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REPORT
GEOTECHNICAL STUDY
JORDAN AND SALT LAKE CANAL
CULVERT STRUCTURE
12300 SOUTH DESIGN-BUILD PROJECT
*HPP-STP-0071(12)0
DRAPER, UTAH

Submitted To:

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Submitted By:

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Salt Lake City, Utah

November 22, 2002

Job No. 2-817-004066/4065

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GRW
12257 South Business Park Drive
Suite 108
Draper, Utah 84020

Attention: Mr. Con Wadsworth

Gentlemen:

Re: Report
Geotechnical Study
Jordan and Salt Lake Canal Culvert Structure
12300 South Design-Build Project
*HPP-STP-0071(12)0
Draper, Utah

1. GENERAL

1.1 INTRODUCTION

This report summarizes geotechnical design recommendations for the new culvert for the Jordan and Salt Lake Canal (Canal) under-crossing at 12300 South Street in Draper, Utah. The general location of the site with respect to major topographic features and existing facilities, as of 1999, is presented on Figure 1, Vicinity Map. A more detailed layout of the site showing the planned layout and cross sections of the culvert and the locations of existing roadways is presented on Figure 2. The locations of the exploration conducted in conjunction with this study, and an exploration conducted for a previous study, are presented on Figure 3, Exploration Location Plan.

The general conclusions and recommendations described herein were provided to the design-build team during the course of this study verbally, via email, and in a memorandum dated October 15, 2002[1]. The recommendations provided herein apply to the design and construction of the culvert and do not address roadway construction or design or construction of temporary shoring and bracing that may be needed. Please note that the conclusions and recommendations provided herein are subject to review and comment by external agencies and may be revised accordingly.

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1 "Memorandum, Geotechnical Recommendations, Culvert Structure, Jordan and Salt Lake Canal, 12300 South design-Build Project, *HPP-STP-0071(12)0, Draper, Utah," AMEC Job No. 2-817-004061.

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1.2 PROJECT DESCRIPTION

The 12300 South Design-Build Project (Project) will consist of widening 12300 South Street at the location of the Canal. In order to widen the roadway, the existing culvert under 12300 South will be replaced with a larger and longer culvert. Based on preliminary culvert design information, the culvert will be a reinforced concrete, box structure with a length of about 137 feet, an inside width of 20 feet, and an inside height of 6 feet. The culvert will connect at the north end to an existing box culvert of similar size and replace the existing, smaller box culvert beneath 12300 South. We understand that, in order to maintain traffic, the new culvert will consist of three sections constructed in two phases.

Site earthwork will include excavation for the culvert sections, placement of a granular bedding fill, and placement of wall backfill. The invert elevation of the culvert will drop from 4414.61 feet at the south end to 4414.58 feet at the north end. As currently designed, the culvert will have a base slab thickness of 1.25 feet and be installed over a minimum 1.5-foot thick zone of granular backfill borrow fill as required by the Canal owner, the Salt Lake City Department of Public Utilities. Planned pavement grades at the culvert will be very close to existing grades, varying from 4424.1 feet at the crown to around 4423.0 feet at the gutter. Accordingly, the base of the required excavation is projected to extend about 11.0 to 12.0 feet below existing roadway grades, with a slightly shallower excavation projected for the portion of the culvert extending south of the existing roadway.

We project that, under normal operations, the culvert will vary from about one-half full to nearly full for an extended period of several months each year. Assuming an even distribution of load across the base of the culvert, we have calculated a maximum base contact pressure of about 1,300 pounds per square foot under the maximum total load of the culvert. The maximum total load of the culvert would include the weight of the culvert, the weight of the water when full, the overlying pavement section, and a traffic surcharge of 250 pounds per square foot. When no water is in the culvert, the maximum base contact pressure is calculated to be about 950 pounds per square foot. It should be noted that these maximum base contact pressures are only slightly more than those calculated for the existing culvert.

2. PREVIOUS REPORTS AND INVESTIGATIONS

In the 1960's, the Utah State Department of Highways (UDOH) completed the Draper Cross Roads to Riverton Project along 12300/12600 South Street. For that project, UDOH conducted soil borings at the canal crossings, including a boring at the Jordan (and Salt Lake) Canal to a depth

of about 37 feet below grade existing at that time. The log of that boring is included on a drawing prepared in 1961[2] for the canal crossings. The approximate location of that boring is indicated on

2 Drawing D-758, Sheet 7 of 7, Titled "Draper Cross Roads to Riverton, Jordan and South Jordan Canal Bridges, Soil Data"

Figure 3, Exploration Location Plan. The approximate location of the boring (labeled UDOT-1) is projected from the location indicated on the 1961 UDOH drawing.

3. EXISTING FACILITIES

The culvert location is currently occupied by the existing 5.5-foot high by 15.0-foot wide (inside dimensions) culvert extending under 12300 South Street and by the 12300 South Street pavement structure, including curbs and gutters. Connected to and extending north of the existing culvert is a 6.0-foot high by 20.0-foot wide (inside dimensions) culvert. At the south end, the existing culvert has a headwall and wing walls and opens to the existing open-channel portion of the Canal. The total length of the existing culvert from headwall to the larger culvert to the north is about 100.0 feet. The invert of the existing culvert matches the planned invert of the new culvert structure. The new culvert structure will extend from the existing larger culvert on the north end to a point approximately 37.0 feet past the southern end of the existing culvert.

According to a 1964 as-built drawing[3] by UDOH, the Draper Cross Roads to Riverton Project included constructing a bridge crossing over the Canal. That bridge consisted of pile-supported abutments supporting a deck spanning about 19 feet. At some point, the existing box culvert apparently replaced the bridge. If so, the timber piles supporting the bridge abutments may still be in place and could be encountered during excavation.

Numerous underground utilities extend along 12300 South Street at the Canal crossing. These utilities include gas lines, numerous telephone and fiber optic lines, storm drain lines, a sanitary sewer line, and culinary water lines. Based on the limited cover over the existing culvert structure, and the limited cover planned over the new structure, we project those utilities pass beneath the existing culvert structure.

4. FINDINGS

4.1 SITE SURFACE CONDITIONS

Site conditions are primarily defined by the existing site structures and other facilities occupying the site as described in the previous section. The site is essentially flat, with the exception of the open-channel portion of the Canal to the south of 12300 South Street. Areas not covered by asphalt pavement or concrete curb and gutter are currently vegetated. North of 12300 South Street vegetation is sparse, consisting mostly of native grasses and weeds. South of 12300 South Street vegetation consists of native grasses and weeds, and scattered brush and trees.

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3 Drawing D-758, Sheet 1 of 7, Titled "Draper Crossroads to Riverton, Jordan & South Jordan Canal Bridges, General Layout," dated February 3, 1964.

4.2 GEOLOGY

The site is located at the eastern edge of the Bonneville Basin, a deep, sediment-filled structural basin along the eastern edge of the Basin and Range physiographic province. The Bonneville Basin covers most of northwestern Utah and is bounded immediately to the east of the site by the Wasatch Range of the Middle Rocky Mountain physiographic province.

The topography and near-surface geology at the site are the result of deposition and erosion processes, as well as tectonic activities, over the past approximately 32,000 years. During the Bonneville Lake cycle, which began approximately 32,000 years ago, Lake Bonneville began a slow transgression (rise), with several fluctuations and pauses, before reaching its highest level at about 5,200 feet above sea level about 16,000 years ago. Approximately 14,500 years ago, the lake level dropped rapidly to a level of about 4,880 feet above sea level as the result of catastrophic down cutting of the lake's natural dam at Red Rock Pass in southeastern Idaho. Isostatic rebound and rapid erosion of shoreline sands and gravels and deltaic deposits of the Bonneville phase generally accompanied the rapid decline in the lake level. Additional down cutting at Red Rock Pass and isostatic rebound, as well as changing climatic conditions, reduced the lake level to that near the present day level (4,210 feet above sea level) of the Great Salt Lake about 11,000 years ago.

During deeper lake periods, relatively thick lacustrine sequences of clays, silts, and laminated clays and silts were deposited in the Bonneville basin. As the lake level dropped, streams down-cut through the lake deposits along the mountain front, resulting in complex alluvial and deltaic deposits of granular silt, sand, and gravel that fingered out into the receding lake. In addition, shoreline processes resulted in beach sand and gravel deposits at various stages of the Lake Bonneville regression, and earthquake activity during the Holocene resulted in areas of liquefaction and consequent deformation of predominantly lacustrine sediments.

Based on geologic mapping, subsurface conditions at the site consist predominantly of soils of lacustrine origin. Based on conditions encountered at the exploration locations, the lacustrine soils are underlain at depths of between about 110 and 120 feet by coarse-grained non-lacustrine soils. These coarse-grained non-lacustrine soils were deposited during interglacial periods similar to conditions today. The lacustrine soils are fine-grained, consisting of layers of clays, silts, and fine sands. Within the site vicinity, the lacustrine soils are overlain by or include near-surface zones of alluvial soils consisting primarily of stream deposits of Holocene age, possibly associated with the Willow Creek drainage, the northern arm of which crosses the Canal to the south of the 12300 South Street crossing. Where mapped, these alluvial soils typically consist of sand, silt, and minor amounts of clay and gravel. The mapped alluvium is poorly to moderately sorted and typically contains parallel and cross bedding.

4.3 GEOLOGIC HAZARDS

Site geologic hazards are considered to be primarily associated with seismic ground motion (versus fault hazard). There is a slight possibility of flooding of Willow Creek under an extreme storm event.

Due to the site location, geologic hazards such as landslides, rock falls, and mud and debris flows will not occur.

4.4 FAULTS AND SEISMICITY

In general, the site area is situated within the Intermountain Seismic Belt (Arabasz et al., 1992; Pechmann and Arabasz, 1995). This seismic zone extends from southern Nevada through the state of Utah northward to the Yellowstone area, and north along the mountainous part of western Montana. The Intermountain Seismic Belt is characterized by infrequent, yet moderate to large magnitude earthquakes with relatively shallow focal depths (typically less than 11 miles). Historically, the epicenter of the largest earthquake to occur within the Intermountain Seismic Belt was located near Hebgen Lake, Montana in 1959. That earthquake had a Moment magnitude of 7.3.

Geologic and geomorphic evidence shows that repeated, normal-slip surface faulting has occurred in the Salt Lake Valley through late Pleistocene to Holocene time. The Wasatch fault zone is the most obvious and continuous structural element of the prominent transition zone between the Basin and Range province to the west and the Colorado Plateau and Middle Rocky Mountain provinces to the east (Machette, et al., 1992). The Wasatch fault zone extends from Malad City, Idaho on the north to Fayette, Utah on the south, a distance of about 240 miles. Studies of the fault zone suggest that it is composed of 10 discrete segments, each of which may rupture independently during a major earthquake. Because the length of surface rupturing is generally dependent on earthquake magnitude, the largest earthquake on a single segment within the Wasatch fault zone is estimated to be a magnitude 7.3 event.

In general, earthquakes with magnitudes less than about 6.5 are not considered to cause surface rupturing. Therefore, the occurrence of these earthquakes is not limited to areas where faults have been mapped on the surface. Earthquakes with magnitudes between about 6.5 and 7.3 are expected in the region and are generally associated with identifiable faults, such as the Salt Lake segment of the Wasatch fault zone, that show evidence of recent seismic activity.

The Salt Lake Segment extends for about 28 miles from the Traverse Mountains to the south to the Salt Lake Salient to the north (Warm Springs area). The site is located about three miles west-northwest of the closest portion of the Salt Lake segment of the Wasatch fault zone, which presents the largest source of seismicity in the immediate project vicinity.

4.5 SUBSURFACE SOIL CONDITIONS

Our understanding of site subsurface conditions is based primarily on the results of our current field exploration and laboratory testing programs. Our field exploration program consisted of drilling one boring, AB-11, to a depth of about 30.5 feet below the existing ground surface. The approximate location of the boring is indicated on Figure 3, Boring Location Plan. The subsurface exploration program included installation of a temporary piezometer in Boring AB-11. A discussion of the field exploration procedures, together with our boring log, is presented in Appendix A.

The information from our current field-exploration and laboratory-testing programs has been supplemented by information recently developed at nearby sites for the Project by AMEC. Those sites include the planned temporary bridge site at the 12300 South/Interstate 15 (I-15) interchange (approximately 600 feet to the east) and the Union Pacific Railroad (UPRR) crossing site (approximately 2,100 feet to the west). That information includes characterization of subsurface conditions to depths ranging up to 145 feet and strength and compressibility characteristics for soils to depths in excess of 100 feet below existing site grades. In addition, information provided on the log of the previous UDOH boring was used to confirm subsurface conditions across the site.

In general, the soils underlying the culvert location to depths of at least 37 feet consist of predominantly fine-grained, cohesive, and cohesionless lacustrine soils. At Boring AB-11, these natural lacustrine soils are overlain by about 7 feet of medium dense to very loose granular fill consisting of silty sands and gravels. Based on the proximity of the boring to the existing larger culvert to the north of the existing culvert structure, this fill may be part of the backfill zone of that larger culvert.

The natural lacustrine soils underlying the site to the depths explored consist of moderately thick, relatively homogeneous to inter-layered zones of silty clays to clayey silts, sandy silts, and silts containing varying amounts of sand and clay. At Boring AB-11, these zones range in thickness from at least 5.5 feet to as much as 10.0 feet. Within each zone, the primary soil types often contain or are inter-layered with other soil types. The lacustrine soils to a depth of about 17.0 feet generally consist of silty clays to clayey silts of low plasticity that contain fine sand and grade with depth with occasional seams and thin layers of silty fine sand. Based on drive sampler penetration resistance, these silty clays to clayey silts range from stiff to very stiff.

Below a depth of about 17 feet Boring AB-11 encountered cohesionless silts to a depth of about 25 feet. These silts generally contain varying amounts of fine sand and occasional seams and thin layers of clean to silty, fine to medium sand. With depth, the silts grade to sandy and inter-layered with thin layers of clayey silt and silty fine sand. Based on drive sampler penetration resistance, these cohesionless silts range from medium dense to loose.

Beneath the cohesionless silts, Boring AB-11 encountered layered silty clays and clayey silts to the full depth explored of 30.5 feet. These layered silty clays and clayey silts are slightly to moderately plastic and contain sand and occasional to numerous seams and thin layers of silty fine sand. Based on drive sampler penetration resistance, these silty clays and clayey silts are medium stiff.

The soils encountered at the location of Boring AB-11 generally correspond to those described on the log of the previous UDOT boring drilled at the site. The UDOT boring encountered loose to medium dense cohesionless sandy silts at a depth of about 14 feet below original site grade. Based on the description on the log, these silts were observed to contain "varved" (seams and thin layers) fine sands.

Strength and consolidation tests were not conducted on samples from Boring AB-11 due to the extensive body of testing data available for soils encountered at the nearby temporary bridge and the railroad crossing sites. The results of the laboratory index testing conducted on samples from Boring AB-11 indicate that the natural soils underlying the site are very similar to those encountered to similar depths at the sites of the temporary bridge and the railroad crossing. Accordingly, the natural silty clays and clayey silts underlying the site are projected to exhibit relatively high strength and low to moderate compressibility. The cohesionless silts encountered at Boring AB-11 are projected to exhibit moderate strength and relatively low compressibility

4.6 GROUNDWATER

A piezometer was installed in Boring AB-11 to measure changes in static groundwater at the site. Stabilized groundwater levels were measured on three separate dates after installation. The results of the groundwater level measurements are summarized in the following table along with the corresponding approximate groundwater elevations (based on available ground surface elevation data provided in the RFP).

Stabilized Groundwater Levels[1]

Date	Depth below ground surface (ft)	Approximate Elevation (ft)
August 8, 2002	8.2	4414
September 7, 2002	8.8	4413
October 22, 2002	9.4	4413

As shown above, the stabilized groundwater level has dropped slightly over a three-month period. It should be noted that these measurements were obtained near the end of summer when groundwater levels are typically near their highest level, and within the fourth year of an extended drought cycle. Seasonal and long-term groundwater levels are expected to fluctuate by one to two feet, with the lowest groundwater levels occurring during the late fall and winter months.

5. EARTHQUAKE CONSIDERATIONS

5.1 SEISMIC HAZARDS

Site faulting and seismicity are discussed in Section 4.4 of this report. Seismic hazards that could be expected at the site would include ground shaking and potential liquefaction.

5.2 DESIGN CRITERIA

5.2.1 Site Class

Based on the subsurface conditions encountered at the site and deep (over 100 feet) subsurface information at the nearby sites of the temporary bridge and UPRR crossing, the site is considered to meet the criteria for Site Class D (stiff soil profile) as described in Table 1615.1.1 of the 2000 International Building Code (IBC 2000).

CPT probes conducted at the I-15/12300 south Street interchange (ACPT-1) and at the UPRR crossing (ACPT-3) included measurement of soil shear wave velocities to depths of over 100 feet. For depths of 100 feet below natural grades, the measured shear wave velocities averaged from 740 to 895 feet per second at ACPT-3 and ACPT-1, respectively.

5.2.2 Ground Motion

The United States Geologic Survey (USGS), through the National Earthquake Hazards Reduction Program (NEHRP, 1997), has evaluated and mapped the general seismic characteristics of the conterminous United States, particularly the western United States. The UDOT Manual of Instruction requires that the NEHRP ground motion data be used in seismic design of highway structures. The NEHRP ground motion data are probabilistic peak horizontal ground accelerations associated with points mapped on a grid system. The ground motion data for a site can be determined based on the latitude and longitude coordinates of the site. The acceleration values apply to the dense soil to rock boundary between Site Classes B and C (Site Class B-C Boundary), as defined by NEHRP 1997.

The coordinates of the 12300 South/I-15 interchange area are 40.5267 degrees north latitude and 111.8927 degrees west (negative) longitude. The ground motion values for the USGS grid point closest to the site location are summarized in the following table. The ground motion values in the table incorporate soil amplification factors for a Site Class D soil profile.

	Probabilistic Peak Horizontal Ground Accelerations (percent g)	
Acceleration Criteria	**10% in 50 yr Event (475-yr return period)**	**2% in 50 yr Event (2475-yr return period)**
PGA* (g)	0.30	0.55

* PGA - Peak Ground Acceleration (horizontal)

5.2.3 Liquefaction

Liquefaction is defined as the condition when saturated, loose, and cohesionless, sand-type soils lose their support capabilities because of excessive pore water pressure that develops during a seismic event. Several conditions are generally necessary for liquefaction to develop: loose cohesionless soils, a moderate to high groundwater table, and sufficient ground motion to cause the soils to liquefy.

We have analyzed the liquefaction potential for the site for both the 10 percent in 50-year and the 2 percent in 50-year seismic events. Our analyses were conducted using methods that correlate liquefaction primarily to soil density determined from sampler driving resistance (standard penetration test (SPT) N-values).

The results of our liquefaction analyses indicate that liquefaction may occur during both the 2 percent and 10 percent in 50-year events. Our analyses indicate that liquefaction could occur within zones of predominantly cohesionless silts and sands at depths of about 17 feet below existing site grades. Based on subsurface information from the nearby temporary bridge site, these zones are projected to range from in thickness from about 4 feet to 21 feet. It should be noted that the results of CPT-based liquefaction analyses at the temporary bridge site indicate that liquefiable layers within these zones range from about 2 to 8 feet in thickness, are separated by layers of non-liquefiable, predominantly cohesive soils, and vary in special and vertical distribution.

The primary effect of potential liquefaction at the site under both earthquake events will be differential, post-earthquake ground-surface settlement. Calculations based on soil density relationships (SPT "N" values), including calculations for the nearby temporary bridge site, indicate that potential ground-surface settlements ranging from about one and one-half to three and one-half inches may occur. Based on the results of CPT-based liquefaction analyses for both the temporary bridge site and the UPRR site, the total thickness of potentially liquefiable soils in the site vicinity is typically about one-half or less than that assumed for the SPT "N" value-based analyses. Accordingly, we project that total post-earthquake, ground-surface settlements at the site will be between about three-quarters and one and one-half inches. This calculation does not consider the benefit provided by the non-liquefiable deposits overlying the potentially liquefiable layers. If should be noted that this effect of liquefaction would be felt broadly across the site vicinity, with similar post-earthquake ground surface settlements occurring along 12300 South Street and along adjacent portions of the Canal.

Another significant effect of liquefaction is "lateral spreading," which can occur on sloping sites, sites with abrupt vertical faces intersecting liquefiable zones, or sites where the liquefiable soils are relatively thick and close to the ground surface. The ground surfaces surrounding the culvert structure are essentially flat, and the closest potentially liquefiable soil zone is considered to be about 17 feet below existing ground surface. Although the Canal banks to the south of the site would be considered steep faces, available subsurface information would indicate that they do not intersect liquefiable soil zones. Accordingly, the potential for liquefaction-induced lateral spreading affecting the Canal culvert structure is considered to be negligible.

6. LABORATORY TEST DATA

6.1 LABORATORY TESTING

A series of laboratory tests were performed on disturbed and undisturbed samples from our current explorations to assess geotechnical properties of the soils. Because a significant amount of shear strength and consolidation tests were conducted for the nearby temporary bridge and UPRR crossing sites, laboratory testing was limited to evaluation of soil index properties. A description of our laboratory program, including test procedures and results, is presented in Appendix C.

Classification testing was performed to determine soil index properties including natural moisture content, in-situ dry density, Atterberg Limits, and grain-size distribution. The index properties were determined for use in soil classification, correlation of field and other laboratory test data, and specific analyses, including liquefaction potential.

7. CULVERT STRUCTURE

7.1 GENERAL

The following sections provide geotechnical recommendations for design of the new culvert structure.

7.2 FOUNDATION DESIGN CRITERIA

7.2.1 Allowable Bearing Pressure

Since the depth of the culvert is less than one-half the width, the culvert is considered to be similar to a shallow, strip footing. In evaluating the allowable bearing pressure for the culvert, the relatively stiff cohesive soils encountered to the depths of about 17 feet below existing site grades were considered to be the controlling subsurface soils. Based on laboratory testing conducted at the nearby temporary bridge and UPRR crossing sites, these soils, when undisturbed, are considered to possess minimum undrained shear strengths of at least 1,800 pound per square foot.

We recommend that the culvert be designed using a maximum allowable bearing pressure of 3,400 pounds per square foot. This value includes a factor of safety of 3. This allowable bearing pressure may be increased by one-third for total load conditions (dead plus transient live loads) provided the increased bearing pressure is uniformly distributed along the base of the structure.

7.2.2 Settlement

As previously described in this report, we project that, under normal operations, the culvert will vary from about one-half full to nearly full for an extended period of several months each year. Assuming an even distribution of load across the base of the culvert, we calculate a maximum base contact pressure of 1,300 pounds per square foot under the maximum total load of the culvert. The maximum total load of the culvert would include the weight of the culvert, the weight of the water when full, the overlying pavement section and a traffic surcharge of 250 pounds per square foot. When no water is in the culvert, the maximum base contact pressure is calculated to be about 950 pounds per square foot.

Based on consolidation testing conducted at the nearby temporary bridge and UPRR crossing sites, the cohesive soils underlying the site are considered moderately to highly over-consolidated, with over-consolidation ratios above 3. The maximum anticipated based contact pressures will be less than the past consolidation pressures experienced by these cohesive soils. Accordingly, settlements due to consolidation of the underlying cohesive soils are expected to be elastic. For the above maximum base contact pressures, we have calculated maximum total settlements ranging from about one-half to five-eighths of an inch. Differential settlements along the culvert are expected to be negligible. Since settlement will be elastic, two-thirds to three-quarters of the expected settlements are expected to be complete within four to five weeks after construction of the culvert and placement of the overlying pavement section.

7.2.3 Lateral Resistance

For the base of the culvert established on at least one and one-half feet of compacted granular backfill borrow fill extending to undisturbed natural cohesive soils, we recommend using a coefficient of 0.45 for determining base sliding lateral resistance. If used in combination with resistance from passive earth pressures, the base-sliding coefficient must be reduced by a factor of safety of 1.5.

7.2.4 Installation

The culvert may be established on granular backfill borrow fill extending to undisturbed natural cohesive soils. Under no circumstances can the culvert be established directly on soft, wet, or disturbed soils, frozen soils, or within ponded water.

The excavation will encounter and likely terminate in undisturbed, natural, fine-grained cohesive soils, and the base of the excavation is projected to be below the static groundwater table. These natural, fine-grained, cohesive subgrade soils may degrade significantly during placement of forms and reinforcing steel, or other construction activities, especially where the base of the excavation is near or below the groundwater level and during wet periods of the year. It is our opinion that including the one and one-half feet of granular backfill borrow fill beneath the culvert will facilitate construction by limiting construction-related disturbance of the anticipated saturated, fine-grained, natural subgrade soils.

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If very soft, saturated, subgrade conditions are encountered, placement of the granular backfill borrow fill to the required density may be difficult, if not impossible. In that case, we recommend over-excavating the subgrade soils to a depth of at least 12 inches and replacing those soils with granular stabilizing fill as described in Section 8.3.2, Stabilizing Fill, of this report. Once the subgrade has been stabilized, the granular backfill borrow fill can be placed and compacted. Alternatively, a geotextile reinforcing/separation fabric must be placed over the exposed subgrade before placement and compaction of the granular backfill borrow fill. If a separation fabric must be used, we recommend placing a 2 to 3-inch layer of coarse gravel prior to placing the fabric. This will help limit displacement of the subgrade soil during placement of the initial backfill lift. If the granular backfill borrow fill becomes disturbed, the fill must be re-compacted to UDOT's requirements for granular backfill borrow prior to pouring concrete.

The width of granular backfill borrow fill below the culvert should extend laterally at least six inches beyond the edges of the culvert for each foot of fill thickness beneath the culvert. For example, if the width of the culvert is 22.0 feet and the thickness of the fill beneath the culvert is 1.5 feet, the width of the fill zone at the base of the excavation would be a total of 23.5 feet.

7.3 LATERAL EARTH PRESSURES

The culvert will retain backfill placed between the structure and the temporary excavation cuts. Since the backfill will be placed after the culvert deck is constructed, the culvert sidewalls will be restrained, resulting in an at-rest earth pressure state.

To facilitate placement and compaction, we recommend that the backfill consist of granular backfill borrow. In evaluating lateral earth pressure parameters, we assigned the granular backfill borrow a moist unit weight of 130 pounds per cubic foot and an internal friction angle of 35 degrees.

The following table lists equivalent fluid densities for use in determining design lateral earth pressures under static and seismic load conditions. The seismic criteria have been developed for horizontal acceleration coefficients of 0.30 and 0.55, which correspond to average ground motion return periods of 475 and 2,350 years.

	Equivalent Fluid Density (pcf)		
Load Condition	**Static**	**475-Year Event**	**2,350-Year Event**
At-Rest (restrained)	55	29[1]	54[1]
Passive	320[2]	629[3]	430[3]

[1] At-rest seismic equivalent fluid densities additional to the static at-rest values
[2] Factor of safety of 1.5
[3] Factor of safety of 1.0

If materials other than those described above are used as backfill immediately adjacent to the abutment walls, the above equivalent fluid densities will change and could increase.

In determining the lateral earth pressures acting on the culvert structure, we recommend the following approaches:

1. At-rest static and seismic earth pressures are determined using the above equivalent fluid densities. The total static at-rest force is determined and applied using a distribution appropriate for braced conditions. The total seismic at-rest force is determined using an inverted triangular pressure distribution that decreases with depth. The maximum seismic at-rest pressure is based on the height of the wall times the seismic equivalent fluid density. The total seismic at-rest force is applied at a point above the base of the wall equivalent to six-tenths the height of the wall.

2. The passive static and seismic earth pressures can be determined using the above equivalent fluid densities and are both applied using a triangular pressure distribution. Note that the seismic component of the total passive earth pressure acts in the opposite direction to the static component. Accordingly, seismic passive values greater than the static values are irrelevant.

If materials other than those identified above are used as backfill adjacent to the culvert sidewalls, the above equivalent fluid densities must be modified. Modified equivalent fluid densities could be significantly higher.

It should be noted that the equivalent fluid density values provided above are based on the assumption that the backfill materials will not become saturated. The equivalent fluid density values may be decreased by 50 percent if the backfill becomes saturated. However, full hydrostatic water pressures will have to be included.

8. EARTHWORK

8.1 SITE PREPARATION

Site preparation will include demolition of existing pavements, curbs and gutters, and the existing culvert structure, as well as removal of existing riprap and vegetation. All demolition debris and vegetation, and any other deleterious materials encountered, must be totally removed from the site.

The subgrade should be prepared in accordance with UDOT requirements. To avoid disturbance of potentially wet and soft subgrade soils, excavation to planned subgrade should be accomplished using a smooth-lipped bucket. Placement of granular backfill borrow, and stabilizing fill if necessary, should proceed immediately after the excavation is complete to avoid unnecessary subgrade disturbance.

The 1964 as-built drawings by UDOT indicate that the Canal crossing consisted of a bridge constructed as part of the Draper Crossroads to Riverton Project. That bridge consisted of pile-supported abutments supporting a deck spanning about 19 feet. At some point, the existing box culvert apparently replaced the bridge. If so, the piles supporting the bridge may still be in place and could be encountered during excavation. Piles encountered in the excavation must be cut off at an elevation of at least 2 feet below the base of the culvert.

8.2 TEMPORARY EXCAVATIONS

Excavations for the culvert are projected to encounter primarily undisturbed, natural, fine-grained, cohesive soils overlain by roadway fills. Based on shear strength testing conducted on similar soils at the temporary bridge and UPRR site, the undisturbed, natural, cohesive soils possess relatively high strength, and excavations within these soils are expected to remain stable even when cut at vertical or nearly vertical slopes.

Temporary construction excavations in the undisturbed, natural, cohesive soils not exceeding 4 feet in depth and not overlain by existing fills may be constructed with near-vertical side slopes. Deeper excavations up to 12 feet in depth in undisturbed, natural, cohesive soils above or below the groundwater table may be constructed with side slopes no steeper than one-quarter horizontal to one vertical (0.25H:1.0V). Loose or disturbed surficial soils at the top of temporary side slopes must be laid back at slopes no steeper than one and one-half horizontal to one vertical (1.5H:1.0V).

If cohesionless fine-grained or granular soils are encountered, particularly below the groundwater table, flatter side slopes, shoring and bracing, and/or dewatering systems will be required. To maintain traffic during phased construction, vertical or near-vertical cuts to depths of 11 to 12 feet may be required adjacent to travel lanes. Those cuts will likely require shoring and possibly bracing during construction for construction safety and protection of the 12300 South travel lanes.

Groundwater seepage, if present, is expected to be low and manageable using typical diversion ditches, small local sumps, and portable pumps.

Qualified personnel must inspect all excavations periodically. If any signs of instability or excessive sloughing are noted, immediate remedial action must be initiated.

8.3 STRUCTURAL FILL MATERIALS AND COMPACTION

8.3.1 General

In general, structural fill materials and their placement must conform to UDOT specifications. We anticipate that most structural fills used at the temporary bridge abutments or to construct the approach embankments will conform to UDOT's requirements for granular backfill borrow or embankment for the bridge. However, fills used to stabilize soft or saturated subgrade soils or for select wall backfill materials must meet the following requirements.

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8.3.2 Stabilizing Fill

Stabilizing fill would be used to stabilize soft subgrade conditions or where structural fill is required below a level one foot above the water table at the time of construction. Stabilizing fill should consist of a mixture of coarse gravels and cobbles or a gap-graded, angular, 1.5 to 2.0-inch-minus gravel. Stabilizing fill, if utilized, should be end-dumped, spread to a maximum loose lift thickness of 15.0 inches, and compacted by dropping a backhoe bucket onto the surface at least twice. Subsequent granular backfill borrow placed over coarse stabilizing fills should be adequately compacted so that the finer fraction of the overlying granular backfill borrow is "worked into" the voids in the underlying coarser stabilizing fills.

9. EXISTING UTILITIES

We understand that 12300 South contains numerous underground utilities that pass under the existing culvert. The new culvert will have a base elevation less than one foot lower than the existing culvert. With the exception of over-excavation to install underlying granular backfill borrow, construction of the new culvert may not directly impact those utilities. We understand that the Salt Lake City Department of Public Utilities has clearance requirements for utilities that pass under the Canal and all canal structures.

The maximum base contact pressures generated by the new culvert will be only slightly higher than those generated by the existing culvert. Accordingly, the loads generated by the new culvert are expected to have a minor, if not negligible, effect on the existing utilities.

10. PROFESSIONAL STATEMENTS

Supporting data upon which our discussions and recommendations are based are presented in previous sections of this report. The recommendations presented herein are governed by the physical properties of the soils encountered in the various explorations, projected groundwater conditions, and the layout and design data discussed in Section 1.2, Project Description, of this report. If subsurface conditions other than those described in this report are encountered and/or if design and layout changes are implemented, AMEC must be informed so that our recommendations can be reviewed and amended, if necessary.

Our professional services have been performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices in use locally at this time.

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We appreciate the opportunity of providing this service for you. If you have any questions or require additional information, please do not hesitate to contact us.

Respectfully submitted,

AMEC Earth & Environmental, Inc.

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And

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Professional Engineer

JWG/WJG/JP:ka

Encl.	Figure	1,	Vicinity Map
	Figure	2	
	Figure	3,	Exploration Location Plan
	Appendix A,		Field Explorations and Instrumentation
	Appendix B,		Explorations by Others
	Appendix C,		Laboratory Testing

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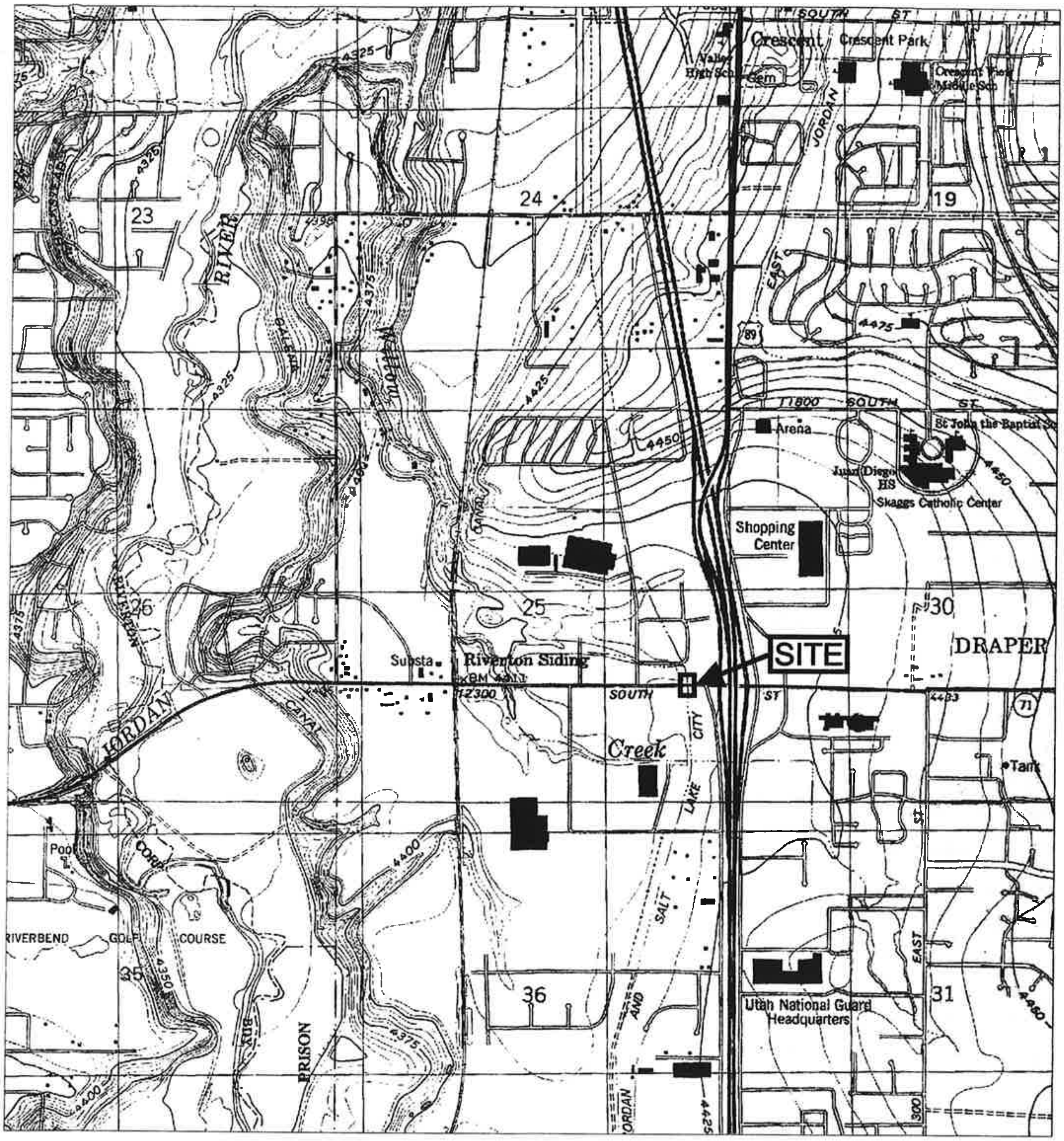
STATE OF UTAH

GRW
12300 SOUTH DESIGN-BUILD PROJECT NO. *HPP-STP-0071(12)0
AMEC JOB NO. 2-817-004066 (4065)

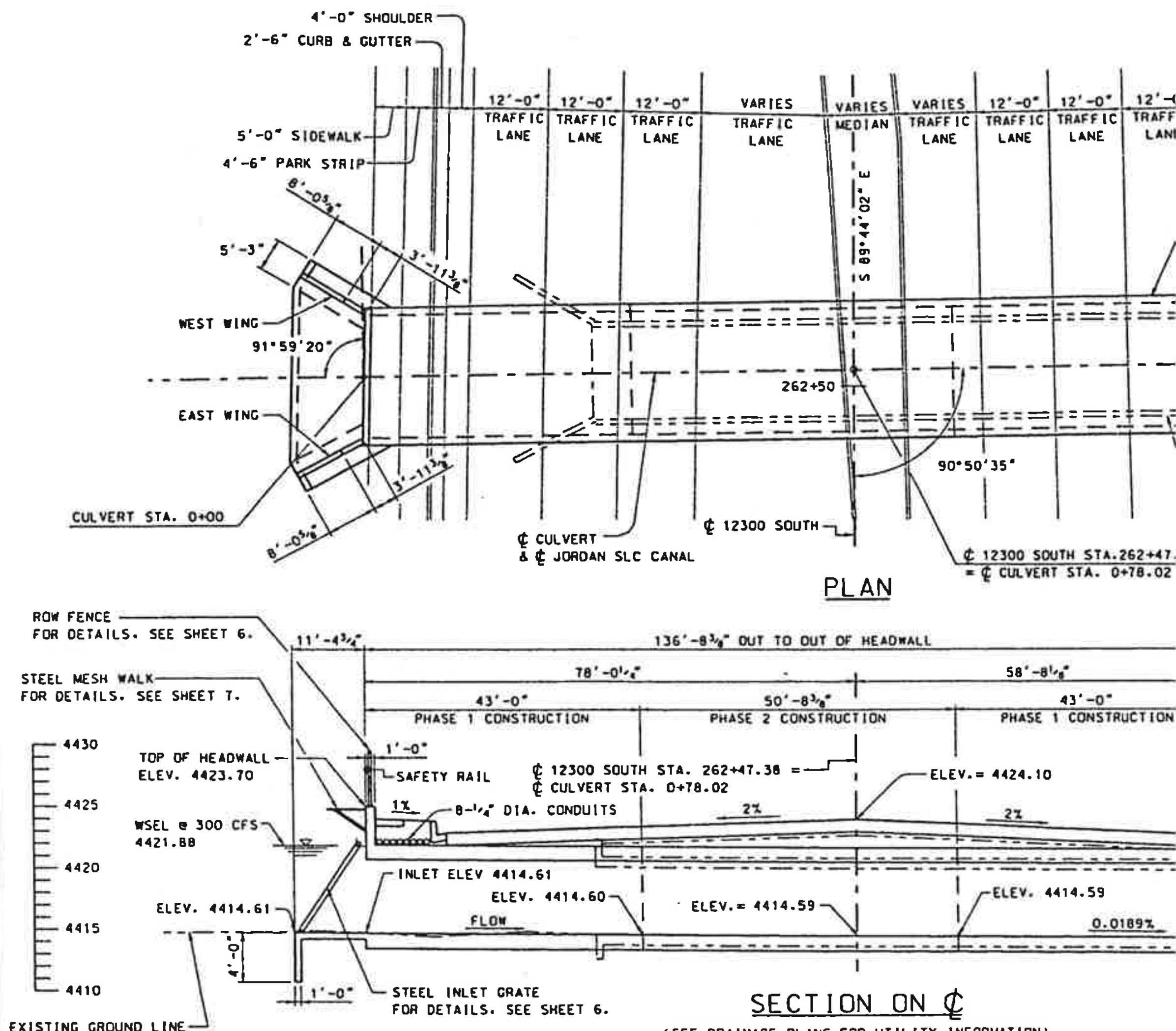


SCALE IN FEET
1000 0 1000 2000

FIGURE 1
VICINITY MAP

REFERENCE:
USGS 7.5 MINUTE TOPOGRAPHIC QUADRANGLE MAP
TITLED "MIDVALE, UTAH"
DATED 1999

GRW
12300 SOUTH DESIGN-BUILD PROJECT NO. *HPP-STP-0071(12)0
AMEC JOB NO. 2-817-004066 (4065)



PLAN

SECTION ON ℄

(SEE DRAINAGE PLANS FOR UTILITY INFORMATION)

GENERAL NOTES

1. ALL DIMENSIONS ARE SHOWN IN FEET AND INCHES. ALL STATIONS AND ELEVATIONS ARE SHOWN IN FEET.
2. PROVIDE 2" COVER TO REINFORCING STEEL UNLESS OTHERWISE NOTED.
3. ALL REINFORCING STEEL SHALL BE COATED, DEFORMED BILLET-STEEL BARS CONFORMING TO AASHTO M-31, GRADE 60.
4. CHAMFER EXPOSED CONCRETE CORNERS ¾ INCH EXCEPT WHERE NOTED OTHERWISE.
5. ALL CAST-IN-PLACE CONCRETE SHALL BE CLASS AA (AE).
6. ALL STRUCTURAL STEEL SHALL CONFORM TO AASHTO M-270, GRADE 36.
7. FOR CONSTRUCTION PHASING, SEE MAINTENANCE OF TRAFFIC PLANS.
8. CULVERT FLOWLINE ELEVATIONS ARE BASED ON SURVEY INFORMATION. FIELD VERIFY THE FLOWLINE ELEVATION FOR THE EXISTING CULVERT.
9. THE LOCATION OF THE EXISTING CULVERT IS BASED ON SURVEY INFORMATION. FIELD VERIFY THE LOCATION OF THE EXISTING CULVERT.

PRELIMINARY
NOT FOR CONSTRUCTION

DESIGN DATA

HL-93 LOADING IN ACCORDANCE WITH CURRENT AASHTO LRFD SPECIFICATIONS.

CAST-IN-PLACE CONCRETE:
f'c = 3.65 KSI
FY (REINFORCEMENT) = 60 KSI

DESIGN DEPTH OF FILL = 2.34 FT.(MAX.); 1.03 FT. (MIN.)

MAXIMUM ALLOWABLE BEARING PRESSURE = 3400 PSF

DESIGN SPEED: 50 MPH 12300 SOUTH

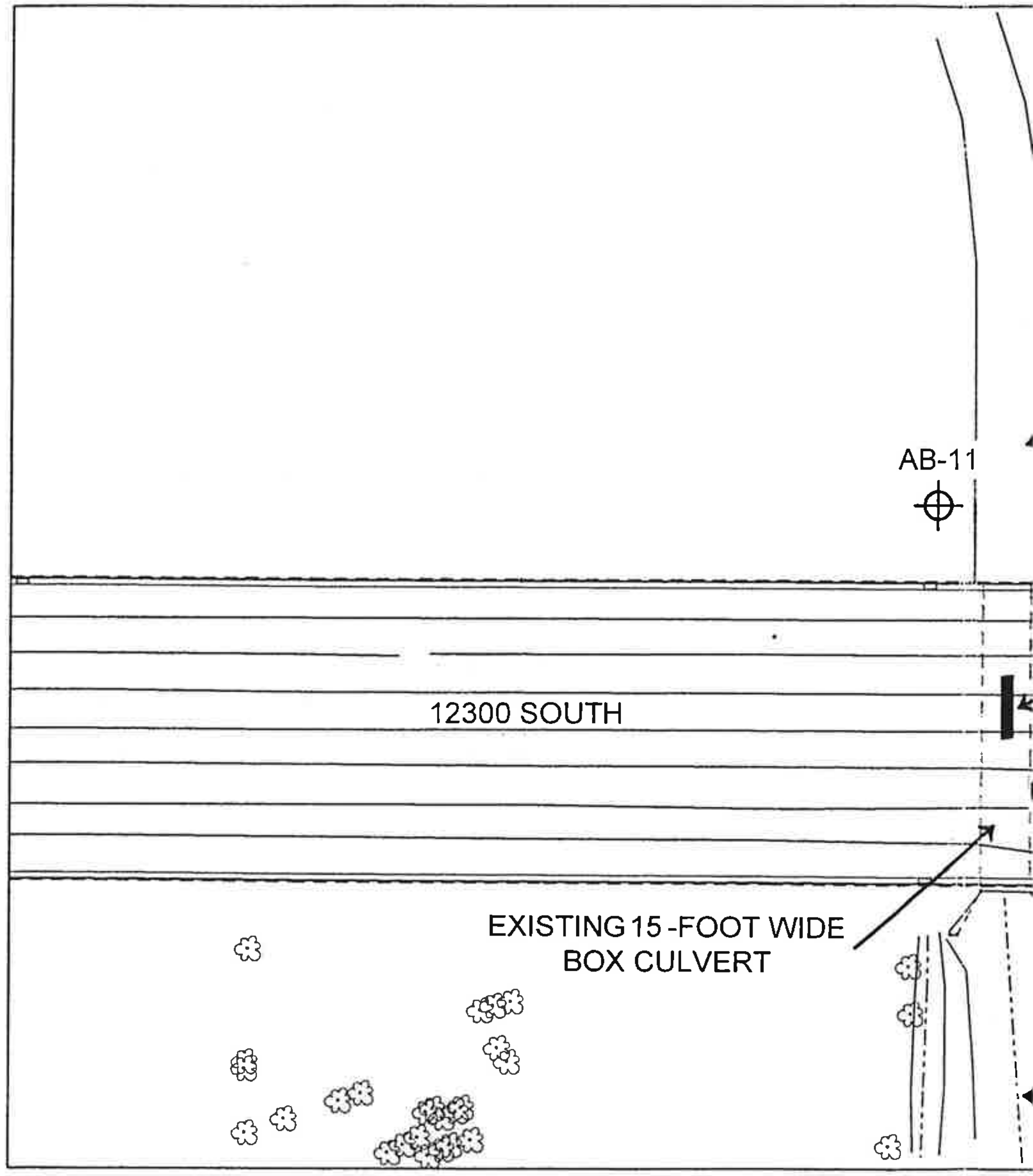
HYDRAULIC DATA

DRAINAGE AREA = NA
DESIGN FREQUENCY = NA
VELOCITY AT OUTLET = 1.51 FOR 150 CFS, 2.50 FOR 300 CF
100 YEAR DISCHARGE = NA
OVERTOPPING DISCHARGE = 300 CFS
FREQUENCY FOR OVERTOPPING FLOW = NA
WATER SURFACE ELEVATION FOR OVERTOPPING FLOW = 4421.88

REFERENCE:
DRAWING PROVIDED
BY UDOT

NO SCALE

N068544



KEY

AB-11	APPROXIMATE AMEC BORING LOCATION
UDOT-1	APPROXIMATE LOCATION PREVIOUS BORING

REFERENCE:
ADAPTED FROM DRAWING
PROVIDED BY UDOT

N068544

APPENDIX A

Field Explorations and Instrumentation

APPENDIX A
FIELD EXPLORATIONS AND INSTRUMENTATION

1. FIELD EXPLORATIONS

Subsurface soil and groundwater conditions at the site were explored by drilling one boring, AB-11, to a depth of about 30.5 feet below existing grades. L & L Drilling, under subcontract to AMEC, advanced the boring using a truck-mounted, Diedrich D120 drill rig equipped with hollow-stem augers. The approximate location of the boring is indicated on Figure 3, Exploration Location Plan.

The field portion of our study was under the direct control and continual supervision of an experienced member of our geotechnical staff. Our representative coordinated and monitored the drilling activities and the installation of the piezometer in Boring AB-11. Our representative maintained a continuous log of the subsurface conditions encountered at the exploration location and obtained representative samples of the soils encountered in the exploration for subsequent laboratory testing and examination.

Relatively undisturbed samples and occasional disturbed samples of the soils encountered in the borings were obtained at 2.5- or 5.0-foot intervals. Relatively undisturbed soil samples were obtained using a D&M split-barrel sampler of the type illustrated on Figure A-1.

The sampler was driven into the undisturbed soil ahead of the auger bit with a 140-lb, automatic drop hammer falling a distance of 30 inches. The number of blows required to drive the sampler for the final foot of soil penetration, or part thereof, is noted on the boring log adjacent to the appropriate sample notation.

The soils encountered in the boring were classified in the field based upon visual and textural examination. These classifications were supplemented by subsequent examination and testing in our laboratory. Soils were classified in general accordance with ASTM D 2488, *Standard Recommended Practice for Description of Soils (Visual-Manual Procedure).* Detailed graphical representation of the subsurface conditions encountered at the boring location is presented on Figure A-2, Log of Test Boring. The exploration log represents our interpretation of the field log and the results of our laboratory classification testing. Figure A-3, Unified Soil Classification System, provides a key to the soil descriptions on the log.

The exploration was located in the field by hand taping or pacing from existing physical features. The ground surface elevation shown on the log was determined by superimposing the exploration location on a map of site topographic contours provided by UDOT in the Project RFP. The exploration location indicated on Figure 3 and the ground surface elevation indicated on the boring log should be considered approximate.

2. INSTRUMENTATION

2.1 PIEZOMETER

Following completion of drilling operations, one and one-quarter-inch diameter slotted PVC pipe was installed to a depth of about 20 feet in Boring AB-11 to provide a means of monitoring groundwater fluctuations.

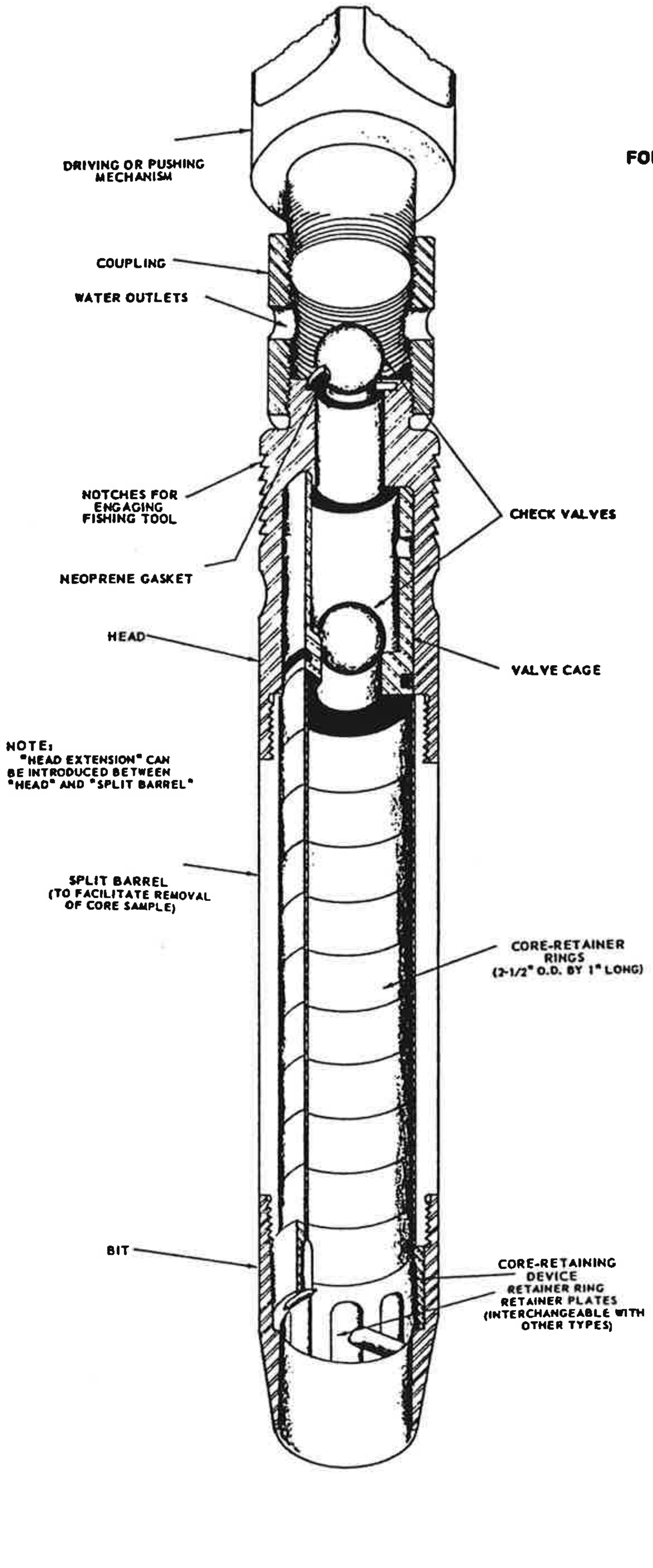
Stabilized groundwater levels were measured on three separate dates after installation. The results of the groundwater level measurements are summarized in the following table along with the corresponding approximate groundwater elevations (based on available ground surface elevation data provided in the RFP).

Stabilized Groundwater Levels

Date	Depth below ground surface (ft)	Approximate Elevation (ft)
August 8, 2002	8.2	4414
September 7, 2002	8.8	4413
October 22, 2002	9.2	4413

SOIL SAMPLER TYPE U

FOR SOILS DIFFICULT TO RETAIN IN SAMPLER



ALTERNATE ATTACHMENTS

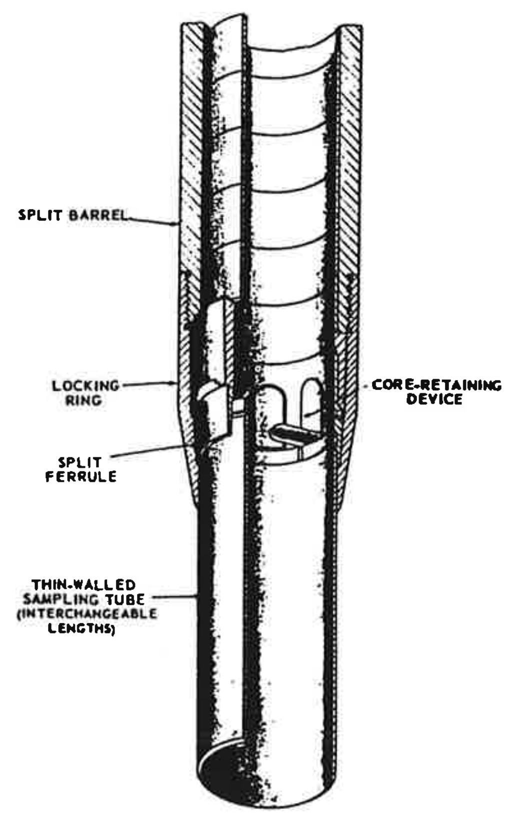


FIGURE A-1

PROJECT 12300 South Design-Build Project
Jordan and Salt Lake Canal, Draper, UT

LOG OF TEST BORING NO AB-11

JOB NO. 2-817-004066 DATE 08-05-02

RIG TYPE Diedrich D120 (L&L)
BORING TYPE 4-1/4" ID Hollow-Stem Auger
SURFACE ELEV. 4422'+/-
LOGGED BY Matt Gallegos

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows/foot 140 lb. 30" free-fall drop hammer	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0								SM/ GM FILL	slightly moist, medium dense	**SILTY SAND AND GRAVEL**; fine to coarse sand; fine and coarse gravel; light brown; FILL
				D	30					
5										grades with clayey silt and fine sand layers to 2" thick
				D	9				moist, very loose	
								CL/ ML	very moist to saturated, stiff	**SILTY CLAY/CLAYEY SILT** with trace fine sand; brown
				D	15	99	24.0			grades with some fine sand; gray to brown-gray with rust-brown mottles
10										
				D	31	101	22.2		very stiff	grades with occasional silty fine sand seams and layers to 1/4" thick; red- to rust-brown
15										
								ML	saturated, medium dense	**SILT** with fine sand and occasional clean to silty sand seams and layers to 1/4" thick; fine to medium sand; brown
				D	28		32.9			
20										
				D	11		32.1		loose	grades to interlayered sandy silt, clayey silt, and silty
25										

GROUNDWATER

	DEPTH	HOUR	DATE
▽	8.2		08-08-02
▼	9.4		10-22-02

SAMPLE TYPE

A - Auger cuttings
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.
D - 3 1/4" O.D. 2.42" I.D. tube sample.
C - California Split Spoon Sample

FIGURE A-3

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PROJECT 12300 South Design-Build Project
Jordan and Salt Lake Canal, Draper, UT

LOG OF TEST BORING NO AB-11

JOB NO. 2-817-004066 DATE 08-05-02

RIG TYPE Diedrich D120 (L&L)
BORING TYPE 4-1/4" ID Hollow-Stem Auger
SURFACE ELEV. 4422'+/-
LOGGED BY Matt Gallegos

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows/foot 140 lb. 30" free-fall drop hammer	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
25								CL/ ML		fine sand; layers to 1/4" thick; gray and yellow-brown
									saturated, medium stiff	**LAYERED SILTY CLAY AND CLAYEY SILT** with sand and occasional silty fine sand seams and layers to 1/4"; dark gray to gray
				D	11	82	39.9			grades with numerous silty fine sand seams
30										
										Stopped drilling at 29.0'. Stopped sampling at 30.5'. Installed 1-1/4" diameter slotted PVC pipe to 20.0'.
35										
40										
45										
										The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary to a proper understanding of the nature of the subsurface materials.
50										

GROUNDWATER

	DEPTH	HOUR	DATE
▽	8.2		08-08-02
▼	9.4		10-22-02

SAMPLE TYPE

A - Auger cuttings
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.
D - 3 1/4" O.D. 2.42" I.D. tube sample.
C - California Split Spoon Sample

FIGURE A-3
(con't)

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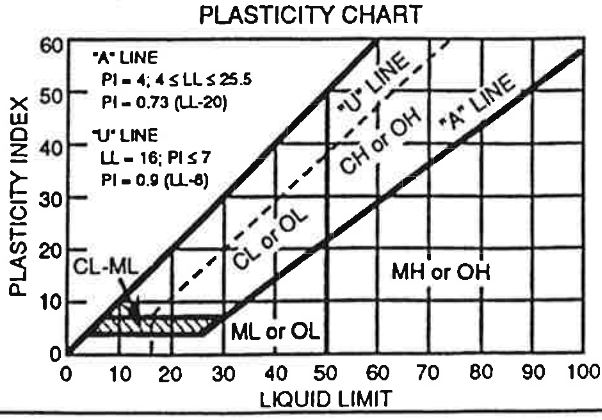
UNIFIED SOIL CLASSIFICATION SYSTEM

Soils are visually classified for engineering purposes by the Unified Soil Classification System. Grain-size analyses and Atterberg Limits tests often are performed on selected samples to aid in classification. The classification system is briefly outlined on this chart. Graphic symbols are used on boring logs presented in this report. For a more detailed description of the system, see "Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)" ASTM Designation: 2488-84 and "Standard Test Method for Classification of Soils for Engineering Purposes" ASTM Designation: 2487-85.

MAJOR DIVISIONS				GRAPHIC SYMBOL	GROUP SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (Less than 50% passes No. 200 sieve)	GRAVELS (50% or less of coarse fraction passes No. 4 sieve)	CLEAN GRAVELS (Less than 5% passes No. 200 sieve)			GW	Well graded gravels, gravel-sand mixtures, or sand-gravel-cobble mixtures
					GP	Poorly graded gravels, gravel-sand mixtures, or sand-gravel-cobble mixtures
		GRAVELS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart		GM	Silty gravels, gravel-sand-silt mixtures
			Limits plot above "A" line & hatched zone on plasticity chart		GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS (50% or more of coarse fraction passes No. 4 sieve)	CLEAN SANDS (Less than 5% passes No. 200 sieve)			SW	Well graded sands, gravelly sands
					SP	Poorly graded sands, gravelly sands
		SANDS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart		SM	Silty sands, sand-silt mixtures
			Limits plot above "A" line & hatched zone on plasticity chart		SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (50% or more passes No. 200 sieve)	SILTS Limits plot below "A" line & hatched zone on plasticity chart	SILTS OF LOW PLASTICITY (Liquid Limit less than 50)			ML	Inorganic silts, clayey silts of low to medium plasticity
		SILTS OF HIGH PLASTICITY (Liquid Limit 50 or more)			MH	Inorganic silts, micaceous or diatomaceous silty soils, elastic silts
	CLAYS Limits plot above "A" line & hatched zone on plasticity chart	CLAYS OF LOW PLASTICITY (Liquid Limit less than 50)			CL	Inorganic clays of low to medium plasticity, gravelly, sandy, and silty clays
		CLAYS OF HIGH PLASTICITY (Liquid Limit 50 or more)			CH	Inorganic clays of high plasticity, fat clays, sandy clays of high plasticity
	ORGANIC SILTS AND CLAYS	ORGANIC SILTS AND CLAYS OF LOW PLASTICITY (Liquid Limit less than 50)			OL	Organic silts and clays of low to medium plasticity, sandy organic silts and clays
		ORGANIC SILTS AND CLAYS OF HIGH PLASTICITY (Liquid Limit 50 or more)			OH	Organic silts and clays of high plasticity, sandy organic silts and clays
ORGANIC SOILS		PRIMARILY ORGANIC MATTER (dark in color and organic odor)			PT	Peat

NOTE: Coarse-grained soils with between 5% and 12% passing the No. 200 sieve and fine-grained soils with limits plotting in the hatched zone on the plasticity chart have dual classifications.

PLASTICITY CHART



DEFINITION OF SOIL FRACTIONS

SOIL COMPONENT	PARTICLE SIZE RANGE
Boulders	Above 12 in.
Cobbles	12 in. to 3 in.
Gravel	3 in. to No. 4 sieve
Coarse gravel	3 in. to 3/4 in.
Fine gravel	3/4 in. to No. 4 sieve
Sand	No. 4 to No. 200 sieve
Coarse sand	No. 4 to No. 10 sieve
Medium sand	No. 10 to No. 40 sieve
Fine sand	No. 40 to No. 200 sieve
Fines (silt and clay)	Less than No. 200 sieve

FIGURE A-4

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APPENDIX B

Explorations by Others

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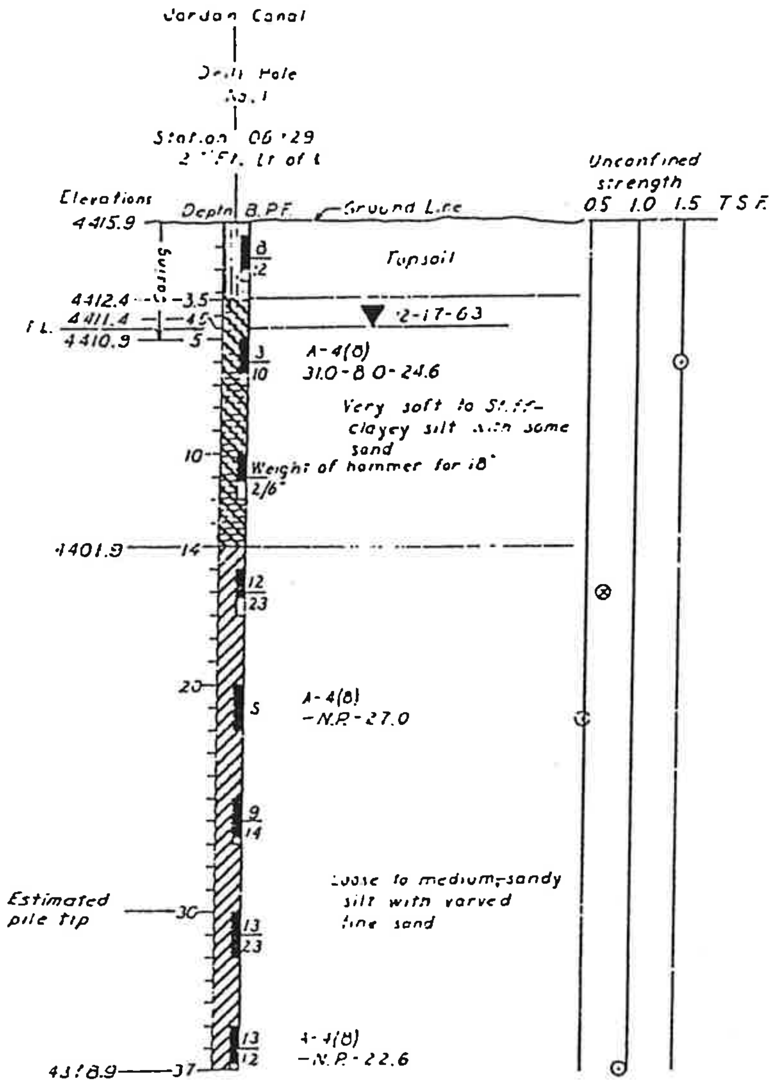
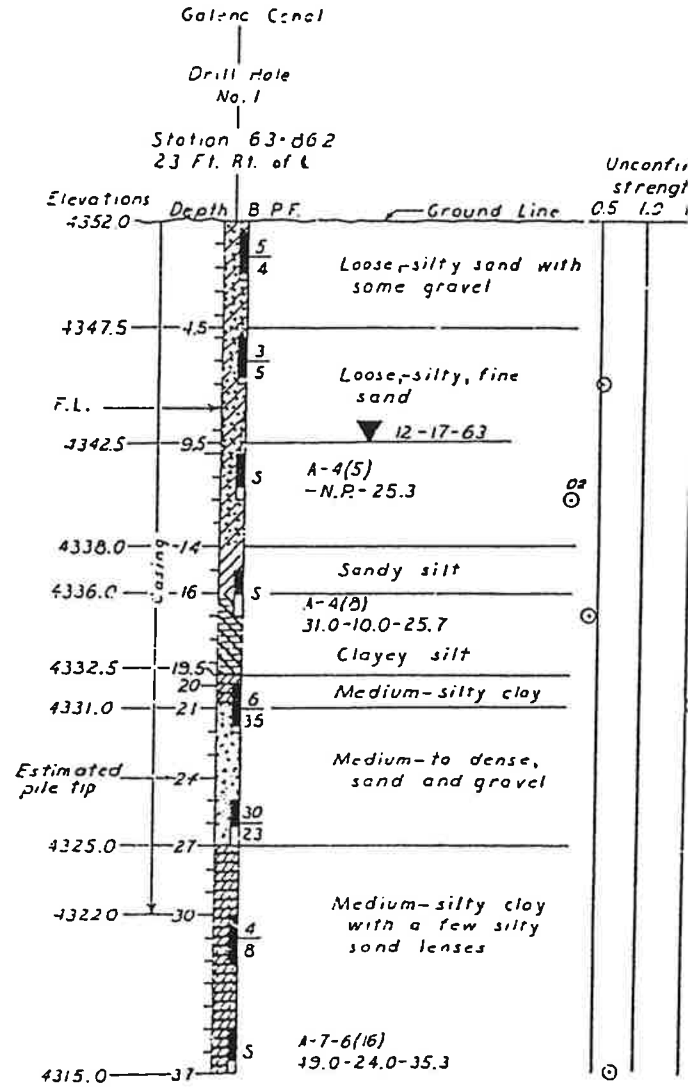
APPENDIX B
EXPLORATIONS BY OTHERS

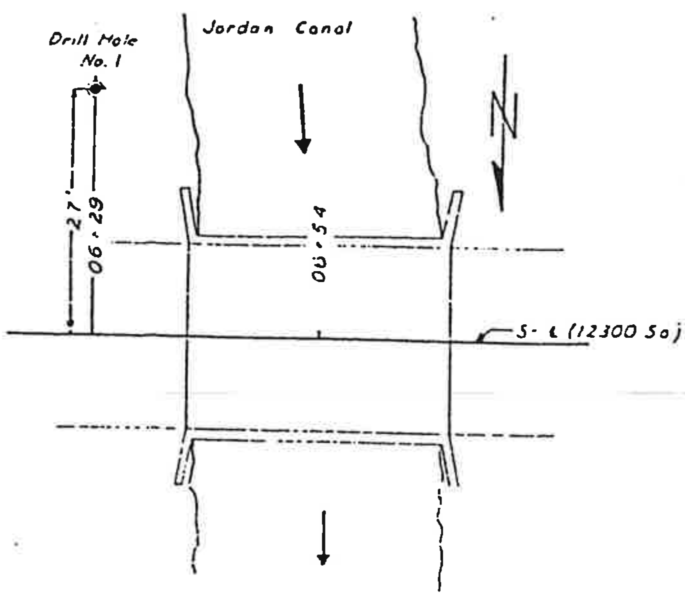
This appendix provides the log of an exploration conducted at the site in the 1960's by the Utah State Department of Highways. The approximate location of the exploration, labeled UDOT-1, is shown on Figure 3, Exploration Location Plan. This exploration log is provided for reference only.



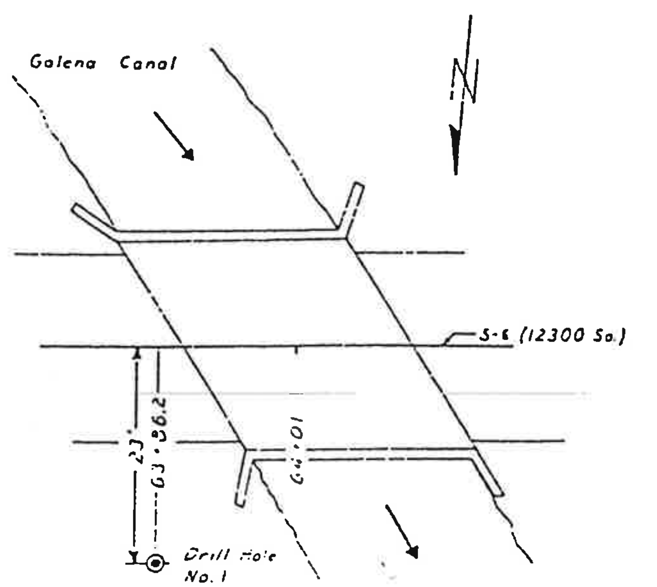
Jordan Canal
Drill Hole No. 1
Elevations
Depth
B.P.F.
Ground Line
Unconfined strength
0.5 1.0 1.5 T.S.F.
4415.9
Casing
Topsoil
4412.4
4411.4
4410.9
F.L.
A-4(8)
31.0-8.0-24.6
Very soft to Stiff-clayey silt with some sand
Weight of hammer for 18"
2/6"
4401.9
A-4(8)
-N.P.-27.0
Estimated pile tip
Loose to medium-sandy silt with varved fine sand
A-4(8)
-N.P.-22.6



Galena Canal
Drill Hole No. 1
Station 63+86.2
23 Ft. Rt. of ℄
Elevations
Depth
B.P.F.
Ground Line
4352.0
Loose-silty sand with some gravel
4347.5
Loose-silty, fine sand
F.L.
12-17-63
4342.5
A-4(5)
-N.P.-25.3
4338.0
Sandy silt
4336.0
Casing
A-4(8)
31.0-10.0-25.7
4332.5
Clayey silt
Medium-silty clay
4331.0
Medium-to dense, sand and gravel
Estimated pile tip
4325.0
Medium-silty clay with a few silty sand lenses
4322.0
A-7-6(16)
49.0-24.0-35.3
4315.0



Drill Hole No. 1
Jordan Canal
27'
S-℄ (12300 So.)



Galena Canal
S-℄ (12300 So.)
23'
63+86.2
Drill Hole No. 1

APPENDIX C

Laboratory Testing

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APPENDIX C
LABORATORY TESTING

1. GENERAL

Laboratory tests were performed on representative samples of the soil encountered in Boring AB-11 to evaluate pertinent physical characteristics, aid in classifying the soils, and correlate other test data. The laboratory program included sample inspection to confirm AMEC's field soil descriptions and classification testing to determine natural moisture content, in-situ soil density, Atterberg limits, and grain-size distribution.

Identification of the test procedures and summaries of selected test results are presented in the following sections of this appendix. An overall summary of selected test results is presented in Table C-1.

2. CLASSIFICATION TESTS

2.1 MOISTURE AND DENSITY TESTS

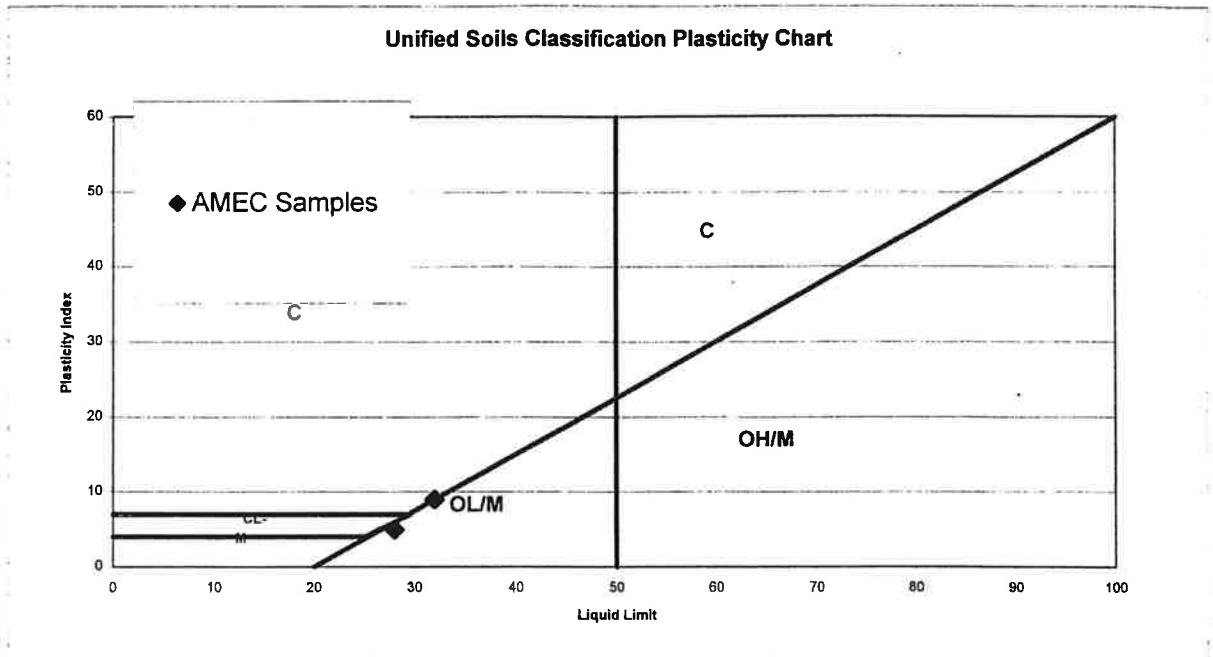
Determination of natural moisture content was performed in general accordance with ASTM D-2216 test procedures. Determination of the in-situ dry density of selected, relatively undisturbed samples was performed in general accordance with ASTM D-4564 test procedures. Natural moisture content and dry density, where determined, are presented adjacent to the corresponding sample notation on the boring log included in Appendix A.

2.2 ATTERBERG LIMITS TESTS

Determination of the Atterberg limits of selected samples was performed in general accordance with ASTM D-4318. Results from Atterberg limits testing are summarized in the following table and plasticity chart, and in Table C-1.

Boring No.	Sample Depth (ft)	Unified Soil Classification System Group Symbol*	In situ Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
AB-11	9.5	ML	22.8	28	23	5
AB-11	14.5	ML/CL	20.7	32	23	9

* Based upon portion of the sample passing the No. 40 sieve.



2.3 PARTIAL GRAIN SIZE ANALYSES

Partial grain-size analyses were performed in general accordance with ASTM C-117, C-136, and D-1140 test procedures. Partial grain-size analyses were performed to determine the fines content (percent material by weight passing the No. 200 sieve). Results from the grain-size analyses are summarized in the following table and are included in Table C-1.

Boring No.	Sample Depth (ft)	Percent Passing by Weight Passing the No. 200 Sieve	Unified Soil Classification System Group Symbol
AB-11	19.5	95	ML
AB-11	24.5	91	CL/ML

Table C-1

12300 South
Laboratory Testing Summary Sheet
12300 South Culvert Crossings
11/22/02

Boring No.	Sample No.	Sample Depth (ft)	Moisture Content (%)	Dry Density (pcf)	Moist Density (pcf)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	-200 (%)	AASHTO Classification
AB-11	1									
	2									
	3	9.5	24.0	98.6	122.2	28	23	5		A-4
	4	14.5	22.2	101.4	123.9	32	23	9		A-4
	5	19.5	32.9						95.0	A-4
	6	24.5	32.1						91.3	A-4
	7	29.5	39.9	81.6	114.2					