnel may also contain high voltage distribution lines, chilled water piping, and domestic water piping.

The proposed tunnel may be constructed using corrugated metal culverts or cast-in-place or precast concrete boxes. The corrugated metal culverts may consist of 30-foot long sections that are 7 to 8 feet in diameter. A concrete floor will be placed at the bottom of the corrugated metal culverts to provide a flat walking surface. The concrete boxes may have inside dimensions of about 6 feet high and 8 feet wide with minimum 8-inch thick reinforced walls, floor, and lid. The tunnel joints will be sealed to prevent groundwater intrusion. Heavy vehicles, such as fire trucks or construction equipment, may be driven over all or parts of the tunnel, so it will be necessary to reinforce concrete box lids or provide sufficient cover over corrugated metal culverts. As necessary, the tunnel will be anchored to prevent upward groundwater pressures from lifting the tunnel. Cast-in-place concrete junctions will be used to link the new tunnel system to the existing tunnel system. The tunnel will slope with the ground surface from north to south (see Plate 1, which includes contour lines).

Construction is scheduled to extend from the late spring to the early fall of 2004 between CEU's spring and fall terms. It is our understanding that if the total project cost exceeds the project budget and/or the required construction time exceeds the allotted time-frame, renovation of the existing tunnel system may be considered as an alternative.

The Utah Department of Administration Services, Division of Facilities Construction Management, has contracted with Intermountain Consumer Professional Engineers, Inc. ("ICPE", Midvale, Utah) to design the new tunnel. ICPE has teamed with EarthFax Engineering, Inc. ("EarthFax") to conduct a site-specific subsurface investigation with the following scope of work:

1. Provide one direct-push drill rig to install three continuously-sampled small-diameter borings to a depth of about 25 feet below the existing ground surface to study the subsurface stratigraphy and to install piezometers to monitor groundwater levels.

2. Survey the elevations of the borings, and use an existing site plan to locate the positions of the borings.

3. Log the soils in the borings in accordance with the Unified Soil Classification System. Develop soil logs to include soil descriptions, soil classifications, and groundwater depths.

4. Conduct slug tests in the piezometers to estimate the hydraulic properties of the subsurface soils.

5. Review the existing geotechnical reports for the nearby structures to determine soil strength parameters, excavation sloping/shielding requirements, anchor capacities and depths to resist buoyant forces, and allowable bearing capacities.

6. Conduct slope stability analyses of the excavation.

7. Prepare a project report to include:

A. Site description
B. Present uses and condition of the land
C. Existing on-site structures
D. Field methods
E. Summary of existing geotechnical information
F. Site plan providing boring locations
G. Boring logs
H. Results of geotechnical analyses and slug tests
I. At rest, active, and passive earth pressures for design of the tunnel
J. Construction recommendations (backfill material and compaction requirements, subsurface drainage, native material re-use)
K. Excavation recommendations
L. Anchorage recommendations to offset buoyant forces

This document is divided into six chapters, including this introduction. Chapter 2 presents background information for the site, followed by a discussion of the investigation methods in Chapter 3. Chapter 4 discusses the field observations and results, with design and construction recommendations provided in Chapter 5. Cited references are listed in Chapter 6.

This report represents an expression of opinions and recommendations based on field observations, field tests, literature reviews, geotechnical analyses, and professional judgement. The characteristics of subsurface soils can vary significantly over a short distance due to the depositional environment and the gradation of the soil, the past stresses that have been imposed on the soil, and the moisture content of the soil. These varying characteristics can influence the shear strengths of the soils and the settlements of the structures bearing on the soils.

Recommendations presented in this report are minimal requirements to prevent failure or excessive movement of the project structures. Surface and subsurface loads will cause compression of the underlying soils and the subsequent settlement of structures and pavements, although these settlements can generally be reduced using good construction practices such as properly designed footings, well-compacted granular fill, proper surface and subsurface drainage, and concrete reinforcement.

The evaluation of geologic hazards at the site was beyond the scope of work for this project. However, geologic hazards may exist within localized areas of the site.

CHAPTER 2
BACKGROUND INFORMATION

2.1 SITE DESCRIPTION AND GEOLOGIC SETTING

The main CEU campus is located between 400 and 600 North Street and between 150 and 550 East Street in Price, Utah, which is located in Carbon County. The ground surface gently slopes from north to south.

Price is located in the Mancos Shale lowlands portion of the Colorado Plateau physiographic province. The subsurface soils typically consist of medium- to fine-grained flood plain and alluvial deposits.

According to Kleinfelder, Inc. (1994), there are no known active faults beneath or adjacent to the CEU campus. The nearest known faults are about 25 miles west of the site in the Joes Valley and Pleasant Valley fault zones, which are capable of magnitude 7 to 7.5 earthquakes and maximum ground accelerations of about 0.3g (g is the acceleration of gravity).

Kleinfelder (1994) estimated that the fine-grained subsurface soils beneath the Student Center had a low liquefaction potential for a magnitude 7 earthquake. Liquefaction is the partial or complete loss of bearing strength in a loose, saturated sand deposit when the deposit is subjected to strong loads of short duration. Shear stresses during loading tend to densify the loose sand, thereby causing the pore water to escape from the voids. Because the loading is of short duration, the water does not have time to escape and the pore-water pressure increases, reducing the effective intergranular stress on the soil particles, thereby reducing the bearing strength of the soil layer. Ultimately, given a sufficient seismic load, a saturated sand deposit can lose its shear strength, causing foundation failure, vertical settlement, and horizontal displacement.

According to Mr. Dennis Geary, a large northeast-to-southwest trending drainage channel traversed part of the campus. The exact position of this drainage channel is shown on some older aerial photographs. The drainage channel was filled with soil to construct the campus, and some garbage may have also been placed in the channel. The Science Building, Student Center, Career Center, Library, and much of the proposed tunnel system are located within the area of this drainage channel.

2.2 PREVIOUS SOILS INVESTIGATIONS

Numerous subsurface investigations have been conducted for the CEU campus. The results of some of these investigations are summarized as follows:

1. Coon, King & Knowlton (1976) investigated differential settlement and cracking at the Gymnasium and the Old Vocational Education Building. They indicated that the probable cause of differential movement at the Old Vocational Education Building was settlement due to a loss of soil strength in some areas and heaving due to expansive soils in other areas, with both conditions caused by an increase in the groundwater surface. The actual footing pressures of the building varied between 650 and 1670 psf. They recommended the installation of a campus-wide subsurface drainage system to minimize future damage to all campus buildings.

2. Fuhriman, Rollins and Company (1964; in Coon, King & Knowlton, 1976) indicated that differential settlement of six inches occurred in the Administration Building. The subsurface soils were very soft with Standard Penetration Test values between 2 and 12 blows per foot. They concluded that footing loads of 1000 to 3500 pounds per square foot ("psf") exceeded the allowable soil pressures of 400 to 700 psf, and that the settlements were due to local shear failure and not to primary consolidation.

3. Fuhriman, Rollins and Company (1965a; in Coon, King & Knowlton, 1976) conducted a subsurface investigation for the Science Building. The groundwater surface was at a depth of 10 feet. Standard Penetration Test values ranged between 7 and 11 blows per foot above the groundwater surface and 1 and 3 blows per foot below the groundwater surface. Unconfined compression tests were 986 to 1300 psf above the groundwater surface and 400 psf below the groundwater surface. They concluded that shear failure would occur before excessive settlement, so the allowable soil pressure was governed by shear failure. They recommended an allowable bearing capacity of 900 psf for footings placed within one or two feet of the ground surface. They indicated that increased moisture contents would greatly reduced the allowable bearing capacity of the deeper soil, and that the allowable bearing capacity near the groundwater surface was only 400 psf. They also recommended the installation of groundwater monitor wells and a subsurface drainage system to prevent a rise in the groundwater surface.

   The Science Building settled. A subsequent investigation by Rollins, Brown and Gunnell (1973; in Coon, King & Knowlton, 1976) indicated that the groundwater surface had increased to a depth of 6.5 feet, thereby increasing the compressibility of the subsurface soil and causing the settlement.

4. Fuhriman, Rollins and Company (1965b; in Coon, King & Knowlton, 1976) conducted a soils investigation for the proposed Heating Plant wherein they concluded that the extremely soft clayey silt below a depth of 8 feet should be limited to an allowable bearing capacity of 400 psf to prevent shear failure. The allowable bearing capacity of the soil near the ground surface was greater than 400 psf, but saturation due to rising groundwater levels or surface water infiltration would limit the allowable bearing capacity of the near-surface soils to 500 psf.

5. Woodward-Clyde-Sherard and Associates (1966; in Coon, King & Knowlton, 1976) conducted a soils investigation for the proposed Student Dormitory, wherein they recommended a suspended floor slab to prevent damage from swelling soils. They also indicated that it was necessary to prevent the foundation soils from becoming wet.

6. Fuhriman, Rollins and Company (1965c; in Coon, King & Knowlton, 1976) conducted a soils investigation for the proposed Music Building, which is about 60 feet south of the southwest end of the proposed tunnel. They indicated that the native silty clay had a low density (87 pounds per cubic foot), a low unconfined compressive strength (751 to 975 psf at in-situ moisture contents and 237 psf at saturated moisture contents), a high compressibility, and a high potential for shear failure for pressures greater than 600 to 700 psf. The groundwater surface was at a depth of 10 feet. They recommended an allowable bearing capacity of 600 to 700 psf provided that the foundation soils be prevented from becoming saturated.

   Coon, King & Knowlton (1976) indicated that the bearing strength of the soils at the Music Building decreased from 750 psf near the ground surface to 237 psf near the groundwater surface, and that shear failures could occur for soil pressures over 700 psf. Dames & Moore, Inc. (1989) conducted a geotechnical investigation of settlement-induced cracking of the walls for the Music Building. They surmised that the cracking may have been the result of incomplete foundation construction or poorly compacted structural fill beneath the footings.

7. Rollins, Brown and Gunnell, Inc. (1966; in Coon, King & Knowlton, 1976) conducted a soils investigation for the proposed Library Learning Center, which was constructed in 1968 and is about 40 feet east of the existing utility tunnel. They indicated that the loose, low-plasticity, highly compressible clayey silt and clay soils were underlain by clayey shale at a depth of 22 to 25 feet. Groundwater was at a depth of 10 feet, and the soil below the groundwater surface was extremely soft. Standard Penetration Test values were 2 to 4 blows per foot above the groundwater surface and 0 to 1 blow per foot below the groundwater surface. The consolidation test data indicated that considerable settlement would occur for footing loads greater than 400 to 500 psf. They recommended the use of a deep foundation system to support the structure, and lightly loaded spread footings only where the footings were not supporting major structural elements.

Delta Geotechnical Consultants, Inc. (1987) conducted a study to determine the cause of floor slab settlement at the Library. The building is supported on 30-inch diameter concrete piers that extend to bedrock at depths of about 28 to 30 feet. The first floor slab overlies 1 to 4 feet of compacted fill. By 1974, the slab had settled over 2 inches and was releveled with concrete capping. By 1987, parts of the slab had settled an additional 3 inches. Delta attributed the settlement to long-term consolidation of the soft subgrade soil as a result of stresses induced by the overlying fill material and the loads on the floor slab. Borings installed east and west of the Library encountered very soft to medium stiff sandy to silty clay extending to depths of 18 to 37 feet overlying very dense weathered siltstone and claystone. A boring installed south of the Library encountered sandy silt fill material and debris to a depth of 14 feet overlying stiff clay to a depth of 20 feet. The fill material and debris in this boring may have been used to fill the drainage channel that traversed the campus as described in Section 2.1. Groundwater was encountered at depths between 9 and 13 feet. Gradation tests indicated that the subsurface soils contained 8 to 16% very fine to fine sand and 84 to 92% clay and silt. Consolidation tests indicated that the subsurface soils were moderately to highly compressible. These soils had moisture contents between 17 and 25%, dry densities between 88 and 105 pounds per cubic foot ("pcf"), liquid limits of 30 and 33, plastic limits of 17 and 21, plastic indexes of 13 and 12, and Standard Penetration Test values between 1 and 8 blows per foot.

8. Rollins, Brown and Gunnell (1974; in Coon, King & Knowlton, 1976) conducted a soils investigation for the proposed Career Building, which is about 280 feet northeast of the north end of the proposed utility tunnel. They discussed the extremely soft subgrade soils and concluded that previous experience indicates that excessive differential settlements of shallow spread footings on campus may occur even though footings were designed using conventional methods of proportioning sizes to produce low soil pressures. The high groundwater surface contributes to this problem.

   Dames & Moore, Inc. (1989) conducted a geotechnical investigation of settlement damage at the Career Center, which included settlement of a column supporting parapets along the west side of the building. Laboratory tests indicated that the stiff near-surface clayey sand soils had a liquid limit of 19, a plastic limit of 15, a plastic index of 4, and a low compressibility with a slight moisture sensitivity. A stiff silt and clay layer at a depth of 10.5 feet had a liquid limit of 20, a plastic limit of 15, a plastic index of 5, and a low collapse potential. A stiff silty clay layer at a depth of 14 feet had a liquid limit of 34, a plastic limit of 18, a plastic index of 16, and it swelled 1.5% upon saturation with an 800 psf surcharge load. The subsurface soils had dry densities between 117 and 133 pcf at moisture contents between 4 and 13 percent. Siltstone was encountered at depths between 12.5 and 17 feet, and groundwater was not encountered at either boring, which extended to depths between 20.5 and 25 feet. Dames & Moore concluded that there were no observed natural subsurface conditions that contributed to the structural distress at the Career Center.

9. Kleinfelder, Inc. (1994) conducted a geotechnical investigation for the proposed Student Center, which is about 55 feet west of the existing utility tunnel. The subsurface soils consisted of medium stiff silty clay underlain by soft to very soft

silty clay and clayey silt extending to Mancos Shale bedrock at depths of 33 to 38 feet. Thin layers of silty gravel were encountered at depths ranging from 16 to 35 feet. Groundwater was encountered at depths between 6.5 and 10 feet. Consolidation tests indicated that the native fine-grained soils were highly compressible. These soils had typical shear strengths between 114 and 1020 psf, unconfined compressive strengths between 251 and 382 psf, moisture contents between 16 and 26%, dry densities between 90 and 116 pcf, liquid limits between 27 and 34, plastic limits between 15 and 18, plastic indexes of 11 and 16, Unified Soil Classifications of ML and CL, and typical Standard Penetration Test values between 3 and 10 blows per foot. To limit excessive structural settlements, Kleinfelder made the following recommendations:

A. The Student Center should be founded on minimum 36-inch diameter drilled piers extended into bedrock, and wall loads should be supported by grade beams between the piers.

B. The main floor should consist of a raised floor system.

C. Excavation bottoms should be dewatered and/or stabilized to facilitate construction.

In other words, Kleinfelder recommended that all loads at the Student Center should be transferred to bedrock because the subsurface soils presented unacceptable shear strength and settlement problems.

It should be noted that an allowable bearing capacity of 400 psf is much less than the 1500 psf that would be allowed for silt and clay soils by the usually conservative 2000 International Building Code. However, given the previous settlement problems of lightly-loaded structures on campus, an allowable bearing capacity of 400 psf does not seem to be overly conservative. The low allowable bearing capacity should be viewed as a prevalent weakness of the subsurface soils, and this weakness seems to further manifest itself in excavation slope failures as will be discussed in Section 2.3.

In addition to the settlement problems described above, it is our understanding that a new electrical vault and manhole near the east end of the existing utility tunnel have groundwater in them. A sump pump in the existing utility tunnel near the Art Building never shuts off due to groundwater accumulation. Therefore, it may be necessary to install permanent dewatering systems in the proposed tunnel system.

In general, the available information indicates that Mancos Shale bedrock is present at shallow depths at the east end of the campus, but soft saturated soils at the central and west end of the campus have caused settlement and excavation problems. The location of these problem soils seems to coincide with the infilled drainage channel described in Section 2.1. A boring installed by Delta Geotechnical (1987) south of the Library was the only boring in the previous subsurface investigations that was identified as containing fill material. However, it is difficult to identify fill material in borings unless debris is recovered with the soil samples. It is possible that many of the subsurface problems that have occurred at the campus have been caused by saturated, loose, fine-grained fill materials in this drainage channel that were not placed or compacted as engineered backfill.

2.3 PREVIOUS EXCAVATION OBSERVATIONS

Kleinfelder (1994) indicated that “temporary construction excavations extending deeper than four feet below existing site grade may become unstable due to the relatively soft soil and high groundwater conditions. Shoring or sloping of excavation walls will be necessary to protect construction personnel and provide temporary stability.”

Conversations with Mr. Dennis Geary indicate that recent slope failures of excavations between 5 and 11 feet deep have occurred for an underground fuel tank at the Heating Plant, for an underground vault near the north end of the existing utility tunnel, and for a water pipe excavation at a dormitory north of the tunnel. In each case, the deeper soft saturated soils lost their shear strength and "flowed like soup" into the excavations, thereby undermining the firmer near-surface soils and eventually causing these soils to slough into the excavations. The slope failures have gradually progressed outward and have even engulfed the excavation equipment. The fuel tank at the Heating Plant was designed to be installed below the ground surface, but slope failures prevented the installation of a deeper excavation. It should be noted that these previous excavations are small in comparison to the proposed utility tunnel excavation, which will be less stable due to the absence of perpendicular excavation sidewalls. Furthermore, deep-seated failure surfaces are more likely for long excavations.

As described in Chapter 1, the excavation depth for the existing utility tunnel was about 7 feet. It is unknown how the excavation was sloped, benched, or shored to facilitate construction.

CHAPTER 3
METHODS

3.1 BORINGS

Three borings (CEU-1 through CEU-3) were installed to depths between 18.5 and 24 feet to study the underlying stratigraphy and to install piezometers along the proposed route for the new tunnel. The piezometers were used to measure groundwater depths and to conduct slug tests. The locations of the borings were field-measured from known structures as shown on Plate 1. The elevations of the borings were surveyed using a level and the contours in Plate 1. The borings were drilled by On-Site Drilling (Taylorsville, Utah) using a tractor-mounted direct-push rig on November 18, 2003. The samples were collected in 4-foot long by 1.25-inch diameter clear acrylic tubes, which were cut lengthwise to examine the soil samples. The tubes were sealed with duct tape and labeled.

Five-foot sections of 1-inch diameter threaded PVC piezometers were installed in each boring. Each piezometer had an end cap on the bottom to prevent siltation. Solid PVC was used at the bottom 5 feet of each piezometer, at the top 9 feet of Boring CEU-1, at the top 4 feet of Boring CEU-2, and at the top 3.5 feet of Boring CEU-3. Screened PVC was used between the solid PVC sections. The annulus of each boring was filled with #20 silica sand from the bottom of the boring to a depth of 3 feet. The top 3 feet of each annulus was sealed using granular bentonite.

The borings were logged by a geotechnical engineer from EarthFax. The soil boring logs are presented in Appendix A. The logs include recovery percentages, graphic logs, soil descriptions, Unified Soil Classifications, and soil colors using a Munsell Soil Color chart.

The Unified Soil Classification System defines soil types by their textural and plasticity qualities that affect the engineering behavior of the soils. The constituents that comprise soils can be categorized into 3 primary groups: gravels, sands, and fines (silt and clay). Gravel particles are larger than the openings in a No. 4 sieve, silt and clay are smaller than the openings in a No. 200 sieve (200 openings per inch), and sand particles are smaller than the No. 4 sieve openings but larger than the No. 200 openings. If the majority of a soil consists of gravel and sand, then it is referred to as a granular soil. If the majority of a soil consists of silt and clay, then it is referred to as a fine-grained soil.

Silt and clay are further classified based on their plasticity, although clay is often defined as particles smaller than 0.002 millimeters. The plasticity of a soil is determined using the Atterberg Limits, which generally consist of the Plastic Limit, Liquid Limit, and Plastic Index. The Plastic Limit, which is the moisture content at which a 1/8-inch diameter thread can be rolled, represents the moisture content at which the soil behaves like a plastic material. The Liquid Limit is the moisture content at which the soil behaves like a liquid with little shear strength. The Plastic Index is the difference between the Plastic Limit and the Liquid Limit.

Soils described by the Unified Soil Classification system are classified by the constituent that comprises the majority of the soil. Descriptors are added to identify the secondary constituent and/or the plasticity. For example, a silty sand consists of greater than 50% granular soil (of which the majority is sand), and silt is the secondary constituent.

3.2 SLUG TESTS

Slug injection tests were conducted on December 2, 2003 in the piezometers at Borings CEU-1 through CEU-3 to estimate the hydraulic conductivities of the upper aquifer. A slug test is conducted by rapidly changing the water level in a well or piezometer by means of the injection or withdrawal of a body of known volume (a "slug") into or from the water column, and monitoring the rate of water level recovery to the static, pre-test level. When the slug is rapidly lowered into the water column, the water level rises abruptly. Rapid withdrawal of the slug after the water level has fully recovered causes the water level to drop abruptly. The slug used in this investigation consisted of a 0.5 to 1 liter of culinary water.

Slug tests are considered to provide adequate information about hydraulic conditions when studies are not aimed at designing an exploitation program of the aquifer (Freeze and Cherry, 1979), although it is recognized that the radius of influence of a slug test is smaller than that of a long-term pumping test. Slug injection and slug withdrawal tests produce similar results if performed under similar field conditions, and if a sufficient length of time is allowed to achieve maximum recovery of the water level.

An electric water level indicator was used to measure the static water level and total depth of each piezometer. The measurements were made relative to the top of the piezometer casing. These values were used to determine the saturated thickness of the zone to be tested.

The pre-test static water level of the subject aquifer was measured with a pressure transducer with a maximum operating pressure of 10 pounds per square inch (23.07 feet of water). The transducer was placed at a known depth in the piezometer, and the water column height measured by the transducer was added to this known depth to approximate the water level. The transducer was also used to measure water levels during the slug tests, and the data were recorded by a data logger as the aquifer recovered to equilibrium. Each data file was transferred to an analytical program (AQTESOLV™), which allows rapid, graphical representation and log-linear regression analysis of test data. An analysis method was used that determines hydraulic conductivity for piezometers that penetrate unconfined aquifers. Values of time and actual water-level displacement due to injection or withdrawal of the slug are displayed on a semi-logarithmic plot (i.e., water-level displacement is represented on a logarithmic y-axis and time is represented on an arithmetic x-axis). The hydraulic conductivity ("K") is estimated from the equation:

(3-1)func{K~=~{{r_c}^2~ln(R_e/r_w)} over {2L}{1 over t}~ln~{y_o over y_t}}

where:

y_o	=	initial drawdown or residual drawdown in piezometer due to instantaneous removal or injection of the slug (ft)
y_t	=	drawdown in piezometer at time t (ft)
L	=	length of piezometer screen (ft)
r_c	=	radius of piezometer casing (ft)
R_e	=	equivalent radius over which head loss occurs (ft)
r_w	=	radius of piezometer, including sand pack (ft)
H	=	static height of water in piezometer (ft)
t	=	time (min)

and

(3-2)func{ln~(R_e/r_w)~=~({1.1} over {ln~(H/r_w)}~+~ {C} over {L/r_w})^{-1}} where:

C = dimensionless parameter which is a function of L/r_w (see Equation 3-1)

Equation (3-1) allows the hydraulic conductivity to be calculated from the water-level change in the piezometer. Because the hydraulic conductivity, casing radius, piezometer radius, the radius over which head loss occurs, and the screen length are constants, $(1/t) \ln y_o/y_t$ must also be a constant. Thus, the time/drawdown data should approximate a straight line if plotted in terms of $\ln y_o$ versus t. The quantity $(1/t) \ln y_o/y_t$ in Equation (3-1) is obtained from the first straight-line segment drawn through the field data.

AQTESOLV™ software generates semi-log plots of the data and automatically fits a straight line to the data according to user-defined weighting. If the entire range of field data do not approximate a straight line, only those early data which form a valid straight-line segment are weighted by the user such that the software produces the desired straight line approximation through the valid part of the data set. The straight-line fit automatically determines the value of y_o (y-intercept) and an arbitrary value of y_t at time t to solve Equation (3-1). Based on user-defined values of screen length and drill hole radius, the software determines the value of C to evaluate R_e in Equation (3-2).

The AQTESOLV™ software generates the straight line approximation by means of a nonlinear weighted least-squares parameter estimation technique, i.e., the Gauss-Newton linearization method. The estimation technique minimizes the difference between observed and estimated values through iterative solution of the system of linearized equations until convergence is achieved. To ensure the fit of the straight line, the software prints out the values of actual water levels, calculated water levels, and residual values (the difference between the actual and calculated water levels) derived by the parameter estimation technique. Additionally, the statistical values of mean, standard deviation, and variance are provided for the weighted residuals. These statistics indicate the goodness-of-fit of the straight line generated by the estimation technique.

CHAPTER 4
RESULTS

4.1 SOIL PROFILE

The logs for Borings CEU-1 through CEU-3 are presented in Appendix A. As described in Chapter 2.0, the soft subgrade soils may consist of loose fill material that was placed in a former drainage channel, but no debris or other evidence was encountered that would suggest the soil consisted of fill material.

The groundwater surface was at depths of 9.5, 8.6, and 9.5 feet for borings CEU-1 through CEU-3, respectively, on December 2, 2003. The groundwater surface will fluctuate in response to irrigation, seasonal changes, precipitation, and groundwater recharge and withdrawal. Groundwater levels are presently below the seasonal high and may rise higher in the spring and early summer months, which is when construction is scheduled to begin for the utility tunnel replacement project.

4.2 ALLOWABLE BEARING CAPACITY

The bottom of the tunnel will be at or below the groundwater surface (see Section 4.1). As described in Chapter 2, the native soils at this depth have an allowable bearing capacity of about 400 psf. A structural geotextile and a minimum 6-inch thick layer of crushed gravel should be placed in the bottom of the excavation to provide a firm working surface.

4.3 SLOPE STABILITY ANALYSES

Geoslope (Version 5.0) was used to conduct slope stability analyses for the proposed utility tunnel excavation. Bishop's Method was used wherein it was assumed that the failure surfaces would have circular shapes. Each trial cross-section and condition was analyzed using 32,000 trial failure surfaces to increase the probability of locating the most critical failure surface.

A review of the available geotechnical reports for the campus (see Chapter 2) indicated that the soil shear strength data consisted entirely of unconfined compression strength tests and Standard Penetration Tests. Both of these test methods evaluate the shear strength of a soil under undrained conditions, which is appropriate for slope failures in cohesive soils where the failures occur rapidly before excess pore water pressures can dissipate and before water can drain from the soils. Therefore, the shear strength data provided by previous projects were used to evaluate the stability of the proposed tunnel excavation.

Kleinfelder (1994) indicated that soils at the Student Center had shear strengths between 114 and 560 psf (average 360 psf) above the groundwater surface and between 125 and 600 psf (average 220 psf) below the groundwater surface. Fuhriman, Rollins and Company (1965a) reported that subsurface soils at the Science Building had shear strengths between 493 and 650 psf above the groundwater surface and 200 psf below the groundwater surface. Fuhriman, Rollins and Company (1965c) indicated that subsurface soils at the Music Building had shear strengths between 375 to 487 psf above the groundwater surface and 119 psf below the groundwater surface. Using these results, the slope stability analyses were conducted assuming that moist near-surface soils had a shear strength of 360 psf, and saturated subsurface soils had an average shear strength of 200 psf and a typical minimum shear strength of 150 psf.

As described in Section 4.1, the groundwater surface ranged between depths of 8.6 and 9.5 feet at Borings CEU-1 through CEU-3. Groundwater levels may rise higher by the planned excavation period during the spring of 2004. As shown on the boring logs in Appendix A, capillary rise caused by the fine-grained nature of the soil has saturated and softened the subsurface soils at depths as shallow as 5 feet. Therefore, the slope stability analyses were conducted assuming that the groundwater surface was at a depth of 7 feet and that saturated conditions prevail below a depth of 5 feet.

The soft saturated fine-grained subsurface soils classify as Type C in accordance with OSHA excavation recommendations, which indicate that excavations between 4 and 20 feet deep should be sloped at 1.5 horizontal to 1 vertical. Therefore, the slope stability analyses were conducted for excavation slopes of 1.5 horizontal to 1 vertical. The slope stability analyses were also conducted assuming that no loads (i.e., soil stockpiles or heavy construction equipment) would be applied to the ground surface adjacent to the excavation.

The results of the slope stability analyses in Table 4-1 indicate that the excavation would become unstable at depths between 9 and 11 feet depending on the shear strength of the soils below a depth of 5 feet. These results may be reasonable, or even slightly unconservative, given that observations described in Chapter 2 indicated that slope failures occurred in excavations between 5 and 11 feet deep. Failures of excavations with slopes of 1.5 horizontal to 1 vertical indicate that even OSHA's recommendations are unconservative for the site soils, just as an allowable bearing capacity of 400 psf is well below the 1500 psf that would be allowed by the 2000 IBC (See Chapter 2). Additional slope stability analyses indicated that flattening the slope to 2 horizontal to 1 vertical did not increase the critical safety factors in Table 4-1.

In general, excavations should have a minimum safety factor against slope failure of 1.3. Using this recommendation, the results in Table 4-1 indicate that the utility tunnel excavation should be limited to a maximum depth of about 7 to 8 feet. This recommendation seems reasonable given previous excavation experience on campus (see Chapter 2) wherein the existing tunnel excavation must have been stable at a depth of 7 feet, but 10- to 11-foot deep excavations were unstable for a nearby fuel tank and vault.

The computer graphics from the slope stability analyses indicated that the critical failure surfaces tended to be deep. These critical failure surfaces started at various locations within the excavation bottom and intersected the ground surface about 15 to 17 feet beyond the top of the excavation. These results have several consequences as follows:

1. Trench shields or shoring may protect site personnel from shallow slope failures, but they will not protect site personnel from deep-seated failure surfaces that pass beneath the shields or shoring.

2. Slope failures may damage nearby surface and subsurface items such as sidewalks, fountain, painted rock, light posts, trees and other landscaping, buried utilities (storm drain, sanitary sewer, chilled water, culinary water, fire water lines, and electrical conduits), and the existing utility tunnel. Furthermore, slope failures near the Library may result in the development of lateral forces on the drilled piers and grade beams supporting the Library.

3. An excavation with 1.5 horizontal to 1 vertical slopes will encompass a large

portion of the 42-foot width between the existing tunnel and the west end of the Library Building. For example, an 8-foot deep excavation with an 8-foot wide base would have an overall width of about 32 feet, which would leave little or no room between the excavation and the Library for access. Therefore, access by construction vehicles would have to occur across the existing utility tunnel, which should be protected with steel decking to prevent heavy equipment from breaking the concrete lid.

4. Soil stockpiles and heavy construction equipment adjacent to the excavation may reduce the stability of the slopes or even cause slope failures.

Given the problems outlined above, sheet piling may be a viable alternative to sloping the excavation. However, the costs and the effects of piling installation on nearby structures and utilities should be evaluated. The design of sheet piling is beyond the scope of work for this project.

4.4 SLUG TEST RESULTS AND DEWATERING

Slug test plots for the piezometers are presented in Appendix B, which include the time/drawdown plots, piezometer constants, and field data used to estimate hydraulic conductivities. Also listed in Appendix B are values of actual water levels, calculated water levels, and residual values (the difference between the actual and calculated water levels) derived by the parameter estimation technique. Statistical values of mean, standard deviation, and variance also are provided for the weighted residuals. Table 4-2 presents the slug test input data, and Table 4-3 presents the results of the analyses, including the hydraulic conductivity and transmissivity of the soil. The hydraulic conductivity values were taken directly from AQTESOLV™ plots, and a plot from each slug test was also analyzed.

The piezometer identified as CEU-3 did not respond to the injection of 0.5 liter of water. Approximately 1.5 hours elapsed with less than 0.05 feet of recovery. Therefore, the test was terminated at this location. CEU-1 and CEU-2 responded to the injected volume of water, and the results indicate a very tight formation that passes water slowly. Consequently, it is expected that groundwater will enter the proposed utility tunnel excavation slowly, and that the groundwater can be removed through the use of a gravel-filled drainage trench and a 4-inch diameter drain pipe in the excavation. The drainage trench and pipe should slope to a sump at the downgradient (south) end of the excavation.

4.5 BUOYANT FORCES AND ANCHORAGE

Kleinfelder (1994) indicated that the groundwater surface at the Student Center was as shallow as 6.5 feet. This shallow groundwater surface can create upward buoyant forces on the proposed utility tunnel. These forces can be eliminated by installing and operating a dewatering system that keeps the groundwater surface below the bottom of the tunnel, but the tunnel system should be designed to resist upward buoyant forces in the event that the dewatering system fails. These resisting forces would include the weight of the tunnel, the weight of overlying materials, and frictional forces on the tunnel walls. If additional resisting forces are necessary, the proposed tunnel could be anchored to the existing tunnel, or the proposed tunnel could be anchored to cast-in-place concrete blocks or helical piers. Conversations with Intermountain Helical

Piers Corp. (Bluffdale, Utah) indicate that helical piers with 8- and 10-inch diameter helixes on the lead section installed to a depth of about 30 feet could provide an allowable resisting force of about 12,000 to 15,000 pounds for an estimated cost of about $900 per pier. It would be necessary to field test the helical piers to verify that these capacities could be achieved. In the interest of conservatism, the resisting forces should have a suitable safety factor against buoyant uplift in the event that the groundwater surface rises to shallower depths.

4.6 COEFFICIENTS OF LATERAL EARTH PRESSURE

The lateral pressure exerted by subsurface soils on the tunnel walls is a function of several factors including the type and density of the soil, the presence of groundwater, movement of the structure relative to the soil, and loading conditions. Generally, lateral earth pressures can be classified as active earth pressures, passive earth pressures, and at-rest earth pressures. In all cases, the coefficients associated with these various earth pressures are defined as the ratio of the effective horizontal stress (pressure) acting on the retaining structure to the effective vertical stress caused by the weight of the soil behind the structure. Both the horizontal and vertical effective stresses are a function of the depth below the ground surface. The lateral pressure coefficients do not include the effect of hydrostatic groundwater pressures against the walls.

4.6.1 Active Earth Pressure

Active earth pressure occurs when a wall moves away from the soil sufficiently to mobilize the shear strength of the soil. Thus, only walls that can rotate slightly should be designed using the coefficient of active earth pressure, K_a. Active earth pressures should not be used for this project because the tunnel walls should not deflect.

4.6.2 Passive Earth Pressure

Passive earth pressures occur when the structure moves toward the soil, thereby compressing the soil mass. The lateral backfill pressures will be approximately equal on both sides of the tunnel, so passive earth pressures will not develop.

4.6.3 At-Rest Earth Pressure

At-rest soil conditions will prevail for the proposed tunnel walls because the walls will not deflect. For lightly compacted structural backfill, the coefficient of at-rest earth pressure (K_o) is 0.50, which equates to a static fluid pressure of 63 pcf for a compacted unit weight of 125 pcf. If the backfill soils are heavily compacted, then K_o is 1.0 to 1.5 depending on the compactive effort (Winterkorn and Fang, 1975).

4.7 FRICTION FACTORS

A friction factor of 0.25 can be used for concrete or metal against the native soils. A friction factor of 0.35 should be used for concrete against compacted structural fill.

4.8 SOIL EXPANSION AND SHRINKAGE POTENTIAL

As a general rule, expansive/shrinking soils have montmorillonite clay in the soil, natural water contents near the Plastic Limit, and a source of water for expansive soils or a reduction in water for shrinking soils. Expansion can occur when pavements or buildings are constructed on dry clay, thereby preventing evaporation and allowing the moisture content to increase due to capillarity. Shrinkage can occur when groundwater levels are lowered after pavements or buildings have been constructed.

As described in Chapter 2, previous consolidation tests indicate that the subsurface soils have exhibited tendencies to swell and shrink upon saturation. It is expected that some movement of the subsurface soils beneath the proposed tunnel will occur with fluctuations in the groundwater surface. Therefore, the joints between the tunnel sections should be flexible enough to tolerate some movement while preventing groundwater from entering the tunnel.

TABLE 4-1

RESULTS OF SLOPE STABILITY ANALYSES FOR THE UTILITY TUNNEL EXCAVATION

Excavation Depth (feet)	Critical Safety Factor[a]	
	Average Shear Strength of 200 psf below depth of 5 feet	Typical Minimum Shear Strength of 150 psf below depth of 5 feet
7	1.6	1.3
8	1.4	1.1
9	1.2	1.0
10	1.1	<1.0
11	1.0	<1.0

[a] Rounded to the nearest 0.1. All slope stability analyses were conducted assuming that the excavation was sloped at 1.5 horizontal to 1 vertical in compliance with OSHA requirements for Type C soils.

TABLE 4-2

SLUG TEST INPUT DATA

Piezometer Identification And Test Number	Static Water Level (ft btc[a])	Diameter Of Casing (in)	Radius Of Borehole (in)	Screen Length (ft)	Total Depth (ft)	Aquifer Saturated Thickness (ft)
CEU-1 #1	9.55	1	1.25	10	24.35	14.8
CEU-1 #2	9.55	1	1.25	10	24.35	14.8
CEU-2 #1	8.61	1	1.25	15	19.4	10.79
CEU-2 #2	8.61	1	1.25	15	19.4	10.79
CEU-3 #1	9.56	1	1.25	10	18.6	9.04

(a) btc = below top of casing.

TABLE 4-3

HYDRAULIC CONDUCTIVITY AND TRANSMISSIVITY VALUES

Piezometer Identification and Test Number	Aquifer Saturated Thickness (ft)	Hydraulic Conductivity (ft/day)	Transmissivity (ft^2/day)
CEU-1 #1	14.8	0.146	2.16
CEU-1 #2	14.8	0.141	2.09
CEU-2 #1	10.79	0.0210	0.23
CEU-2 #2	10.79	0.0244	0.26
CEU-3 #1	NA	NA	NA

Note:
NA = Aquifer did not respond at this location.

CHAPTER 5
RECOMMENDATIONS

This report represents an expression of opinions and recommendations based on field observations, field tests, literature reviews, geotechnical analyses, and professional judgement. The characteristics of subsurface soils can vary significantly over a short distance due to the depositional environment and the gradation of the soil, the past stresses that have been imposed on the soil, and the moisture content of the soil. These varying characteristics can influence the allowable bearing capacity of the soil, structural settlement, and the stability of excavations.

It is recommended that a geotechnical or civil engineer inspect the tunnel excavation and backfilling for compliance with these recommendations. If the actual subsurface conditions differ from the conditions presented in this report, EarthFax requests that we be notified to facilitate modification of the recommendations in this report as necessary.

Recommendations presented in this report are minimal requirements to prevent slope failures or excessive settlement of the project structures. Slope failures and settlements can be reduced using good construction practices such as sloped or braced excavations, properly designed footings, well-compacted granular fill, proper surface and subsurface drainage, and concrete reinforcement.

The evaluation of geologic hazards at the site was beyond the scope of work for this project. However, geologic hazards may exist within localized areas of the site.

5.1 EXCAVATION RECOMMENDATIONS

All topsoil, surface vegetation, debris, frozen soil, ice, and other deleterious materials should be removed from the excavation area. These removed materials should not be used as backfill, but the topsoil can be used in landscaping areas.

Excavations not exceeding 4 feet in depth may be constructed with near-vertical sides. Excavations greater than 4 feet deep should be sloped or benched at 1.5 horizontal to 1 vertical to provide protection against collapse and slope failure. The excavations should be limited to a maximum depth of 7 to 8 feet to provide a minimum safety factor against slope failure of 1.3. However, excavation contractors should use their experience at the site and construct flatter slopes or shallower excavations as necessary. Excavation slope failures can be extremely dangerous. All excavations should be frequently inspected by a qualified engineer, and remedial actions should be taken if signs of slope instability (i.e., slumping, cracking, etc.) are observed. Sheet piling may enable a deeper excavation, but the costs and the effects of piling installation on nearby structures and utilities should be evaluated. The design of sheet piling is beyond the scope of work for this project, but the sheet piling system should be designed by a qualified engineer.

It is expected that wet to saturated soils will be encountered below a depth of 1 to 5 feet, and that the groundwater surface will be encountered between depths of 6.5 and 9 feet at the time of the excavation. As a result, the excavations will be soft and muddy, the excavated soil will have only a limited use as backfill unless it is dried out, and it will be necessary to dewater excavations that extend below the groundwater surface. Given the fine-grained nature of the native soils, well points are not recommended for dewatering. The tunnel excavation could be dewatered by digging a narrow 1-foot deep trench in the bottom of the excavation. This trench should be lined with a porous structural geotextile, and it should then be filled with clean 1-inch crushed gravel and a 4-inch diameter perforated drain pipe. The drain pipe should flow to one or more sump locations at the south (downgradient) end of the excavation, from which the groundwater can be pumped. The excavation work should begin at the downgradient (south) end of the tunnel to facilitate the installation of a sump. It is expected that groundwater flow into the excavation will decrease over time as the groundwater surface near the excavation is drawn down to the elevation of the drain pipe.

As a minimum, other excavation recommendations are as follows:

1. The landscape sprinkling systems within 20 feet of the planned excavation should be turned off at least one month prior to starting the excavation to minimize surface water infiltration that may saturate and soften the subgrade soils.

2. Precipitation runoff and construction water should be controlled such that it is not allowed to accumulate in the excavation.

3. Loose soils, organic materials, debris, and nonstructural fill should be removed from all excavations.

4. Soil stockpiles should be maintained at least 20 feet away from the excavation edge to prevent the stockpiles from becoming driving forces that could cause slope failures.

5. The contractor should use caution when placing heavy construction equipment near the edge of the excavation. Construction equipment should be moved away from the excavation if signs of slope instability develop (i.e., sloughing, cracking, settlement, etc.).

6. All underground pipes and utilities should be located and clearly identified on the ground surface prior to beginning the excavation. Pipes and utilities that are exposed in the excavation should be protected and supported to prevent their failure.

7. No personnel should be allowed to enter the excavation unless the groundwater dewatering system is operating (as necessary) and the groundwater surface is below the bottom of the excavation.

8. The excavator bucket should have a flat plate attached to the bucket teeth to minimize disturbance of subgrade soils in the bottom of the excavations. It is expected that the bottom of the excavation will be too soft to compact or to support large compaction equipment.

9. Excavations performed during cold weather should be protected to prevent the subgrade soils from freezing.

10. All surface encumbrances that may create a hazard to personnel should be removed or supported as necessary.

11. A ladder, ramp, or other safe means of egress should be located in the excavation such that no more than 25 feet of lateral travel is required to exit the excavation.

12. All personnel entering excavations greater than 4 feet deep should wear a safety harness. Emergency rescue personnel and the appropriate equipment should be

readily available whenever personnel enter the excavation. Unauthorized personnel should never be allowed to enter the excavation.

13. The potential impact of the excavation depth, width, and position on nearby underground utilities and surface items should be considered. As a minimum, the underground utilities include the storm drain, sanitary sewer, chilled water, culinary water, fire water lines, and electrical conduits. The surface items include sidewalks, the fountain, the painted rock, light posts, trees, and other landscaping. The costs for protecting, relocating, or replacing these utilities and surface items should be included in the construction cost estimate for the project. Furthermore, the impact of these items should be considered when developing the construction schedule.

14. The potential impact of the excavation on the existing utility tunnel and the Library should be considered. Slope failures could damage the existing tunnel or result in the development of lateral forces on the drilled piers and grade beams that support the Library.

15. The concrete lid over the existing utility tunnel should be protected from damage by heavy construction equipment during construction of the new tunnel.

16. Trench shields or shoring may protect site personnel that enter the excavation from shallow slope failures, but they will not protect site personnel from deep-seated failure surfaces that pass beneath the shields or shoring.

17. The excavation position and width should be considered when evaluating access to the utility tunnel for construction equipment and access to the Library for campus personnel.

18. The total costs and construction time requirements for the proposed tunnel system should be compared to the available budget and the allowable construction period. If the budget and time requirements are exceeded, then it may be necessary to consider redesigning the existing tunnel to meet the design criteria for the project.

5.2 ALLOWABLE BEARING CAPACITY RECOMMENDATIONS

The saturated native soils have a net allowable bearing capacity of 400 psf. A structural geotextile and a minimum 6-inch thick layer of clean crushed gravel should be placed in the bottom of the excavation to provide a firm working surface. The gravel surface should extend at least 6 inches beyond the edges of the tunnel, and the gravel should be encased in the geotextile to prevent piping of the overlying soils. The crushed gravel should have a maximum size of 2 inches with less than 10% passing a #4 U.S. Standard Screen. The gravel should be densified using repeated passes of a vibratory plate.

5.3 NATIVE MATERIAL REUSE AND TUNNEL BACKFILL REQUIREMENTS

Backfilling against the tunnel walls should progress upward at approximate equal levels on both sides to prevent unbalanced loading on the tunnel. Soils excavated from the site can be used to backfill the excavation in landscaping areas, although it will be necessary to dry excavated soils before they can be compacted. Structural fill should be used to backfill the excavation beneath sidewalks or other items that can not tolerate some settlement. A minimum 6-inch thick layer of road base should be placed beneath all pavements.

The excavated soils used as backfill in landscaping areas should be uniformly compacted to a minimum of 90% of the Modified Proctor dry density (ASTM D-1557; AASHTO T-180).

Structural fill should consist of durable granular soils that are free of vegetation, sod, rubbish, shale, clay, friable particles, frozen soils, ice, and other deleterious material. Structural fill should be well-graded with a maximum particle size of 2 inches, less than 15% fines, and an AASHTO classification of A-1-a. Standard road base material is typically suitable for structural fill (see Table 5-1 for gradation limits for a one-inch road base), although less expensive granular fills are commercially available. Structural fill should be placed in lifts that do not exceed 6 inches in compacted thickness.

Structural fill and road base should be uniformly compacted to at least 95 percent of the Modified Proctor maximum dry density (ASTM D1557; AASHTO T-180) at +/-2% of the optimum moisture content. Failure to achieve these compaction criteria can result in settlement and cracking of the surface pavements.

Precipitation runoff and construction water should be controlled such that it is not allowed to accumulate on the fill layers. Soft soils, organic materials, debris, frozen soil, ice, and nonsuitable fill should be removed from all fill areas. Ponded water should be removed prior to placing the overlying fill layer. Fill layers that become saturated and softened should be removed and replaced with suitable compacted fill. Frequent nuclear density/moisture tests should be conducted on each lift of compacted fill to verify that the required degree of compaction has been achieved. No overlying materials or lifts should be placed until the compaction tests have been reviewed and the lift has been approved.

Underground utility pipes should be set on at least 6 inches of sand to allow for even weight distribution. The soil beneath belled joints should be removed so that the weight of the pipe is not carried by the belled ends at the joints. The trenches should initially be backfilled to 6 inches above the utility with free-flowing sandy soils. Rocks larger than 1 inch should not be used in the bedding or backfill material adjacent to the utility. Structural fill should be used to fill the trenches above the sand. The backfill should be placed in 6-inch lifts, moisture conditioned, and compacted to a minimum of 90% of the Modified Proctor maximum dry density (ASTM D1557; AASHTO T-180).

5.4 AT-REST EARTH PRESSURES AND FRICTION PARAMETERS

The following lateral load coefficients and corresponding fluid pressures are recommended for use in designing the tunnel:

At-rest earth pressure due to static loading with lightly compacted backfill:

Lateral Load Coefficient, K_o = 0.50
Equivalent static fluid pressure = 63 pcf

At-rest earth pressure due to static loading with heavily compacted granular backfill:

Lateral Load Coefficient, K_o = 1.0 to 1.5
Equivalent static fluid pressure = 125 to 188 pcf

Friction factor for concrete against native soils = 0.25

Friction factor for concrete against compacted structural fill = 0.35

The at-rest earth pressures do not include hydrostatic pressures. Furthermore, these pressures assume that backfilling against tunnel walls progresses upward at approximately equal levels on both sides to prevent unbalanced loading on the tunnel.

5.5 CEMENT TYPE

Kleinfelder (1994) indicated that special sulfate-resisting cements are not necessary. Therefore, Type I/II cement can be used for the project.

5.6 TUNNEL JOINTS AND ANCHORAGE

The joints between the tunnel sections should be flexible enough to tolerate some differential movement caused by shrinking and swelling soils while simultaneously preventing groundwater from entering the tunnel.

Unless the groundwater surface around the proposed tunnel is permanently lowered using a subsurface drainage system, it is expected that the future groundwater surface could rise to within 5 or 6 feet of the ground surface. In this event, upward buoyant forces imposed by the groundwater on the tunnel should be resisted by the weight of the tunnel, overlying materials, and frictional forces on the tunnel walls. If additional resisting forces are necessary, the proposed tunnel could be anchored to the existing tunnel, or the proposed tunnel could be anchored to cast-in-place concrete blocks or helical piers. Intermountain Helical Piers Corp. indicates that helical piers with 8- and 10-inch diameter helixes on the lead section installed to a depth of about 30 feet could provide an allowable resisting force of about 12,000 to 15,000 pounds for an estimated cost of about $900 per pier. It would be necessary to field test the helical piers to verify that these capacities could be achieved. The total resisting forces should have a suitable safety factor against buoyant uplift.

TABLE 5-1

GRADATION LIMITS FOR 1-INCH ROAD BASE

Sieve Size	Gradation Limits (% Passing)
1 Inch	100
½ Inch	79 - 91
No. 4	49 - 61
No. 16	27 - 35
No. 50	15 - 23
No. 200	7 - 11

CHAPTER 6

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APPENDIX A

SOIL BORING LOGS

BORING CEU-1 Page 1 of 1

Recovery	Depth	Graphic		
			BORING No.:	CEU-1
			PROJECT NAME:	College of Eastern Utah - Central Tunnel Replacement
			COMMENTS:	1" diameter PVC piezometer w/ screen from 9 to 19 feet. #20 Sand from 3' to 24'.

OWNER/CLIENT:	Utah DFCM	PROJECT No.:	UC905.01
STATIC WATER:	9.5 feet	DATE:	November 18, 2003
DRILLER:	On-Site Drilling	METHOD:	Direct Push
RIG TYPE:	Tractor-mounted	LOGGER:	RKB, EarthFax Engineering
LOCATION:	By tunnel northwest of Library	ELEVATION:	5631.1 feet
DEPTH (Ft.):	24 feet	BACKFILL:	Bentonite seal top 3 feet of annulus

Recovery	Depth	Description
65		Sod and Topsoil. Topsoil is silty sand. SM.
	1.0	Sandy silt with clay and gravel. Possible fill. About 40% silt, 40% sand, 10% clay, and 10% gravel. Sand is very
	2.0	Sandy silt with clay. About 50% silt, 40% sand, 10% clay. Sand is very fine grained. Low plasticity. Moist -
	3.0	
	4.0	
25	5.0	
	6.0	
	7.0	
	8.0	
88		Clayey silt with sand. About 80% silt/clay and 20% very fine sand. Low to medium plasticity. Saturated - moisture
	9.0	
	10.0	
	11.0	
	12.0	
92	13.0	
	14.0	
	15.0	
	16.0	
73	17.0	
	18.0	
	19.0	
	20.0	
98	21.0	
	22.0	
	23.0	
	24.0	

CLAY SILT SAND

BORING CEU-2 Page 1 of 1

Field	Value	Field	Value
BORING No.:	CEU-2		
PROJECT NAME:	College of Eastern Utah - Central Tunnel Replacement		
COMMENTS:	1" diameter PVC piezometer w/ screen from 4 to 19 feet. #20 Sand from 3' to 24'.		
OWNER/CLIENT:	Utah DFCM	PROJECT No.:	UC905.01
STATIC WATER:	8.6 feet	DATE:	November 18, 2003
DRILLER:	On-Site Drilling	METHOD:	Direct Push
RIG TYPE:	Tractor-mounted	LOGGER:	RKB, EarthFax Engineering
LOCATION:	By tunnel southwest of Library	ELEVATION:	5626.8 feet
DEPTH (Ft.):	24 feet	BACKFILL:	Bentonite seal top 3 feet of annulus

Recovery	Depth	Graphic	Description
54	0.0–0.5		Sod and Topsoil. Topsoil is silty sand. SM.
	0.5–1.0		Sandy silt with clay and gravel. Possible fill. Gravel is 1/4". Low plasticity. Wet. Very stiff. ML.
	1.0–4.0		Sandy silt with clay. About 50% silt, 40% sand, 10% clay. Sand is very fine grained. Low plasticity. Wet to
52	4.0–24.0		Clayey silt with sand. About 80% silt/clay and 20% very fine sand. Low to medium plasticity. Saturated - moisture
67	8.0		
67	12.0		
85	16.0		
100	20.0		

Depth scale: 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0, 11.0, 12.0, 13.0, 14.0, 15.0, 16.0, 17.0, 18.0, 19.0, 20.0, 21.0, 22.0, 23.0, 24.0

CLAY SILT SAND

BORING CEU-3 Page 1 of 1

BORING No.:	CEU-3		
PROJECT NAME:	College of Eastern Utah - Central Tunnel Replacement		
COMMENTS:	1" diameter PVC piezometer w/ screen from 3.5 to 13.5 feet. #20 Sand 3' to 18.5'.		
OWNER/CLIENT:	Utah DFCM	PROJECT No.:	UC905.01
STATIC WATER:	9.5 feet	DATE:	November 18, 2003
DRILLER:	On-Site Drilling	METHOD:	Direct Push
RIG TYPE:	Tractor-mounted	LOGGER:	RKB, EarthFax Engineering
LOCATION:	By tunnel southeast of Library	ELEVATION:	5628.1 feet
DEPTH (Ft.):	18.5 feet	BACKFILL:	Bentonite seal top 3 feet of annulus

Recovery	Depth	Graphic	Description
44			Sod and Topsoil. Topsoil is silty sand. SM.
	1.0		Clayey silt with very fine sand and gravel. Possible fill. Gravel is 1/4". Low plasticity. Wet. Very stiff. ML.
			Clayey silt with sand. About 80% silt/clay and 20% very fine sand. Low to medium plasticity. Saturated - moisture
	2.0		
	3.0		
	4.0		
44			
	5.0		Silty sand with gravel. About 70% sand, 25% silt, and 5% gravel. Sand is very fine to fine grained. Gravel is 1/4"
	6.0		Clayey silt with sand. About 80% silt/clay and 20% very fine sand. Low to medium plasticity. Saturated - moisture
	7.0		
	8.0		
75			
	9.0		
	10.0		
	11.0		
			Weathered Mancos Shale bedrock. Soil texture is clayey silt with a trace of very fine sand. Medium plasticity.
	12.0		
100			
	13.0		
	14.0		
	15.0		
	16.0		
100			
	17.0		
	18.0		
			Mancos Shale bedrock. Penetration refusal at 18.5 feet.
	19.0		
	20.0		

CLAY SILT SAND

APPENDIX B

SLUG TEST RESULTS