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ADDENDUM #1

Date:	3 March 2008
To:	Interested Parties
From:	Dave McKay, Project Manager
Reference:	Utah State University - USTAR Life Science Research Center - Design DFCM Project No. 06292770
Subject:	**Addendum No. 1**

Pages	Addendum	1	page
	Geotechnical Engineering Report	51	pages
	Gramoll Construction CM/GC Schedule	1	page
	Total	**53**	**pages**

Note: This Addendum shall be included as part of the Contract Documents. Items in this Addendum apply to all drawings and specification sections whether referenced or not involving the portion of the work added, deleted, modified, or otherwise addressed in the Addendum.

1.1 SCHEDULE CHANGES – There are no changes to the Project Schedule.

1.2 General

See attached Geotechnical Engineering Report.
See attached schedule. This is a tentative schedule prepared by Gramoll Construction. It is not a set schedule, but please be prepared to respond to questions regarding this schedule and possible actions as a team member during the coming interviews.

End of Addendum #1

GEOTECHNICAL ENGINEERING REPORT
USU USTAR RESEARCH INSTITUTE
1600 NORTH 600 EAST
LOGAN, UTAH

Submitted To:

Ms. Jill Jones
AJC Architects
703 East 1700 South
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Submitted By:

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February 1, 2008
Project No. 7-817-005223

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FIGURE 1 VICINITY MAP
FIGURE 2 SITE PLAN
FIGURE 3A-3H LOG OF BORINGS
FIGURE 4 SOIL CLASSIFICATION CHART & LEGEND
FIGURE 5 FOOTING BEARING PRESSURE CHART

APPENDIX A FIELD EXPLORATIONS-BORINGS
APPENDIX B LABORATORY TESTING
APPENDIX C CONE PENETRATION TESTING

February 1, 2008

Ms. Jill Jones
AJC Architects
703 East 1700 South
Salt Lake City, UT 84105

Re: **Geotechnical Study Report**
USU USTAR Research Institute
1600 North 600 East
Logan, Utah
AMEC Job No. 7-817-005223

1. INTRODUCTION

1.1 Objectives and Scope

This report presents the results of our geotechnical study for the proposed Utah State University USTAR Research Institute building. The location of the planned project is approximately 1600 North 600 East Street in Logan, Utah. The approximate location of the site is shown on Figure 1, Vicinity Map. The objectives of this investigation were to explore and evaluate subsurface materials and conditions and develop recommendations for the design and construction of the new building. The studies were conducted in accordance with the scope of work outlined in AMEC's proposal PL07-086 dated October 17, 2007 and a scope change letter, dated December 4, 2007. AMEC's scope of work included a site reconnaissance, field explorations, laboratory testing, engineering analyses, and report preparation.

2. PROJECT DESCRIPTION

We understand the proposed construction will consist of a three story above grade steel and concrete building. The building will be "L" shaped and will have a footprint of approximately 33,000 sf. We anticipate maximum column loads to be on the order of 750 to 1,000 kips. Areas surrounding the building will be landscaped and parking areas will be included. We anticipate that traffic in the parking areas will consist of a light volume of automobiles and light trucks, and occasional medium-weight trucks.

3. SITE DESCRIPTION

3.1 Site Conditions

The majority of the project site is situated on land that has been primarily used for agriculture. There are several existing structures such as stables and hay covers on the east of the site, which will be removed for the project, and an above grade storm water detention basin is located in the northwest corner of the site. The site is located within the Utah State University Research Park and is bordered on the north by 1600 North, on the south by buildings and pastures, on the east by an adjacent Utah State research building, and on the east by stables and hay covers. The site is relatively flat with a slope down to the west. The approximate elevation of the site is 4,580 feet above sea level.

3.2 Geology

The project site is located in Cache Valley Utah near the eastern edge of the Basin and Range physiographic province, which extends from the Sierra Nevada Mountains to the Wasatch Mountains. The Basin and Range province is characterized by north-trending mountain ranges and intervening sediment-filled valleys. The mountain ranges are bounded by high-angle normal faults formed in response to regional extension of the earth's crust. A geologic map prepared by Dover, 1995[1] indicates that the site is underlain by alluvial and lacustrine deposits placed during the Provo Stage of Lake Bonneville. Soils consists of silt, clay, sand, and gravel to depths of approximately 50 to 75 feet.

4. FIELD EXPLORATIONS & LABORATORY TESTING

4.1 Field Explorations

Subsurface materials and conditions at the project site were investigated on October 25, 2007 with 8 borings designated B-1 through B-8 and on January 18, 2008 with 2 cone penetration tests designated CPT-1 and CPT-2. The approximate locations of the borings and cones are shown on Figure 2, Site Plan. All field operations were observed by a technician provided by our firm, who maintained a detailed log of the materials and conditions encountered in each bore hole and directed the sampling and testing operations. Additional information on the field exploration is presented in Appendix A, Field Explorations-Borings and Appendix C, Cone Penetration Testing.

4.2 Laboratory Testing

Laboratory testing consisted of natural moisture content, gradations, fines washes, Atterberg limits, consolidation, and corrosion testing. Details concerning the tests and the laboratory results can be found in Appendix B, Laboratory Testing.

5. SUBSURFACE CONDITIONS

5.1 Fill and Disturbed Soil Conditions

Subsurface investigations encountered agriculturally disturbed surface soils over the majority of the site extending down approximately 1 foot below grade. In addition to these disturbed soils, 3 feet of fill was present over the eastern part of the site. The top of fill is 3 feet above adjacent agricultural land and appears to have been placed during the construction of adjacent structures. This fill contains debris such as concrete blocks. At the northeast corner, there is a 6-foot high berm, which encloses a storm water detention area.

[1] Dover, J.H.; 1995; Geologic Map of the Logan 30' x 60' Quadrangle, Cache and Rich Counties, and Lincoln and Uinta Counties, Wyoming; U.S. Geological Survey Miscellaneous Publication MAP I-2210, Scale 1:100,000

5.2 Geotechnical Profile

Logs of the borings B-1 through B-8 are presented on Figures 3A through 3H, Log of Borings. The terms used to describe the soils disclosed by the borings are defined on Figure 4, Soil Classification Chart & Legend. Cone Penetration Results can be found in Appendix C.

The native soil profile is comprised of surficial lean clay underlain by silty sand with gravel and layered silts and clays. Cone penetration tests indicate upper clay is underlain by a 5-feet layer of sand and gravel followed by 20 to 30 feet of inter-layered silt, silty clay, and silty sand. Underlying this layer of silt and sand is 7 to 10 feet layer of clay, followed by at least a 12-foot layer of dense sand and gravel. This layer of sand and gravel was encountered at 40 feet in CPT-1 and 48 feet in CPT-2. Underlying this dense layer of sand and gravel is silt and clay followed by inter-layered sand and silt.

Liquid limits on tested samples typically ranged from 23 to 35, and plasticity indices ranged from 3 to 14. One sample from boring B-3 at a depth of 14 feet was non-plastic. Dry densities ranged from 106.6 to 109.5 pcf with moisture contents ranging from 20.3 to 22.8 percent.

5.3 Groundwater

At the time of the boring investigation, groundwater was encountered at depths ranging from 6.0 to 8.5 feet below grade in borings B-2 and B-8. Subsequent measurements more than a week later indicate groundwater at depths ranging from 7.7 to 9.5 feet. Additionally, equilibrium pore pressures measured in sand layers at 40 feet in CPT-1 and 52 feet in CPT-2 indicate pressures equivalent to a water table at approximately 20 feet. Fluctuations in groundwater and perched groundwater do occur due to variations in precipitation, runoff, water levels in nearby ditches, drainages and other factors. Longer-term groundwater fluctuations should be anticipated with the highest seasonal levels generally occurring during the late spring and summer months.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

The site is generally unfavorable to the support of the proposed building on shallow foundations due to heavy building loads. It is anticipated that these heavy loads will require the use of alternative foundation systems such as a mat foundation, rammed aggregate piers, driven piles, or auger cast piles.

For structures associated with the project where shallow spread footings can be used, it is recommended that the footings be established upon structural fill extending down to suitable undisturbed native soils. Flatwork (inside and outside) may be established upon properly prepared native soils, and/or upon structural fill extending down to suitable undisturbed native soils.

Undocumented fills are often poorly compacted and contain deleterious material within their matrix. It is our experience that undocumented fills have an increased risk of total and differential settlements, which can lead to poor performance of foundations and pavements. Existing undocumented fill in the east of the site should be completely removed from beneath the building footprint and pavement areas. If existing site fill meets structural fill requirements, it may be reused on site as structural fill. Excavated native soils may be placed in landscaped areas, but should not be used as structural fill.

Agriculturally disturbed soil must be improved through scarification and re-compaction or removed entirely from below structures and parking areas. Compaction of these soils should meet the same standards as the compaction standards for structural fill.

Fine grained soils encountered on site are prone to moisture sensitivity and are easily disturbed and softened by construction equipment.

Considering the depth of shallow footings with respect to the water table, the water table could be an issue during the construction of the project. Soils at the bottom of footing excavations may be soft and wet and easily disturbed. Although we do not anticipate shallow footings or mat foundations to be below the water table, dewatering and soil stabilization may be necessary, if these conditions occur.

Liquefaction settlement is a concern with the site. Measures can be taken to improve structural connections, improve site soils, or use deep foundations as dictated by the risk the client is willing to accept. Mat foundations can cope with additional uniform settlements, but may have some problems if significant differential settlement occurs. Rammed aggregate piers can reduce the overall liquefaction settlement, while driven piers and auger cast piles can bypass liquefiable layers to bear on deep stable soils. All alternative foundation systems offer improvements over the basic shallow spread footing foundation with respect to liquefaction settlement, but the selection of a foundation system is related to acceptable cost and the risk that the client is willing to accept.

If grades are to be raised more than approximately 3 feet above existing grade, our office should be contacted for further engineering analyses. Thick areal fill can often induce significant settlement over time as underlying layers of soft saturated clays and silts consolidate under the weight of the fill. Further analysis will be needed to determine settlement and its affects on any structure if fills exceed 3 feet.

Subgrade pavement characteristics indicate fair support characteristics. Pavement sections include 4 inches of base course over 8 inches of granular borrow materials as subbase improvement. Asphalt thickness options range from 3 to 4 inches of asphaltic concrete, depending on expected traffic loads.

An alternative to shallow foundations is to establish the foundation upon a rammed aggregate pier. Rammed aggregate piers are a proprietary foundation system developed by the Geopier Foundation Company (Geopier^{R}). They are constructed by drilling a 24 or 30 inch diameter hole, removing a volume of soil, and then building a bottom bulb of clean, open-graded stone using beveled, high-energy tamper. The Geopier shaft is constructed on top of the bottom bulb using well-graded highway base course stone placed in lifts (12-inches compacted thickness). Geopier shaft lengths typically range between 8 and 25-feet as measured from the footing subgrade. The result of construction is a reinforced zone of soil directly under footings that allows for the construction of shallow spread footings proportioned for a relatively high bearing pressure.

Geopier-reinforced soils improve the subgrade below conventional spread footings, reduce the compressibility of underlying soil, allow for a higher bearing pressure to be used for design, and can often reduce liquefaction settlement potential. If serious consideration is given to a rammed aggregate pier foundation, then our office should be contacted for additional information after which Geopiers will need to be contacted to offer preliminary design recommendations.

Our analysis indicates driven piles or auger cast piles can bear upon a layer of sand and gravel at approximately 40 to 50 feet below grade.

Considering above factors, several foundation alternatives are considered feasible for the research building, depending on the cost and risk the client is willing to accept. A summary of foundation alternatives along with the potential advantages and disadvantages is presented in the following table.

Foundation Type	Advantages	Disadvantages
Mat Foundations	• Conventional construction • No significant equipment mobilization other than for earthwork • The prepared pad can readily accommodate variations in equipment layouts and future foundations	• High concrete requirement for thick mats. • Structural fill must be imported to the site – should be granular soils • Excavations within two feet of the groundwater table may encounter soft, saturated subgrade conditions – careful excavation and placement will be required. • May be prone to additional settlement and differential settlement from liquefaction
Rammed Aggregate Pier	• Will provide higher bearing capacities for conventional foundation designs • Helps densify surrounding soils as rock lifts are rammed into place. • Can help reduce the potential for liquefaction seismic settlements.	• Moderately expensive • Must mobilize specialty contractor • Expensive for lightly loaded structures
Auger-cast piles	• Relatively high compressive capacities • Less vibrations than driven piles • Can install to significant depths to reduce effects from seismic settlements.	• Moderately expensive • Must mobilize specialty contractor • Expensive for lightly loaded structures • It is difficult to verify proper installation. • Requires large amounts of concrete grout
Steel Pipe Piles	• Relatively high compressive and uplift capacities • Can install to significant depths to reduce effects of seismic settlements • Helps densify soils as they are being driven, which can help with preventing liquefaction events.	• Moderately expensive • Must mobilize specialty contractor • Expensive for lightly loaded structures • It produces significant vibrations that can affect nearby structures. • Susceptible to corrosion from soluble chlorides.

6.2 Earthwork

6.2.1 Site Preparation

Preparation of the site should consist of stripping all fill, debris, vegetation, frozen soils, loose soils, and disturbed soils from the area. Any foundation elements from prior structures should be removed entirely and replaced with structural fill.

For prior agricultural areas, stripping should extend down at least 6 inches below buildings and parking areas. After stripping is complete, the site soils should be scarified down at least 8 inches, moisture conditioned, and re-compacted to the same compaction standard as structural fill. As an alternative to scarification and re-compaction, stripping should extend down to 14 inches below existing grade, after which grades can be raised back up with structural fill. Upon completion of site preparation, the exposed subgrade should be observed by a qualified representative of the geotechnical engineer to assess the result of the stripping and scarification processes.

For areas with several feet of fill, the fill should be entirely removed along with the upper 12 inches of underlying soils. Berm soils associated with the detention area will also need to be removed down to native undisturbed soils.

If pavement areas are not paved closely after preparation of the subgrade, they should be proof-rolled with a heavy pneumatic-tire roller or equivalent rubber-tire construction equipment to verify the subgrade has not been weakened by ponding and infiltration of precipitation. Any soft areas identified by the proof rolling should be removed down to firm native soil or a maximum of two feet below grade and replaced with structural fill.

The site soils are predominately fine-grained. If the fine-grained soil is exposed to significant precipitation, snow melt or other sources of water, it may become slippery and soft, and disturbed by construction traffic. Disturbed and softened soils are unsuitable for support of foundations and pavement and should be removed and replaced with granular structural fill in building and pavement areas. On site soil that may need to be used for backfill or grading fill may become too wet to achieve proper compaction without drying.

The contractor should be aware of these potential difficulties. The risk of problems can be reduced by performing earthwork activities during warmer months. Other precautions may be desirable such as placing gravel working pads, temporary grading to channel run off away from roads, stockpiles and excavations, and covering the stockpile soils.

6.2.2 Excavations

Temporary construction excavations in soils not exceeding 4 feet in depth may be constructed with near-vertical side slopes. Temporary excavation slopes up to 10 feet in height <u>and above the water</u> table may be constructed no steeper than one horizontal to one vertical (1H:1V). If excessive sloughing occurs, the excavation slope should be flattened. Excavations encountering the groundwater table or perched groundwater will require much flatter slopes, shoring and bracing, and/or dewatering. Excavation safety and dewatering is the responsibility of the contractor. All excavations should be constructed in conformance with Federal, State and local regulations. All excavations must be inspected periodically by qualified personnel. If any signs of instability are noted, immediate remedial action must be initiated.

6.2.3 Fill Requirements

Fill material should be free from debris, vegetation, roots, other unsuitable material, frozen material, and excess moisture. Structural fill should also conform to the gradation and plasticity requirements shown in the following table, Fill Material Requirements.

FILL MATERIAL REQUIREMENTS

Fill Name	Type	Application	Max Size in.	Max Percent Passing			Max Liquid Limit	Max Plasticity Index
				No. 4	No. 10	No. 200		
Structural	S1	Below structural elements	4	-	60	30	35	15
Upper Slab	UF	Immediately below slabs, upper 4 inches	2		25	5	-	-
Free Draining	FD	Drainage layers of drainage backfill	4		5	2	-	-

Existing site fill may be reused as structural site grading fill if it meets the requirements of structural fill.

6.2.4 Fill Placement and Compaction Requirements

Structural fill and floor slab fill should be compacted to at least 95 percent of the maximum dry density at a moisture content within about 3 percent of optimum as determined by ASTM D-1557 (modified Proctor). Structural fill should extend out from the edge of footings a distance equal to half the depth of the fills. For example, if the structural fill depth is 4 feet, the fill should extend out at least 2 feet past the outside edge of the footing.

Fill should be placed and compacted in lifts. The lift thickness should be appropriate for the type of equipment being used so that the entire lift thickness is compacted to the required level. With heavy compaction equipment, we recommend that loose lift thickness be limited to a maximum of 12 inches unless specific arrangements are made with the testing entity to verify compaction in thicker lifts. Fill compaction should be tested frequently. The contractor should have sufficient testing early to verify that compaction methods are adequate to meet compaction requirements and regular additional testing to demonstrate consistent compaction.
Where free draining fill is used to collect or drain water, a filter fabric capable of preventing the migration of fines into the free draining fill should be placed between the fill and native soil on all sides.

Fill in landscaped areas should be compacted to a minimum of 85 percent of the maximum dry density as determined by ASTM D-1557.

If pumping of the subgrade occurs when compacting fill, compaction should immediately stop and the geotechnical engineer consulted for appropriate action.

Excess compaction of backfill behind walls can cause significant stresses against walls and should be avoided. The use of moderate to heavy equipment, especially compactors, near walls can also cause significant stresses against walls and should be avoided. Such equipment should not operate within a distance equal to the height of the wall to minimize the potential for excessive lateral pressure. Compaction close to the walls should be accomplished using hand-operated vibratory plate compactors or small trench compactors.

6.2.5 Fill Placement Considerations

In general, we recommend that the contractor be left to determine the most cost effective and practical means to place and compact fill. However, the following information may be helpful.

When performing compaction testing, the measured degree of compaction is only meaningful if gradation of the soil tested in the field corresponds to the gradation of the samples tested in the lab from which the maximum dry density and optimum moisture was determined. The fill material should be sampled and tested in the laboratory at a frequency appropriate for the variability of the fill. For highly variable soils this can be extremely difficult to ensure and there is a significant risk that field testing may not be representative. Additional measures such as limiting lift thickness may be advised.

The maximum particle size should generally be limited to ½ of the compacted lift thickness. Oversize pieces at the lift surface can carry the weight of the compaction equipment resulting in a poorly compacted zone around the oversized particle. Over a relatively firm subgrade, large pieces extending above the surface of the fill can result in a concentrated foundation load and/or thin section of footing.

All compaction equipment has a limited depth of influence. For hand operated equipment such as vibratory plate or "jumping jack" compactors, we recommend that the compacted lift thickness be limited to 4 inches. For small "trench" rollers, moderate sized roller compactors and larger roller compactors we recommend that compacted lift thickness be limited to 6, 8 and 12 inches unless it can be demonstrated that the recommended compaction can be achieved throughout the lift with thicker lifts.

6.2.6 Utility Trenches

It should be noted that utility trench excavations have the potential to degrade the engineering properties of the adjacent fill materials. Utility trench walls that are allowed to move laterally can lead to reduced bearing capacity and increased settlement of adjacent structural elements and overlying slabs. Backfill for utility trenches is as important as the original preparation or structural fill placed to support either a foundation or slab. Therefore, it is imperative that the backfill for utility trenches be placed to meet the project specifications for the structural fill of this project.

Most utility companies and municipalities are now requiring that AASHTO Type A-1 or A-1-a soil (granular soil with less than less than 25 or 15 percent fines, respectively) be used as backfill over utilities. These organizations are also requiring that in public roadways the backfill over major utilities be compacted over the full depth of fill to at least 96 percent of the maximum dry density as determined by the AASHTO T-180 (ASTM D-1557) method of compaction. We recommend that as the major utilities continue onto the site that these compaction specifications are followed.

6.2.7 Finished Grading

Finish grading should be established to convey water away from foundation walls and backfill and to prevent ponding. Down spouts should discharge away from foundation backfill. Irrigation above or near any wall backfill should be minimized. We recommend that landscaped surfaces adjacent to buildings be sloped down away from the buildings at a minimum slope of 6 inches down in the first 10 feet (5 percent) away from buildings. Concrete flatwork or pavement adjacent to buildings should slope down away from the buildings at a slope of 1 percent or more.

6.3 Foundations

6.3.1 Design Criteria

Foundation support for the proposed project can be provided by conventional wall and column-type spread footings provided resulting capacities are sufficient for the required loads. If loads exceed the provided soil capacities, other foundation may be necessary. The following table presents general options for footing design:

DESIGN CRITERIA

Footing Location	Foundation Type	Bearing Soils	Foundation Depth (feet)	Allowable Bearing Capacity (psf)	Max Width (feet)	
					Square Column	Wall
At Grade Level	Spread Foundations	Min. 2' Structural Fill [2]	1.0 [1]	2,500	12	5
		Min 4' Structural Fill [2]	2.5 [1]	3,000	12	5

Notes:

1. Bottom of footing elevation below finished floor. For exterior footings, footings should be at the depth listed in this table, or 2.5' below exterior grade, whichever is deeper.
2. Footings should be founded upon properly compacted structural fill, which has been placed on undisturbed native soil.

In addition to the above table, a footing bearing graph is provided in Figure 5 for square footings underlain by 4 feet of granular structural fill. Selected footings should have widths and bearing pressures below both the allowable bearing pressure line and the 1-inch settlement line. This graph allows for flexibility and optimization of the design. Footing depths are assumed to be at least 2.5 feet below grade for this graph (Figure 5).

Strip (wall) footings should have a minimum footing width of 1½ feet, and square footings should have a minimum footing width of 2 feet in order to maintain bearing capacity. The allowable bearing pressure applies to the total of real loads, i.e., dead load plus frequently and/or permanently applied live loads. The allowable bearing pressure can be increased by one-third for the total of all loads: dead, live, and wind or seismic.

Soft, loose, or otherwise unsuitable soils, if encountered at footing depth, should be removed down to firm subgrade material and replaced with granular structural fill or a lean concrete flowable fill.

6.3.2 Settlements

Settlement of foundations designed and installed in accordance with the above recommendations should not exceed 1 inch.

6.3.3 Installation

Under no circumstances should the footings be installed upon loose or disturbed soil, sod, rubbish, construction debris, topsoil, frozen soil, non-engineered fill, highly expansive clays, other deleterious materials, or within ponded water. If there are unsuitable conditions encountered, the soils must be completely removed and replaced with compacted granular structural fill. If granular soils become loose or disturbed, they must be properly re-compacted before the footings are poured. The width of replacement fill below footings should be equal to the width of the footing plus ½ foot for each foot of fill thickness on either side of the footing. For example, if the width of the footing is 2 feet and the thickness of the structural fill beneath the footing is 2 feet, the width of the structural fill at the base of the footing excavation would be a total of 4 feet.

6.3.4 Lateral Resistance

Lateral loads imposed upon foundations due to wind or seismic forces may be resisted by the development of passive earth pressures and friction between the base of the footings and the supporting soils. In determining frictional resistance, ultimate coefficient of friction values of 0.35 and 0.45 may be utilized for footings established on silt or on granular structural fill, respectively.

Passive resistance provided by properly placed and compacted granular structural fill above the water table may be considered equivalent to a fluid with a density of 300 pounds per cubic foot

(pcf). Below the water table, this granular soil should be considered equivalent to a fluid with a density of 150 pcf.

A combination of passive earth resistance and friction may be utilized provided that the friction component of the total is divided by 1.5.

6.4 Mat Foundations

6.4.1 Mat Design

The site is generally favorable to supporting the proposed building on mat foundations. A k_1 modulus of subgrade reaction of 100 pci can be used for design. This value represents an estimate of the modulus of subgrade reaction for a 1 x 1 plate at the site. The value should be adjusted for the larger areas associated with mats using the following expression for cohesive and cohessionless soil:

Modulus of Subgrade Reaction, $k_s = (\frac{k_1}{B})$ for Cohesive Soil

$k_s = k_1 (\frac{B+1}{2B})^2$ for Cohessionless Soil

Where:
- k_s = Coefficient of Vertical Subgrade Reaction for Loaded Area
- k_1 = Coefficient of Vertical Subgrade Reaction for 1x1 square foot area
- B = Width of Area Loaded, in feet

6.4.2 Mat Settlements

Settlements of mat foundations should be approximately less than 1.5 inches, using a maximum net allowable bearing pressure of 650 psf for the mat. Settlement will occur throughout the construction process as soils are gradually loaded. Typically more than 50 percent of the settlement occurs during the construction phase of the project. Differential settlements should be approximately ½ inch, or less, between the corner of the mat and the center of the mat.

6.4.3 Mat Recommendations

Mats should be established on native soils or structural fill extending to suitable native soils. It is recommended that mat foundations are underlain by a minimum thickness of 6-inches of “free-draining” granular material, such as 1-inch to ¾-inch crushed rock. Base course should be installed in a single lift and compacted until well keyed.

Under no circumstance should mats be established upon loose or disturbed soils, sod, rubbish, construction debris, non-engineered fill, other deleterious materials, expansive soils, frozen soils, or within ponded water.

6.5 Pipe Pile Foundation

6.5.1 Design Criteria

We have evaluated the static compressive capacities for 12.75- and 16.0-inch diameter, closed end steel pipe piles, using data collected primarily from cone penetration tests. Boring logs and laboratory testing were also referenced in pile capacity calculations. Piles will rely primarily on a dense layer of sand and gravel for end bearing at approximately 40 to 50 feet below grade; the majority of the pile capacity will result from tip resistance within these soils. A factor of safety of 2.25 was applied against ultimate soil failure under real load conditions, a factor of safety of 1.2 was applied for a post-liquefaction condition. The calculated ultimate vertical capacities are based on an installed center-to-center spacing equal to or greater than three times the pile diameter. Piles must be spaced no closer than three pile diameters, center to center. Piles driven at a closer spacing will require reduction in their axial lateral capacity due to pile group effects.

Soils that experience liquefaction during a major earthquake are capable of generating approximately 2 to 3 inches of additional settlement at the site. This settlement will produce additional loading upon piles in the form of negative skin friction as the soils around the pile settle. We estimate that the depth of down-drag to be approximately 25 feet. Piles should bear upon the layer of dense sand and gravel encountered at a depth of 40 to 50 feet below grade. Therefore pile should be driven until they encounter this dense layer at a minimum of 40 feet below grade. Piles supported by this dense sand layer can be designed with an allowable pile capacity of 65 and 100 kips for 12.75 and 16-inch piles, respectively.

6.5.2 Installation

Driving through the lacustrine soils should be relatively straightforward. Diesel and hydraulic hammers are presently utilized in the northern Utah area. Based on Wave Equation Analysis Program (WEAP) analysis, we anticipate that hammers used at the site should have a striking energy in the range of 39,000 to 52,000 foot-pounds per blow for 12.75-inch pipe piles and between 42,000 to 76,000 foot-pounds per blow for 16-inch pipe piles. Piles should have a yield strength of 45 ksi. Diesel Hammers should use a pile cushion with steel piles to reduce induced stress.

The process of pile installation can cause significant vibrations in nearby facilities, sometimes causing damage. Monitoring of nearby buildings is advised.

6.5.3 Pile Settlements

Ultimate settlement of pile groups designed and installed in accordance with the above recommendations and supporting the maximum anticipated real loads previously described are expected to be less than approximately one-half of an inch with differential settlements between adjacent pile caps on the order of one-half the total settlement. Settlements should occur rapidly as the foundation is being loaded. We estimate approximately 60 percent of the ultimate settlements to occur during construction.

6.5.4 Pile Lateral Capacities

The response of piles to potential applied lateral loads was analyzed using LPILE Plus for Windows, Version 5.0. Piles with diameters of 12.75 and 16 inches were each analyzed for top of pile deflections of approximately ¼, ½, and 1 inch. A fixed head condition was assumed for the analysis. The lateral capacities of concrete filled pipe piles do not differ significantly from empty piles; therefore, piles were assumed to be 50-feet, empty, close-ended, pipe piles.

The native soil profile representative of the site was used in the analyses. The soil design parameters used in the LPILE analysis, are summarized in the following table.

Soil Profile for Pile Lateral Resistance

Soil/Bedrock Type	Depth Interval (ft)	Unit Weight (pcf)	Cohesion / Compressive Strength (psi)	Friction Angle (degrees)	ε50 / Soil Modulus (lbs/inch)	k-Value ($lb/inch^3$)
Stiff Clay	0 to 7	110	10.42	--	.005	--
Sand and Gravel	7 to 11	75 [1]	--	33	--	90
Silt and Sandy Silt	11 to 35	71 [1]	3.47	29	.010	50
Soft Lean Clay	35 to 45	60 [1]	11.11	--	.020	--
Sand and Gravel	45 to 55	75 [1]	--	35	--	125

[1] Effective unit weight below groundwater level

The following deflections at the top of the piles, 12 inches above grade, and the indicated lateral resistances are tabulated below:

Estimated Pile Lateral Resistance

Top Deflection (inches)	Shear Force (kips)	
	12.75-in dia.	16.0-in dia.
0.25	12	17.5
0.5	17	24
1.0	24	36

6.5.5 Pile Uplift Capacities

The uplift pile capacities for the 12.75 and 16 inch pile are 45 and 60 kips, respectively. The uplift capacity includes a factor of safety of 2.25 against ultimate soil failure under real load conditions. The uplift capacities do not include the weight of the piles. The design uplift capacity of the pile group should be the capacity of a single pile times the number of piles.

<u>All pile capacities presented in this report relate to geotechnical capacities only. Structural capacities are determined by others.</u>

6.5.6 Driving Observation

Driving of the piles should be supervised on a continuous basis. The project geotechnical engineer should review all driving data on a day-to-day basis. Pile installation must be monitored dynamically using a Pile Driving Analyzer (PDA) on 5 percent of all the piles. This will provide verification of the pile capacities during and after driving. For each pile tested, if the capacity can not be verified based on the initial drive test, a re-strike test must be performed. Case Pile Wave Analysis Program (CAPWAP) should be performed in order to give pile capacity and an estimate of the distribution of resistance along the pile and at the toe.

6.6 Auger Cast Pile Foundation

6.6.1 Design Criteria

We have evaluated the static compressive capacities for 1.5-feet diameter auger cast piles. Capacities have been developed using measured cone skin and tip resistance. A factor of safety of 2.5 was applied against ultimate soil failure under real load conditions, a factor of safety of 1.2 was applied for a post-liquefaction condition. The calculated ultimate vertical capacities are based on an installed center-to-center spacing equal to or greater than three times the auger-cast pile diameter. Auger cast piles must be spaced no closer than three pile diameters, center to center. Piles drilled at a closer spacing will require reduction in their axial lateral capacity due to pile group effects.

Auger-cast piles will also experience 25 feet of soil down-drag after a liquefaction event. Analyses have taken into account the resulting negative skin friction. There should be a minimum tip elevation of 40 feet for the auger cast piles. The allowable capacity for an 18-inch auger cast pile embedded into the dense sand or gravel layer at 40 to 50 feet below grade is 100 kips.

6.6.2 Auger Cast Pile Installation

Drilling through the upper layer of the lacustrine and alluvial soils should be relatively straightforward. However, drilling should become more difficult as the dense layer of sand and gravel is encountered. During installation, it is imperative that the auger end within dense sand or gravel at a minimum depth of 40 feet below grade. Maintaining a constant head of grout within the auger stems is important to the proper installation of auger cast piles. It will help to keep the hole open and will help to keep debris and soil from being incorporated into the column.

6.6.3 Auger Cast Pile Settlements

Ultimate settlement of pile groups designed and installed in accordance with the above recommendations and supporting the maximum anticipated real loads previously described are expected to be less than approximately one-half of an inch with differential settlements between adjacent pile caps on the order of one-half the total settlement. Settlements should occur rapidly as the foundation is being loaded. We estimate approximately 60 percent of the ultimate settlements to occur during construction.

6.6.4 Auger Cast Pile Lateral Capacities

The response of piles to potential applied lateral loads was analyzed using LPILE Plus for Windows, Version 5.0. A pile with a diameter of 1.5 feet was analyzed for top of pile deflections of approximately ¼, ½, and 1 inch. A fixed head condition was assumed for the lateral analysis, and the piles were assumed to be 40-feet, (4000 psi) concrete shafts.

The native soil profile representative of the site was used in the analyses. The soil/bedrock design parameters used in the LPILE analysis, are the same as used in section 6.5.4 Pile Lateral Capacities.

The following deflections at the top of the piles, 12 inches above grade, and the indicated lateral resistances are tabulated below:

Estimated Auger Cast Pile Lateral Resistance

Top Deflection (inches)	Shear Force (kips)
	1.5 ft dia.
0.25	17.5
0.5	25
1.0	36

6.6.5 Auger Cast Pile Uplift Capacities

The design value for the allowable uplift capacity of auger cast piles installed to a minimum of 40 feet below grade is 30 kips. The uplift capacity includes a factor of safety of 2.5 against ultimate soil failure under real load conditions. The uplift capacities do include the weight of the piles. The design uplift capacity of pile groups should be the capacity of a single pile times the number of piles.

<u>All pile capacities presented in this report relate to geotechnical capacities only. Structural capacities are determined by others.</u>

6.6.6 Auger Cast Pile Observation

The drilling of piles should be supervised on a continuous basis. The volume of grout pumped into the holes per depth should be tracked in order to help verify proper installation of the pile. During grouting, the bottom of the auger should remain below the grout surface until the pile grout has reached the surface. The project geotechnical engineer should be allowed to review all data on a day-to-day basis. We recommend that pile installation be tested by means of a full scale load test.

6.7 Lateral Earth Pressures

Design lateral earth pressures for embedded walls depend on the type of construction, i.e., the ability of the wall to yield. The two possible conditions regarding the ability of the wall to yield include the at-rest and the active earth pressure cases. The at-rest earth pressure case applies to walls that are relatively rigid and laterally supported at top and bottom and therefore is unable to yield. The

active earth pressure case applies to walls that are capable of yielding slightly away from the backfill by either sliding or rotating about the base. A conventional cantilevered retaining wall is an example of a wall that develops the active earth pressure case by yielding.

Yielding and non-yielding walls can be designed using a lateral earth pressure based on an equivalent fluid having a unit weight of 35 and 55 pcf, respectively. The ground surface should be sloped down at a minimum of 5 percent away from the wall.

6.7.1 Seismic Lateral Earth Pressures

Lateral earth pressure resulting from seismic loading can be calculated based on an equivalent fluid weight of 15 and 30 pounds per cubic foot for active and at-rest cases, respectively. This is assuming an even grade or negative slope at the top of the backfilled wall. For seismic loading the pressure should be inverted increasing from 0 at the base of the wall to a maximum at the top of the wall.

6.8 Floor Support

Floor slabs may be established upon suitable native soils and/or upon structural fill extending to suitable native soils. Slabs may be established upon properly prepared existing near-surface soil, suitable undisturbed natural soils, and/or upon structural fills extending down to suitable natural soils or properly prepared existing near-surface soils. It is recommended that floor slabs are underlain by a minimum thickness of 4-inches of "free-draining" granular material, such as 1-inch to ¾-inch crushed rock. Base course should be installed in a single lift and compacted until well keyed. Settlements of lightly loaded floor slabs are anticipated to be minor.

Under no circumstance should floor slabs be established upon loose or disturbed soils, sod, rubbish, construction debris, non-engineered fill, other deleterious materials, expansive soils, frozen soils, or within ponded water.

6.9 Seismic Hazards

6.9.1 General

Northern Utah is an area of high seismic activity associated with the East Cache fault zone, which defines the eastern boundary of the Basin and Range province. The East Cache fault zone is considered capable of generating earthquakes as large as magnitude 7.3[2].

Utah municipalities have adopted the International Building Code (IBC) 2006. The IBC 2006 code determines the seismic hazard for a site based upon regional acceleration mapping prepared by the United States Geologic Survey (USGS) and the soil site class. The structure must be designed in accordance with the procedures presented in the IBC 2006 edition. The risk from geologic hazards other than those discussed below is low.

6.9.2 IBC Site Class

For dynamic structural analysis, Site Class "D," as defined in Table 1615.1.1, Site Class Definitions of the 2006 IBC, can be utilized.

6.9.3 Earthquake Ground Motions

The IBC 2006 code provides values of ground and structural acceleration for structural design. These design accelerations are based on data collected and interpreted by the US Geological Survey (USGS, 1997) for the maximum considered earthquake (MCE), a level of ground

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[2] Arabasz, W.J., Pechmann, J.C., and Brown, E.D., 1992, Observational seismology and the evaluation of earthquake hazards and risk in the Wasatch Front area, Utah, in Gori, P.L., and Hays, W.W., eds., Assessment of regional earthquake hazards and risk along the Wasatch Front, Utah: U.S. Geological Survey Professional Paper 1500-D, 36 p.

acceleration associated with a 2 percent probability of being exceeded in 50 years (which we abbreviate as 2%PE50yrs). The IBC allows the use of 2/3 of these values. This represents a standard design and risk level, adjusted for local seismicity. Structures could be designed for higher accelerations if the additional costs are out weighed by reduced risk.

Using 41.7667 degrees north latitude and 111.8167 degrees west longitude as the project coordinates; the following table summarizes spectral accelerations for the maximum considered earthquake.

DESIGN EARTHQUAKE ACCELERATIONS

Spectral Acceleration Value	MCE* Ground Motion Values for Site Class B % g
0.2-Sec Spectral Acceleration (S_S)	89.9
1.0–Sec Spectral Acceleration (S_1)	31.7

*MCE – Maximum considered earthquake

For Site Class D and the above-referenced short and long term spectral acceleration values, the amplification factors F_a = 1.14 and F_v = 1.767 values can be used for design.

6.9.4 Surface Fault Rupture

Known active faults are not mapped in the immediate vicinity of the site. The risk of surface fault rupture affecting the site is very low.

6.9.5 Liquefaction & Lateral Spread

Liquefaction is a condition where earthquake ground motion causes a build up of water pressure in the spaces between saturated soil particles causing the soil to behave like a fluid. Liquefaction will generally occur only in relatively loose granular or low-plasticity fine-grained soil subjected to earthquake ground motion with sufficient intensity and sufficient duration. Damaging settlement may result from liquefaction. Damaging lateral movement known as lateral spread may occur if liquefaction occurs beneath a slope or near a free-face, such as the bank of a river.

The site is located in an area that has been mapped as having a “moderate to high liquefaction potential” on planning maps. This generally means that high groundwater is present below the

site. Our investigation confirmed a high water table of approximately seven feet below ground surface at the site. Additionally, field and laboratory analyses confirm that highly susceptible liquefiable soils like low consistency saturated silts are present below the site.

From analysis of cone data, we estimate that settlement due to liquefaction could be as high as two to three inches during a major seismic event. Current methods do not allow precise estimates of settlement due to liquefaction. If liquefaction were to occur, the settlement could be greater or less than estimated.

There are several potential options for addressing potential settlement due to liquefaction including:

- Accept the risk without any additional measures.
- Design the structure to minimize the potential for collapse. This might include designing grade beams to tie foundation together, extra reinforcing in foundations, strengthening key connections, or other measures.
- Mitigate the liquefaction by grouting or densification of the liquefiable layers. This would partly be accomplished through a rammed aggregate foundation system.
- Support the structure on deep foundations extending through the liquefiable layers such as driven piles.

Mitigation is generally very expensive and usually not considered except for critical structures. Deep foundations can also be costly, but are used more widely to help prevent liquefaction settlement.

Although disturbed layering was not encountered during our soil investigation, the results of a lateral spread analysis indicate a potential for lateral spread at the site. Although slopes are mild, they are still within the limits of observed lateral spread; grain-size and soil density is also consistent with potentially liquefiable materials. Available analysis methods are weighted toward sites where lateral spread has occurred; therefore, the lateral spread displacement model may be skewed to predict lateral spread when in fact no lateral spread may occur. Lateral spreading is also dependent on the presence of continuous liquefiable layers below the site. With these considerations, our model indicates total lateral displacements could be approximately a foot, if lateral spread were to occur.

Complete mitigation of lateral spread potential can be difficult to realize. Lateral spread can be a regional problem with movement occurring over large land areas. Prevention often requires mitigation at a regional scale because spreading can overwhelm localized mitigation efforts. Typical mitigation of lateral spread consists of subsurface barriers, which include slurry walls, sheet-pile walls, and columnar walls consisting of packed gravel or a soil cement mix. Barrier walls can also consists of liquefaction ground improvement procedures. Deep foundation such as driven piles or drilled pile can also provide some lateral resistance to soil movement, but such resistance can be overwhelmed if the area and displacement of the lateral spread slide is too great.

The risk of lateral spread occurring during the life of the structure is low, but not negligible. In order for lateral spread to occur there has to be a sizeable earthquake near the site, which may or may not induce liquefaction in site soils. If liquefaction does occur, it does not necessarily mean lateral spread will occur. Soil conditions and empirical models suggest it may occur, but it is not a certainty with a large scale earthquake. Therefore the risk of lateral spread is generally considered low. Similar to liquefaction settlement, mitigation measures for lateral spread are generally very expensive and usually not considered except for critical structures.

6.10 Soil Corrosivity and Sulfate Attack on Concrete

Soil corrosivity and sulfate attack was performed on site soils and was found to be negligibly corrosive. It is our judgment that site soils can use cement type I or II for concrete placed in contact with the on-site soil.

6.11 Pavements

Existing site surface soils exhibit good support characteristics for pavements. From available laboratory data, we estimate the subgrade to have a CBR value of 4. This value was used to calculate pavement sections consistent with Utah Department of Transportation design procedures and recommendations.

Good drainage is vital to the long-term performance of a roadway surface. Parking areas should allow for complete drainage of surface water without the formation of puddles.

Prior to placement of any structural fill or the pavement design section, the exposed subgrade should be prepared as discussed in Section 6.2.1, Site Preparation. If subgrade soils become loose, saturated, or disturbed they should be recompacted to the requirements for structural fill or be removed and replaced with structural fill. A suitable pavement section resulting in adequate pavement performance is highly dependent on actual traffic loading [18 kip equivalent single axle loads (ESALs) especially for heavy truck traffic]. The designer/owner should choose the appropriate sections to meet the anticipated traffic volume and life expectancy. The section capacity is reported as daily ESALs, Equivalent 18 kip Single Axle Loads. Typical Light Trucks impart 0.25 to 0.50 ESALs per truck; medium sized trucks and school buses impart 1.0 to 1.5 ESALs per truck; heavy trucks impart 2.0 to 2.5 trucks per day. It takes approximately 1,200 passenger cars to impart 1 ESAL.

If the design team considers that any assumptions are not accurate, AMEC should be informed in order that we may review the pavement designs as necessary. Similarly, AMEC should be contacted if alternate designs are needed. The pavement materials and placement should be in accordance with the Utah Department of Transportation or American Public Works Association specifications.

Pavement Design Parameters

Design Life	20 years
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliablilty	90%
Std Deviation - Flexible	0.45
Asphaltic-Concrete Structural Coefficient	0.4
Untreated Road Base	0.10
Granular Subbase	0.08
Design CBR	4

Flexible Pavement Sections

Alternate	Area of Placement	Daily 18-kip ESALs	Flexible		
			AC	UTBC	GB
Alternate 1	Auto Parking/Drives	4	3.0"	4.0"	8.0"
Alternate 2		8	3.5"	4.0"	8.0"
Alternate 3		16	4.0"	4.0"	8.0"

Notes:
1. Full depth asphalt or increased asphalt thickness can be increased by adding 1.0-inch asphalt for each 4 inches of base course or granular borrow replaced.
2. Based on our experience, limited data, and analysis, we anticipate Alternate 1 as the best cost effective option for the project. However, we recommend that the designer/owner perform their own assessment to determine whether this suggested pavement section really does meet project traffic needs, or whether one of the other alternates would have a capacity better suited to the expected traffic.

Rigid Pavement Sections

Pavement Use	Daily 18-kip ESALs	Layer Thickness (inches)	
		Portland Cement Concrete	Untreated Base Course
Auto Drives	12	5	4
Truck Drives	32	6	4

Sidewalks not subject to vehicle traffic can consist of 4 inches of concrete over 4 inches of granular base. Trash dumpster pads should consist of at least 6 inches of concrete over 4 inches of granular base. Areas in front of dumpsters can be subject to repeated heavy loading from dump trucks, which can cause early failure in asphalt. Great consideration should be given to using a concrete apron in front of the dumpster to help prevent pavement failure.

7. LIMITATIONS

This report has been prepared to aid the architect and engineer in the design of this project. The scope is limited to the specific project and location described herein, and our description of the project represents our understanding of the significant aspects of the project relevant to the design and construction of the earthwork, foundations, and floor slabs. In the event that any changes in the design and location of the building as outlined in this report are planned, we should be given the opportunity to review the changes and to modify or reaffirm the conclusions and recommendations of this report in writing.

The conclusions and recommendations submitted in this report are based on the data obtained from the borings made at the locations indicated on Figure 2, Site Plan, and from other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in soil conditions may exist between explorations. This report does not reflect any variations that may occur between these explorations. The nature and extent of variation may not become evident until construction. If, during construction, subsurface conditions are different from those encountered in the explorations, we should be advised at once so that we can observe and review these conditions and reconsider our recommendations where necessary.

Our professional services have been performed, our findings obtained, and our recommendations prepared in accordance with generally accepted engineering principles and practices at this time along the Wasatch Front.

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

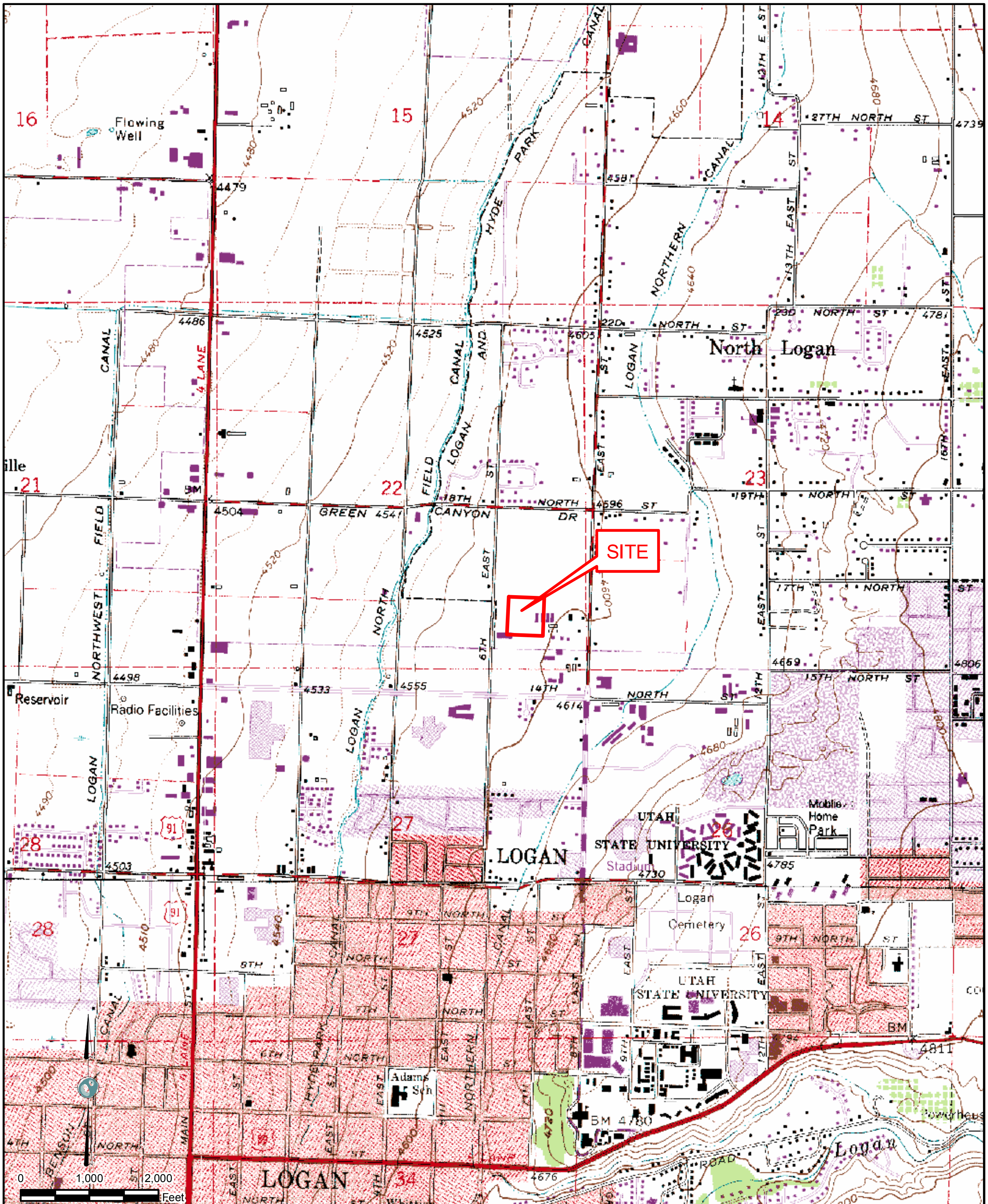
Respectfully submitted,
AMEC Earth & Environmental, Inc.

Reviewed by:

Daniel W. DeDen, P.E.
Professional Engineer

David K. Fadling, P.E.
Senior Engineer

Addressee (4)



AMEC Earth & Environmental
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Smithfield Quadrangle
USGS 7.5 Minute
Series (Topographic)

CLIENT
AJC Architects
703 East 1700 South
Salt Lake City, Utah 84105

PROJECT: UTAH STATE UNIVERSITY USTAR RESEARCH INSTITUTE 1600 North 600 East Logan, Utah	DWN BY: MKW	DATUM: NAD 83	DATE: 11/07/07
	CHK'D BY: BMP	P:\Geo\2007\7-817-005223\GIS\Figure1 Vicinity Map	PROJECT NO: 7-817-005223
TITLE: VICINITY MAP	PROJECTION: UTM 12 North	SCALE: 1 inch equals 2,000 feet	FIGURE NO: 1

NOTE: THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE AMEC EARTH & ENVIRONMENTAL REPORT NO. 7-817-005223.

REFERENCE: AERIAL PHOTOGRAPHY IS HIGH RESOLUTION ORTHO-PHOTOGRAPHY (HRO), DATED 2006. 1-FOOT RESOLUTION.

Legend

- ● Approximate Bore Location In Proposed Building Footprint
- ● Approximate Bore Location At Proposed Parking Area
- ▲ Cone Penetration Test Location

CLIENT
AJC Architects
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Salt Lake City, Utah 84105

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9865 South 500 West
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amec

DWN BY:	MKW
CHK'D BY:	BMP
DATUM:	N/A
PROJECTION:	N/A
SCALE:	NOT TO SCALE

PROJECT
UTAH STATE UNIVERSITY USTAR RESEARCH INSTITUTE
1600 North 600 East
Logan, Utah

TITLE
SITE PLAN

P:\Geo\2007\7-817-005223\GIS\Figure2 Site Plan

DATE:	01/24/08
PROJECT NO:	7-817-005223
FIGURE NO:	2

LOG OF BORING NO. B-1

Project Name: USU USTAR
Location: 1600 North 600 East
Logan, UT
Project No: 7-817-005223
Sheet 1 of 1

Date Drilled: 10/25/07
Rig Type: SIMCO 2800
Drilled By: A Cache
Logged By: R. Buxton

amec

Surface El.:

Elevation, feet	Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Penetration Blows / Foot	Recovery, in	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200 Sieve	Liquid Limit	Plasticity Index	REMARKS
	0		**Lean CLAY [CL]** very stiff, dark brown, dry to moist, low to medium plastic, pinholes, major roots to 2", plow disturbance to 12"									
				D-1	71	16						
	5		5.0 **Layered SILTS and CLAYS with fine sand [CL-ML]** stiff to very stiff, light brown to grey, damp to wet, low to medium plasticity, occasional silty sand layers 1" to 12".	D-2	52	18	107	21	86	28	7	
			Some Gravels at 8.5' - 12'									
	10			D-3	16	16			47			
	15			D-4	20	16	108	22				
	20		21.5	D-5	24	0						
			Bottom of Boring at 21.5' 1 1/4" Slotted PVC pipe Installed to 19'									
	25											

Remarks:

Water Level Observations	
∇	
▼ 9.5 ft	11/6/07

The discussion in the report is necessary for a proper understanding of the nature of subsurface materials.

Figure 3A

AMEC.SLC.BORING.1.BASE 7-817-005223 USU USTAR RESEARCH INSTITUTE.GPJ LAGNNN10.GDT 11/29/07

LOG OF BORING NO. B-2

Project Name: USU USTAR
Location: 1600 North 600 East
Logan, UT
Project No: 7-817-005223
Sheet 1 of 1

Date Drilled: 10/25/07
Rig Type: SIMCO 2800
Drilled By: A Cache
Logged By: R. Buxton

amec

Surface El.:

Elevation, feet	Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Penetration Blows / Foot	Recovery, in	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200 Sieve	Liquid Limit	Plasticity Index	REMARKS
	0–3.5		**Lean CLAY [CL]** very stiff, dark brown, dry to moist, low to medium plastic, pinholes, major roots to 2", plow disturbance to 12"	D-1	36	13						
	3.5–7.0		**Layered SILTS and CLAYS with fine sand [CL-ML]** stiff to very stiff, light brown to grey, damp to wet , low to medium plasticity, occasional silty sand layers 1" to 12"	D-2	27	13						
	7.0–10.0		**Silty GRAVEL with Sand [GM]** medium dense to dense, wet, subangular to angular clasts	D-3	16	16	110	20	61	23	3	
	10.0–15.0		**Layered SILTS and CLAYS with fine sand [CL-ML]** stiff to very stiff, light brown to grey, damp to wet, low to medium plasticity, occasional silty sand layers 1" to 12" some gravel at 11.5' to 12.5'	D-4	16	13						
	15.0		Bottom of Boring at 15'									

Remarks:

Water Level Observations	
▽ 6.0 ft	10/25/07
▼	

The discussion in the report is necessary for a proper understanding of the nature of subsurface materials.

Figure 3B

AMEC.SLC.BORING.1.BASE 7-817-005223 USU USTAR RESEARCH INSTITUTE.GPJ LAGNNN10.GDT 11/29/07

LOG OF BORING NO. B-3

Project Name: USU USTAR
Location: 1600 North 600 East
Logan, UT
Project No: 7-817-005223
Sheet 1 of 1

Date Drilled: 10/25/07
Rig Type: SIMCO 2800
Drilled By: A Cache
Logged By: R. Buxton

amec

Surface El.:

Elevation, feet	Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Penetration Blows / Foot	Recovery, in	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200 Sieve	Liquid Limit	Plasticity Index	REMARKS
	0–1.0		**Lean CLAY [CL]** very stiff, dark brown, dry to moist, low to medium plastic, pinholes, major roots to 2", plow disturbance to 12"									
	1.0–2.0		**Silty GRAVEL with Sand [GM]** medium dense to dense, dry, angular to subangular clasts									
	2.0–20.5		**Layered SILTS and CLAYS with fine sand [CL-ML]** stiff to very stiff, light brown to grey, damp to wet, low to medium plasticity, occasional silty sand layers 1" to 12"	D-1	22	16	103	23	60			
					15	0						
	10			D-2	15	15						
	15			D-3	12	12	107	23	77	NP	NP	
	20			D-4	12	12	106	23		28	8	
	20.5		Bottom of Boring at 20.5' 1 1/4" Slotted PVC pipe Installed to 19'									
	25											

Remarks:

Water Level Observations	
▽	
▼ 7.7 ft	11/6/07

The discussion in the report is necessary for a proper understanding of the nature of subsurface materials.

Figure 3C

AMEC.SLC.BORING.1.BASE 7-817-005223 USU USTAR RESEARCH INSTITUTE.GPJ LAGNNN10.GDT 11/29/07

LOG OF BORING NO. B-4

Project Name: USU USTAR
Location: 1600 North 600 East
Logan, UT
Project No: 7-817-005223
Sheet 1 of 1

Date Drilled: 10/25/07
Rig Type: SIMCO 2800
Drilled By: A Cache
Logged By: R. Buxton

amec

Elevation, feet	Depth, feet	Graphic Log	Surface El.: MATERIAL DESCRIPTION		Samples	Penetration Blows / Foot	Recovery, in	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200 Sieve	Liquid Limit	Plasticity Index	REMARKS
			Lean CLAY [CL] very stiff, dark brown, dry to moist, low to medium plastic, pinholes, major roots to 2", plow disturbance to 12"	0.8									
			Silty GRAVEL with Sand [GM] medium dense to dense, dry, angular to subangular clasts	3.0	D-1	22	16						
			Layered SILTS and CLAYS with fine sand and gravel [CL-ML] stiff to very stiff, light brown to grey, dry, low to medium plasticity	3.5									
	5		Bottom of Boring at 3.5'										
	10												
	15												
	20												
	25												

Remarks:

Water Level Observations	
▽	
▼	

The discussion in the report is necessary for a proper understanding of the nature of subsurface materials.

Figure 3D

AMEC.SLC.BORING.1.BASE 7-817-005223 USU USTAR RESEARCH INSTITUTE.GPJ LAGNNN10.GDT 11/29/07

LOG OF BORING NO. B-5

Project Name:	USU USTAR	Date Drilled:	10/25/07
Location:	1600 North 600 East	Rig Type:	SIMCO 2800
	Logan, UT	Drilled By:	A Cache
Project No:	7-817-005223 Sheet 1 of 1	Logged By:	R. Buxton

amec

Surface El.:

Elevation, feet	Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Penetration Blows / Foot	Recovery, in	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200 Sieve	Liquid Limit	Plasticity Index	REMARKS
			Lean CLAY [CL] very stiff, dark brown, dry to moist, low to medium plastic, pinholes, major roots to 2", plow disturbance to 12" 0.8									
			1.3									
			Silty GRAVEL with Sand [GM] medium dense to dense, brown, dry, angular to subangular clasts									
			Layered SILTS and CLAYS with fine sand [CL-ML] stiff to very stiff, light brown to grey, damp, low to medium plasticity, occasional silty sand layers	D-1	18	18						
	5		5.0									
			Bottom of Boring at 5'									
	10											
	15											
	20											
	25											

Remarks:

Water Level Observations	
∇	
∇	

The discussion in the report is necessary for a proper understanding of the nature of subsurface materials.

Figure 3E

LOG OF BORING NO. B-6

Project Name: USU USTAR
Location: 1600 North 600 East
Logan, UT
Project No: 7-817-005223
Sheet 1 of 1

Date Drilled: 10/25/07
Rig Type: SIMCO 2800
Drilled By: A Cache
Logged By: R. Buxton

amec

Surface El.:

Elevation, feet	Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Penetration Blows / Foot	Recovery, in	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200 Sieve	Liquid Limit	Plasticity Index	REMARKS
			Lean CLAY [CL] very stiff, dark brown, dry to moist, low to medium plastic, pinholes, major roots to 2" plow disturbance to 12" — 1.0									
			SILT [ML] very stiff, brown, dry to moist — 3.0	D-1	16	16						
	5		**Layered SILTS and CLAYS with fine sand [CL-ML]** stiff to very stiff, light brown to grey, dry, low to medium plasticity, occasional silty sand layers — 5.0	GS-2								
			Bottom of Boring at 5'									
	10											
	15											
	20											
	25											

Remarks:

Water Level Observations	
▽	
▼	

The discussion in the report is necessary for a proper understanding of the nature of subsurface materials.

Figure 3F

AMEC.SLC.BORING.1.BASE 7-817-005223 USU USTAR RESEARCH INSTITUTE.GPJ LAGNNN10.GDT 11/29/07

LOG OF BORING NO. B-7

Project Name: USU USTAR	Date Drilled: 10/25/07
Location: 1600 North 600 East Logan, UT	Rig Type: SIMCO 2800 Drilled By: A Cache
Project No: 7-817-005223 Sheet 1 of 1	Logged By: R. Buxton

amec

Surface El.:

Elevation, feet	Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Penetration Blows / Foot	Recovery, in	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200 Sieve	Liquid Limit	Plasticity Index	REMARKS
			Lean CLAY [CL] very stiff, dark brown, dry to moist, low to medium plastic, pinholes, major roots to 2", plow disturbance to 12" (1.5)									
			Layered SILTS and CLAYS with fine sand [CL-ML] stiff to very stiff, light brown to grey, damp, low to medium plasticity, occasional silty sand layers	D-1	9	11			92			
	5		(5.0) Bottom of Boring at 5'									
	10											
	15											
	20											
	25											

Remarks:

Water Level Observations	
∇	
▼	

The discussion in the report is necessary for a proper understanding of the nature of subsurface materials.

Figure 3G

LOG OF BORING NO. B-8

Project Name: USU USTAR	Date Drilled: 10/25/07
Location: 1600 North 600 East, Logan, UT	Rig Type: SIMCO 2800
	Drilled By: A Cache
Project No: 7-817-005223 Sheet 1 of 2	Logged By: R. Buxton

amec

Surface El.:

Elevation, feet	Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Penetration Blows / Foot	Recovery, in	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200 Sieve	Liquid Limit	Plasticity Index	REMARKS
	0–4.0		**CLAY with Silt [CL]** very stiff, dark brown, dry to moist, low to medium plastic, pinholes, major roots to 2", plow disturbance to 12"									
	4.0		**Silty GRAVEL with Sand [GM]** medium dense to dense, dry, subangular to angular clasts									
	5			S-1	39	2						
	8.0		**SILT to SILT with Clay [CL-ML]** layered silts and clays with fine sand; stiff to very stiff, light brown to grey, wet, low to medium plasticity, occasional silty sand and clay layers 1" to 12"									
	10			S-2	8	18						
	13.5		some gravel at 13.5'									
	15			S-3	6	15						
	20			S-4	10	16						
	25			S-5	8	18						

Remarks:

Water Level Observations	
▽ 8.5 ft	10/25/07
▼ 8.0 ft	11/6/07

The discussion in the report is necessary for a proper understanding of the nature of subsurface materials.

Figure 3H

LOG OF BORING NO. B-8

Project Name: USU USTAR
Location: 1600 North 600 East
Logan, UT
Project No: 7-817-005223
Sheet 2 of 2

Date Drilled: 10/25/07
Rig Type: SIMCO 2800
Drilled By: A Cache
Logged By: R. Buxton

amec

Surface El.:

Elevation, feet	Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Penetration Blows / Foot	Recovery, in	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200 Sieve	Liquid Limit	Plasticity Index	REMARKS
	30			S-6	9	14						
	35			S-7	17	14						
			37.0									
			Lean CLAY [CL] medium stiff, grey to dark grey, wet, low to medium placticity									
	40			S-8	4/6"	24				35	14	
	45			S-9	5	18						
			45.5									
			Bottom of Boring at 45.5' 1 1/4" Slotted PVC pipe Installed to 19'									
	50											

Remarks:

Water Level Observations		
∇	8.5 ft	10/25/07
▼	8.0 ft	11/6/07

The discussion in the report is necessary for a proper understanding of the nature of subsurface materials.

Figure 3H

AMEC.SLC.BORING.1.BASE 7-817-005223 USU USTAR RESEARCH INSTITUTE.GPJ LAGNNN10.GDT 11/29/07

SOIL CLASSIFICATION CHART & LEGEND

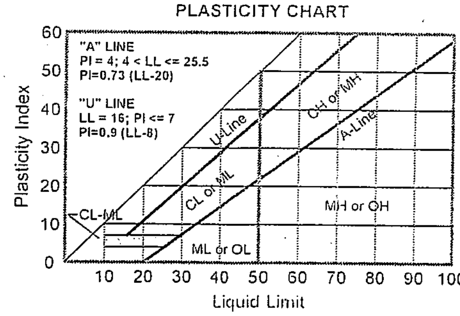
amec

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% passing 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% Passing 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-silt mixtures
	SANDS (50% or more of coarse fraction passes No. 4 sieve)	CLEAN SANDS (Less than 5% passing No. 200 sieve)			SW	Well graded sands, gravelly sands
					SP	Poorly graded sands, gravelly sands
		SANDS WITH FINES (More than 12% Passing 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	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 diatomacaeous silty soils, elastic silts
	CLAYS Limits Plot Above A Line	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
	ORGANICS 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 OF HIGH PLASTICITY (Liquid Limit 50 or more)			OH	Organic silts and clays of high to medium 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 gray zone on the plasticity chart have dual classifications.

- D - Dames and Moore Sampler
- S - Split Spoon Sampler (SPT)
- T - Pushed Thin Walled Tube
- GS - Grab Sample
- BS - Bulk Sample
- DT - Driven Thin Wall
- C - Rock Core Sample
- CS - Continuous Soil Sample
- R - California Ring Sampler
- Water Level at Time of Drilling
- Stabilized Water Level
- CBR California Bearing Ratio
- PP Pocket Penetrometer, tsf
- ST Swell Test
- TOR Torvane Shear, psf
- UC Unconfined Compression, psf
- NR No Recovery

PLASTICITY CHART

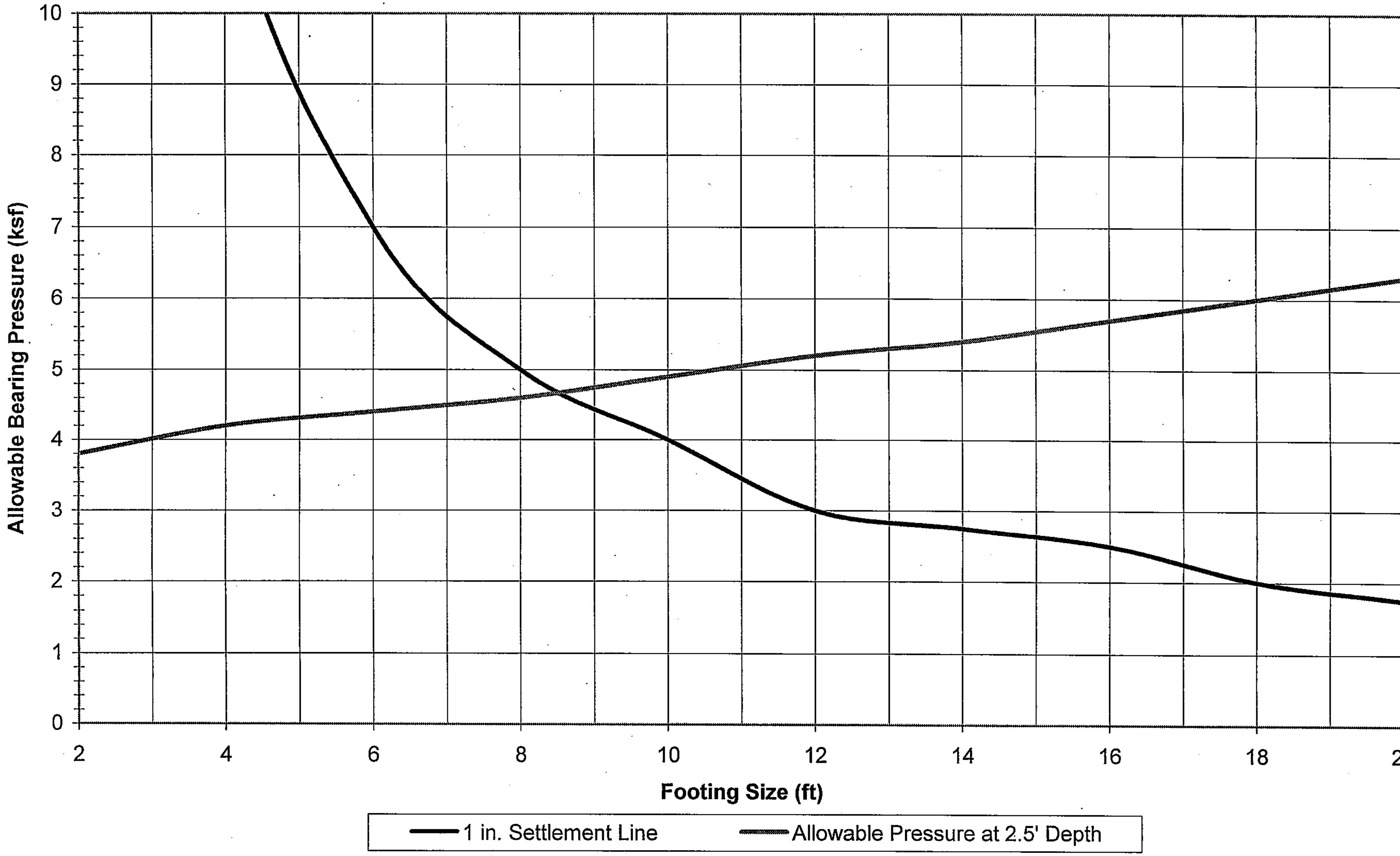


Material	Particle Size	
	mm	Sieve sizes
Boulders	304.8 to 914.4	12 in to 36 in
Cobble	76.2 to 304.8	3 in to 12 in
Gravel Fine Coarse	 4.76 to 19.1 19.1 to 76.2	 3/4 in to 3 in #4 to 3/4 in
Sand Fine Medium Coarse	 2.00 to 4.76 0.42 to 2.00 0.074 to 0.42	 #10 to #4 #40 to #10 #200 to #40
Silt & Clay	<0.074	<#200

Figure 4

AMEC.SLC.LEGEND.SOIL 1.BASE 7-817-005223 USU USTAR RESEARCH INSTITUTE.GPJ 11/30/07

Footing Bearing Pressure Chart



* For square footing underlain with 4 feet of granular fill

Figure 5

APPENDIX A

FIELD EXPLORATIONS

APPENDIX A

FIELD EXPLORATIONS - BORINGS

General

Subsurface materials and conditions at the project site were investigated on October 25, 2007 with 8 borings designated B-1 through B-8. The approximate locations of the borings are shown on Figure 2, Site Plan. All field operations were observed by a senior technician provided by our firm, who maintained a detailed log of the materials and conditions encountered in each boring and directed the sampling operations.

Borings

The borings were drilled with a truck-mounted SIMCO 2800 drill rig provided and operated by A Cache of Mendon, Utah. The borings were advanced to depths ranging from 5 to 45.5 feet below grade using hollow-stem auger drilling and sampling techniques. Disturbed samples were obtained from the borings at three to five-foot intervals of depth. Disturbed samples were obtained using a three inch O.D. Dames & Moore sampler and a two inch standard split spoon sampler. At the time of sampling, the Standard Penetration Test was conducted. This test consists of driving the split-barrel sampler into the soil a distance of 18 inches using a 140-lb hammer falling from a height of 30 inches. The number of blows required to drive the sampler the last 12 inches is recorded as the penetration resistance for the Dames & Moore Split barrel sampler and the standard split spoon sampler. The penetration resistance provides a measure of the relative density of granular soils, such as sand, and the relative consistency, or stiffness, of cohesive soils, such as silt. It should be recognized that penetration resistance values tend to overestimate the relative density of coarse granular soils, such as those containing significant amounts of gravel and cobble-sized particles. The soil samples obtained in the split-spoon sampler were carefully examined in the field, and representative portions were saved in containers for further examination and physical testing in our laboratory.

Logs of the borings are shown on Figures 3A through 3H, Log of Borings. Each log presents a descriptive summary of the various types of material encountered and notes the depth where the materials and/or characteristics of the materials change. To the right of the descriptive summary, the numbers and types of samples taken during the drilling operation are indicated. The terms used to describe the soils are defined on Figure 4, Soil Classification Chart & Legend.

APPENDIX B

LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

General

All samples obtained from the field were transported to our laboratory for examination and testing. The physical characteristics were noted, and the field classifications were modified where necessary. The laboratory testing program was conducted to provide data for our engineering analyses. The laboratory program included determinations of natural moisture content, washed sieve analyses, gradation, Atterberg Limits, consolidation, and chemical tests. The following sections describe the testing program in more detail.

Natural Moisture Content

Natural moisture content determinations were made in general conformance with ASTM D 2216. The results are presented on Figures 3A through 3H, Log of Borings.

Unit Weight

The dry unit weight, or density, of undisturbed soil samples was determined in the laboratory in general conformance with ASTM D 2937.

Percent Passing the No. 200 Sieve (Washed Sieve Analysis)

The silt and clay content (percent passing the No. 200 sieve) were evaluated for selected soil samples in general conformance with ASTM D 1140. Oven-dried samples were weighed and placed on the No. 200 sieve. The silt and clay were washed through the sieve, and the sample remaining on the sieve was oven-dried and weighed. The change in sample weight is used to calculate the percent of material passing the No. 200 sieve.

Gradation Tests

Gradation tests were preformed on selected samples in general accordance with ASTM C 136 to aid in classifying soils. The oven-dried samples were weighed and vibrated through a series of different size sieves. The individual sieves were then weighed in order to calculate the percentage of gravel, sand and fine grained material.

Atterberg Limits

Atterberg Limit tests were performed in general accordance with ASTM D 4318 on representative samples of the native soils encountered at the site to verify field classifications.

One-Dimensional Consolidation Tests

Consolidation tests were performed in general accordance with ASTM D 2435 to obtain data on the compressibility characteristics of samples of relatively undisturbed soil.

Chemical Tests

Chemical tests were conducted on selected samples collected from the site. Water Soluble Sulfate tests were performed by TEI Testing Services, Inc. of Salt Lake City, Utah.

APPENDIX C

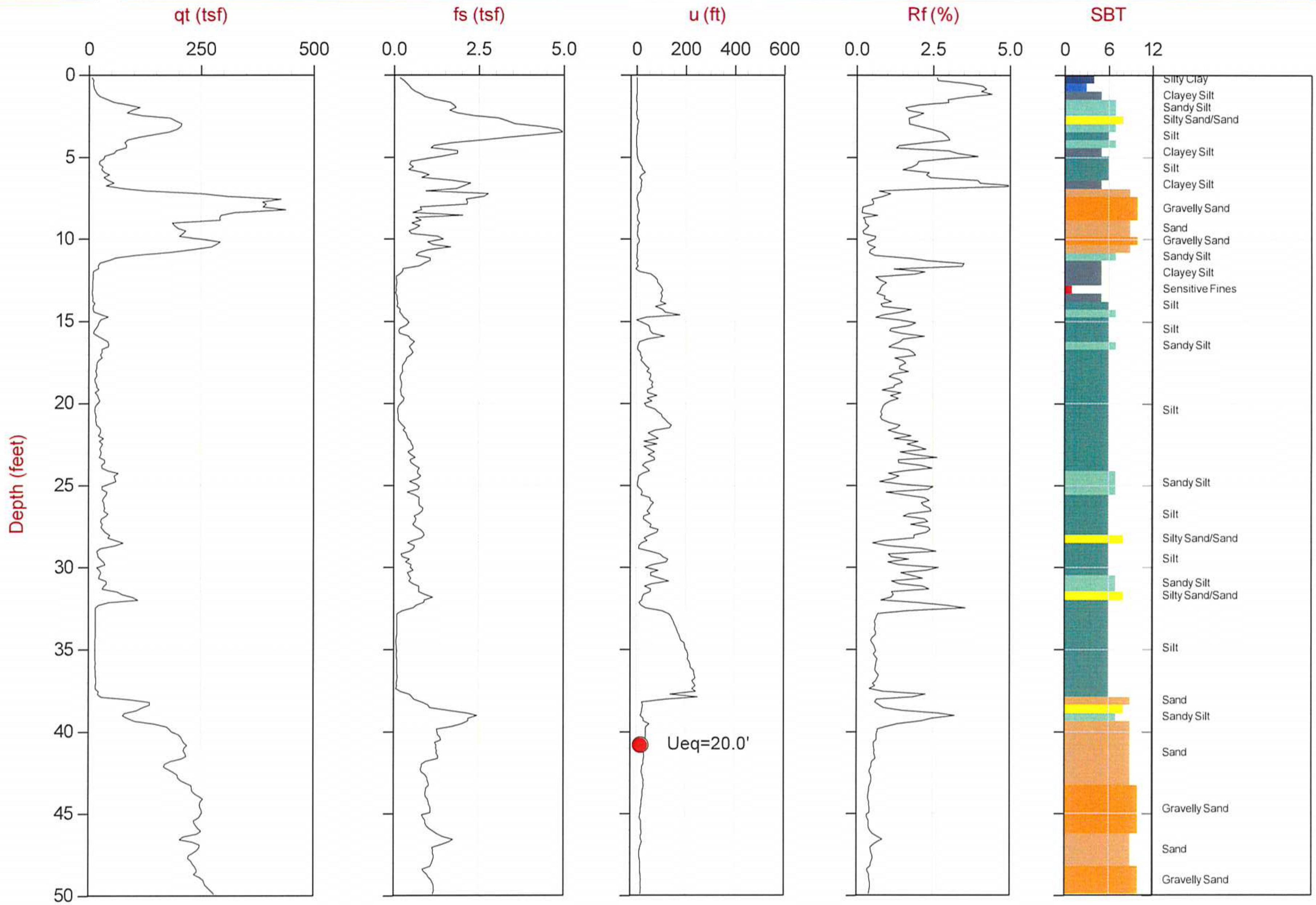
CONE PENETRATION TESTING
By Contec, Inc.

ConeTec

AMEC

Job No: 08-310
Date: 01:18:08 13:04
Site: USU USTAR BUILDING

Sounding: CPT-01
Cone: STD 20T AD183



Max Depth: 16.050 m / 52.66 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.150 m

File: 310CP01.COR

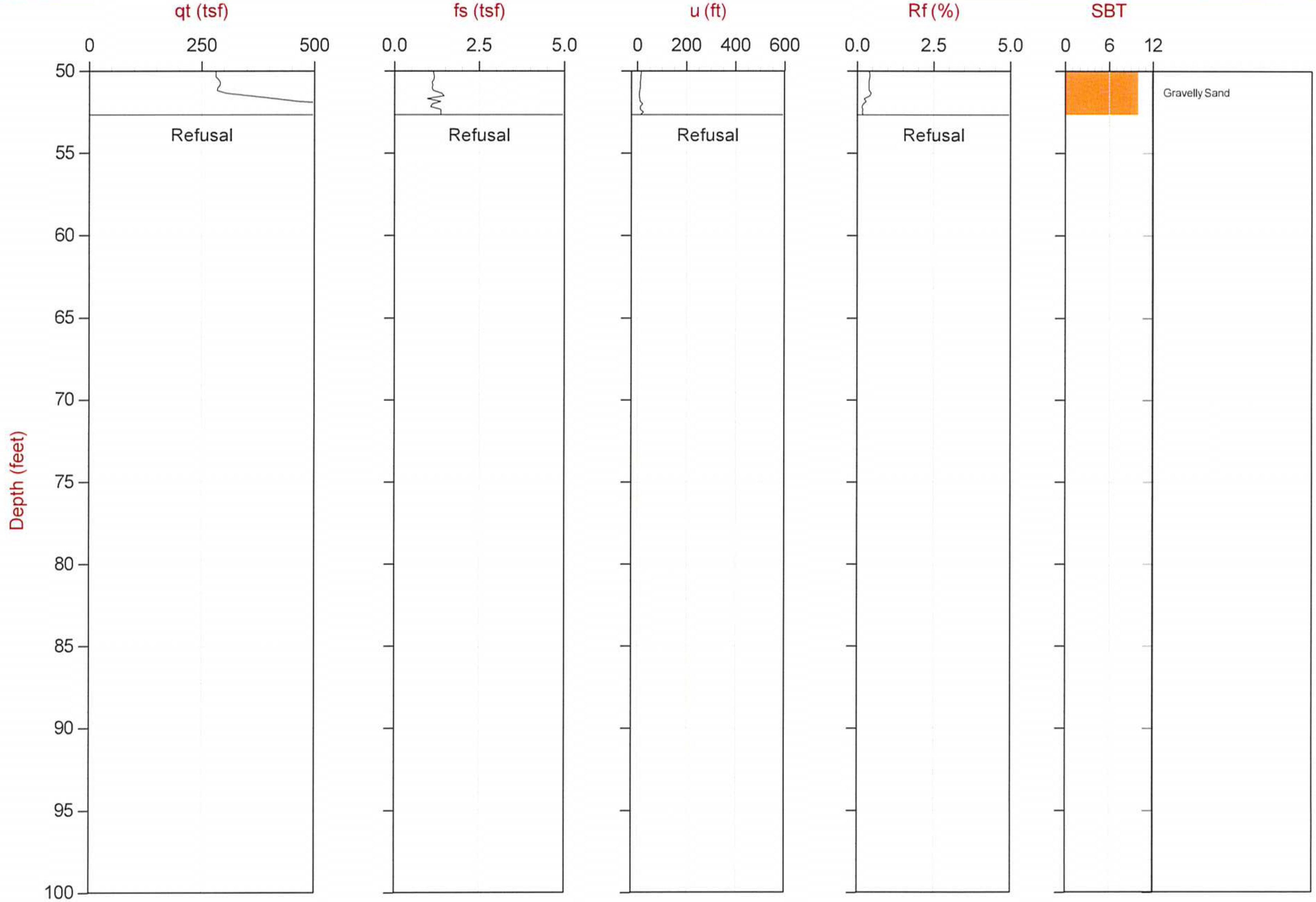
SBT: Lunne, Robertson and Powell, 1997

Equilibrium Pore Pressure from Dissipation

CONETEC | AMEC

Job No: 08-310
Date: 01:18:08 13:04
Site: USU USTAR BUILDING

Sounding: CPT-01
Cone: STD 20T AD183



Max Depth: 16.050 m / 52.66 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.150 m

File: 310CP01.COR

SBT: Lunne, Robertson and Powell, 1997

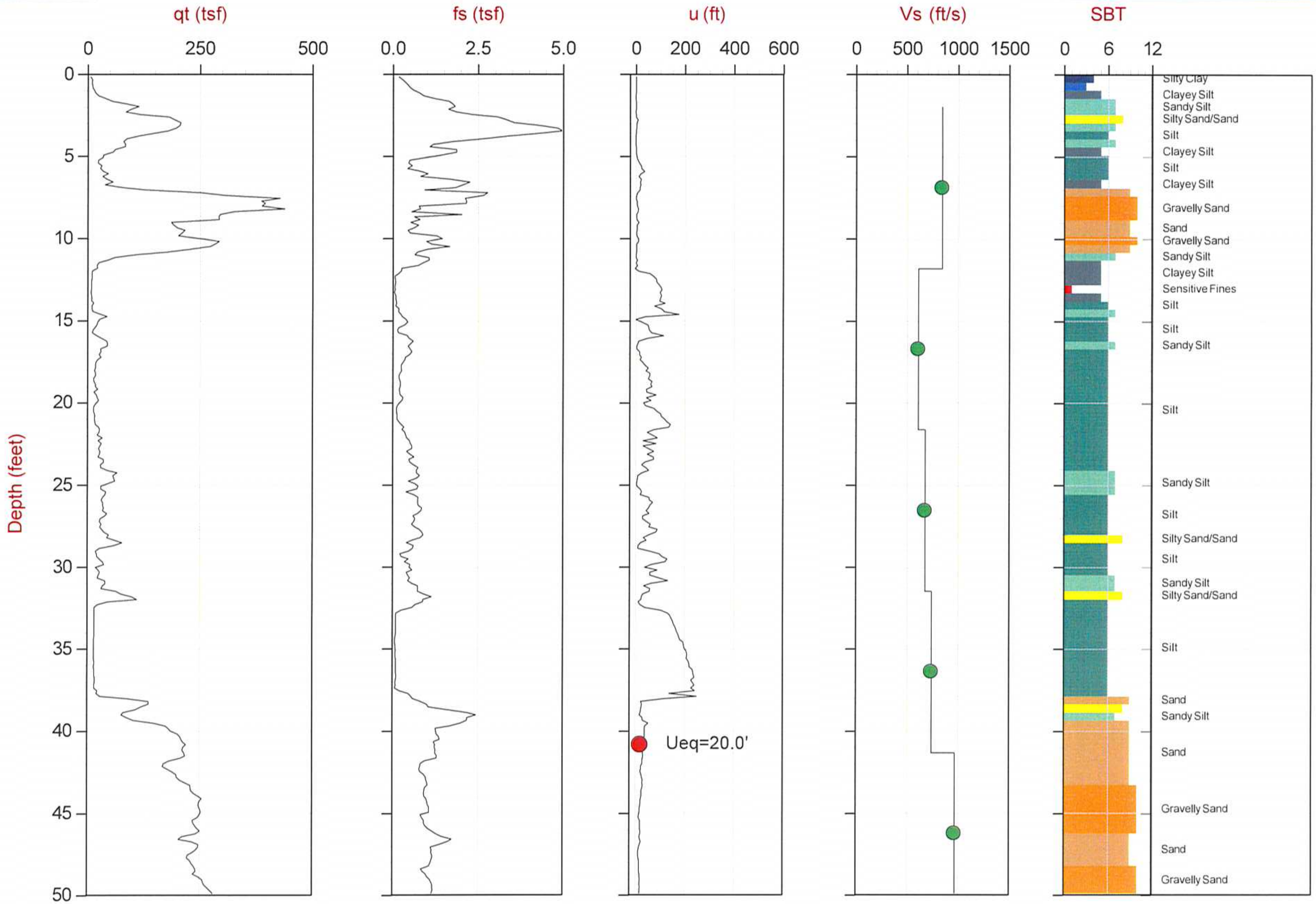
● Equilibrium Pore Pressure from Dissipation

CONETEC

AMEC

Job No: 08-310
Date: 01:18:08 13:04
Site: USU USTAR BUILDING

Sounding: CPT-01
Cone: STD 20T AD183



Max Depth: 16.050 m / 52.66 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.150 m

File: 310CP01.COR

SBT: Lunne, Robertson and Powell, 1997

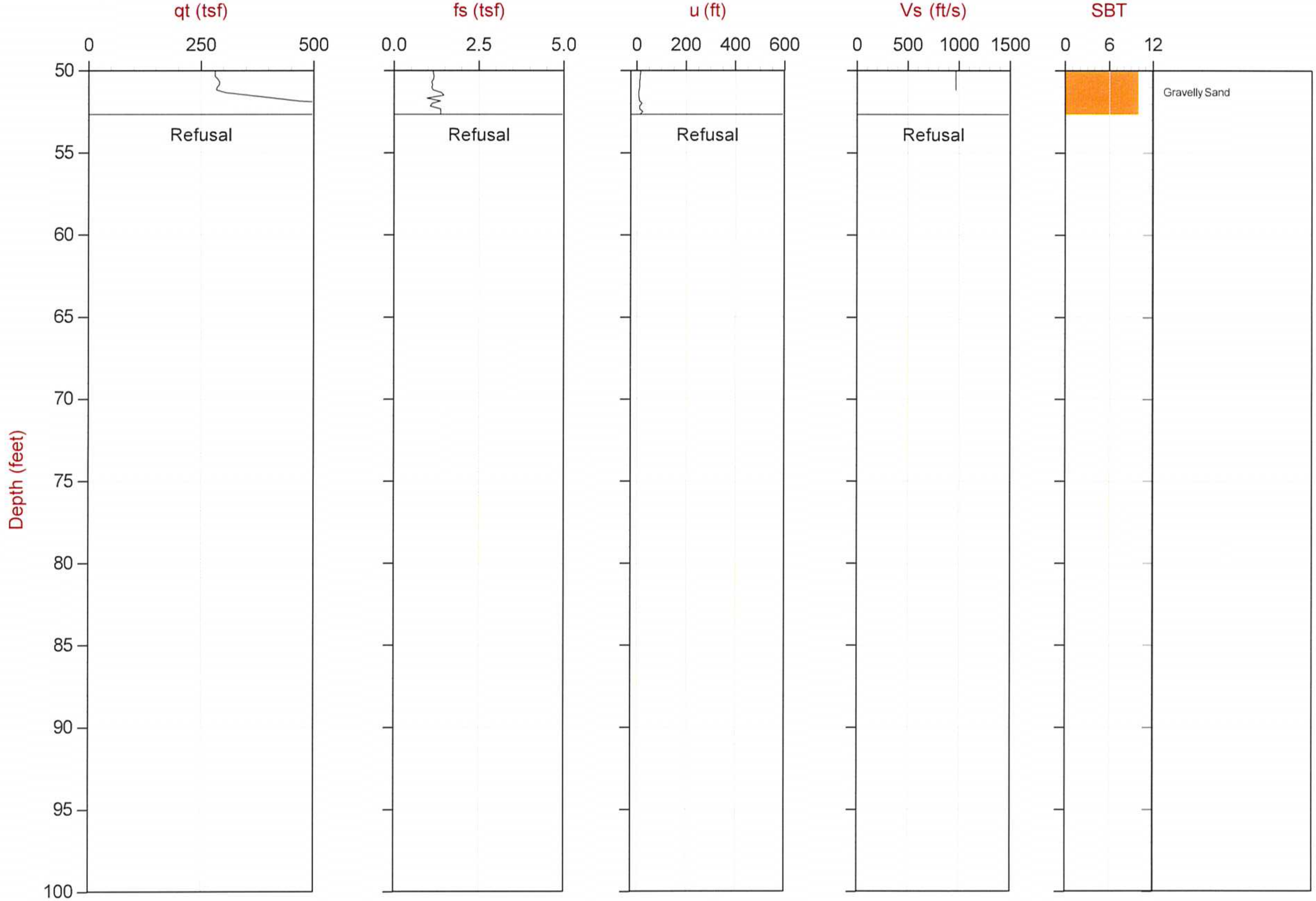
● Equilibrium Pore Pressure from Dissipation

CONETEC

AMEC

Job No: 08-310
Date: 01:18:08 13:04
Site: USU USTAR BUILDING

Sounding: CPT-01
Cone: STD 20T AD183



Max Depth: 16.050 m / 52.66 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.150 m

File: 310CP01.COR

SBT: Lunne, Robertson and Powell, 1997

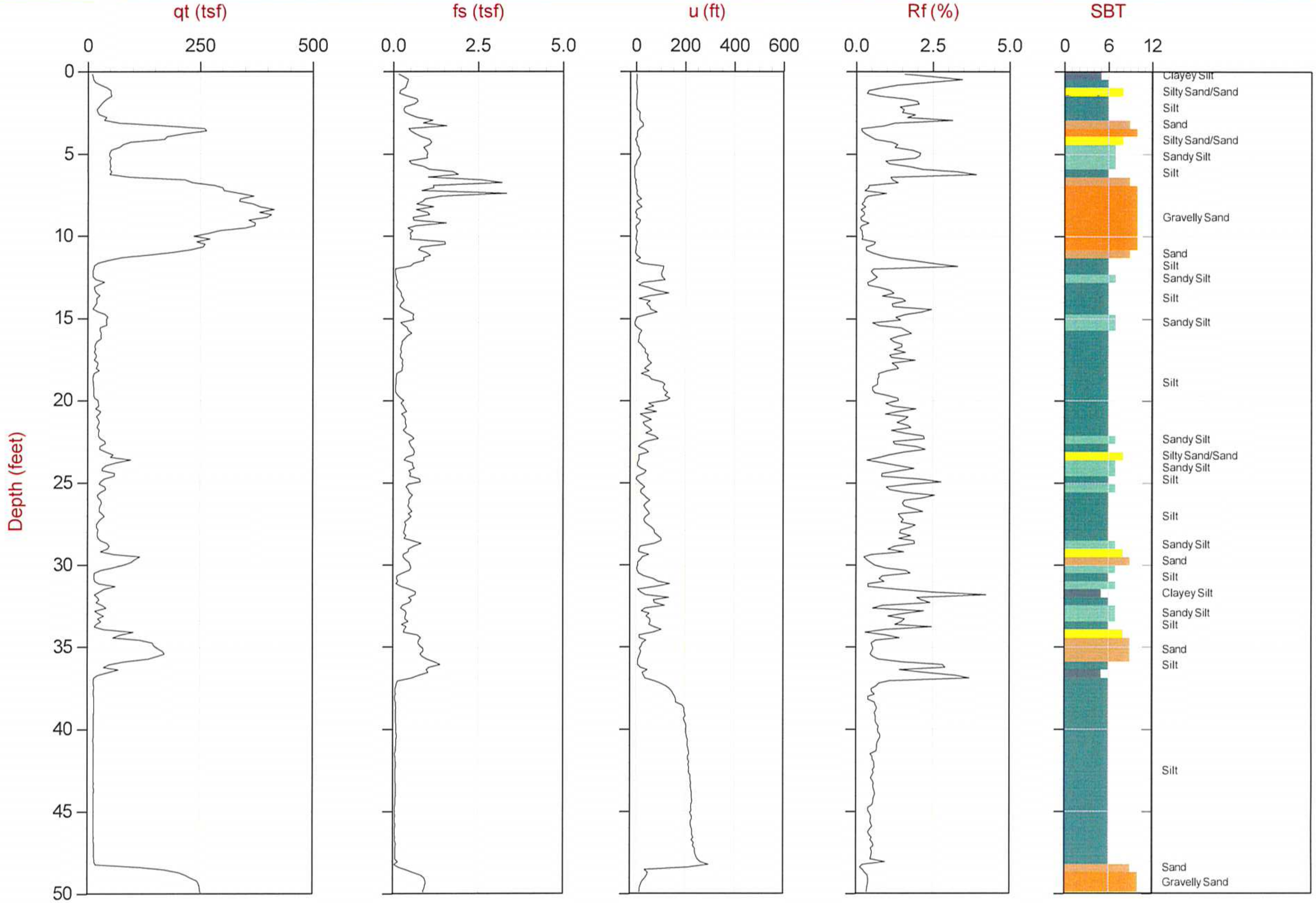
Equilibrium Pore Pressure from Dissipation

CONETEC

AMEC

Job No: 08-310
Date: 01:18:08 14:08
Site: USU USTAR BUILDING

Sounding: CPT-02
Cone: STD 20T AD183



Max Depth: 28.800 m / 94.49 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.150 m

File: 310CP02.COR

SBT: Lunne, Robertson and Powell, 1997

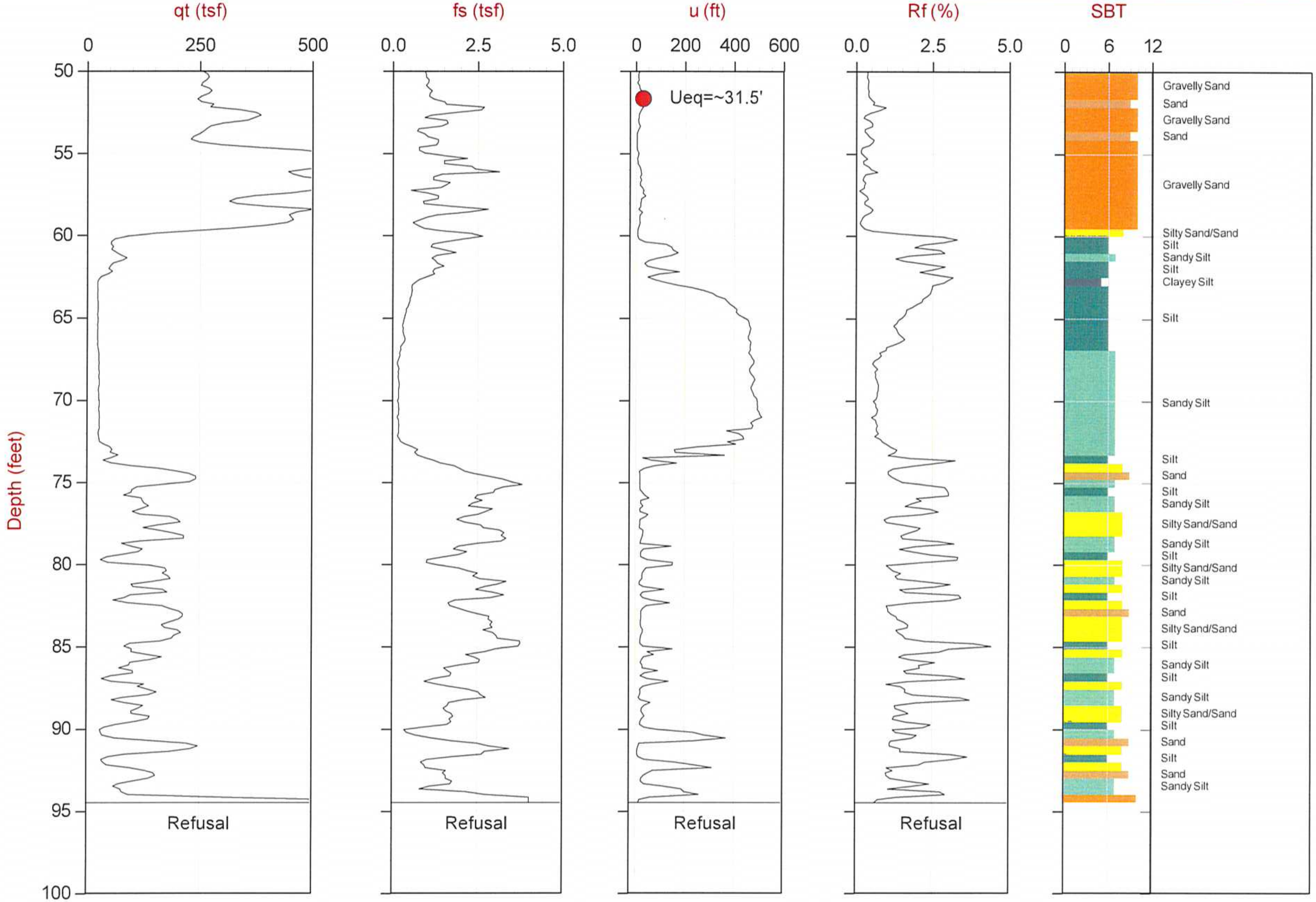
● Equilibrium Pore Pressure from Dissipation

CONETEC

AMEC

Job No: 08-310
Date: 01:18:08 14:08
Site: USU USTAR BUILDING

Sounding: CPT-02
Cone: STD 20T AD183



Max Depth: 28.800 m / 94.49 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.150 m

File: 310CP02.COR

SBT: Lunne, Robertson and Powell, 1997

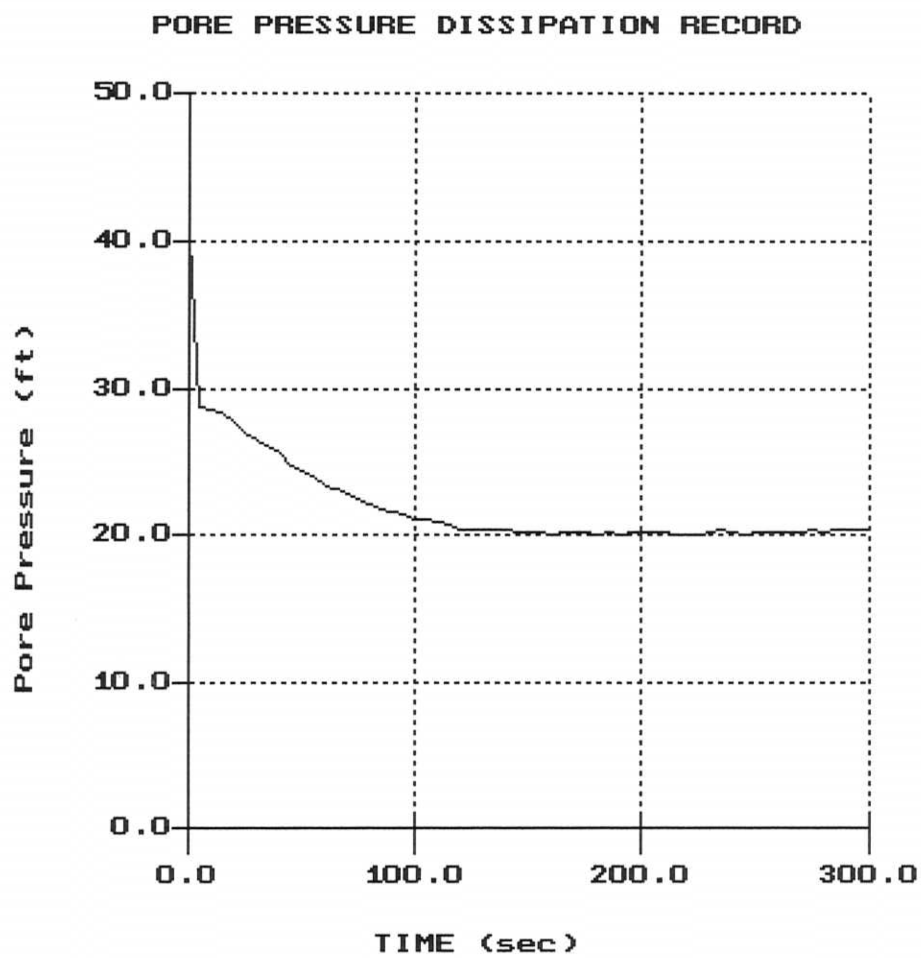
● Equilibrium Pore Pressure from Dissipation

AMEC

Sounding: CPT-01
Site:USU USTAR

Cone:STD 20T AD183
Date:01:18:08 13:04

File: 310CP01.PPD
Depth (m): 12.45
(ft): 40.85
Duration : 300.0s
U-min: 19.90 215.0s
U-max: 40.45 0.0s



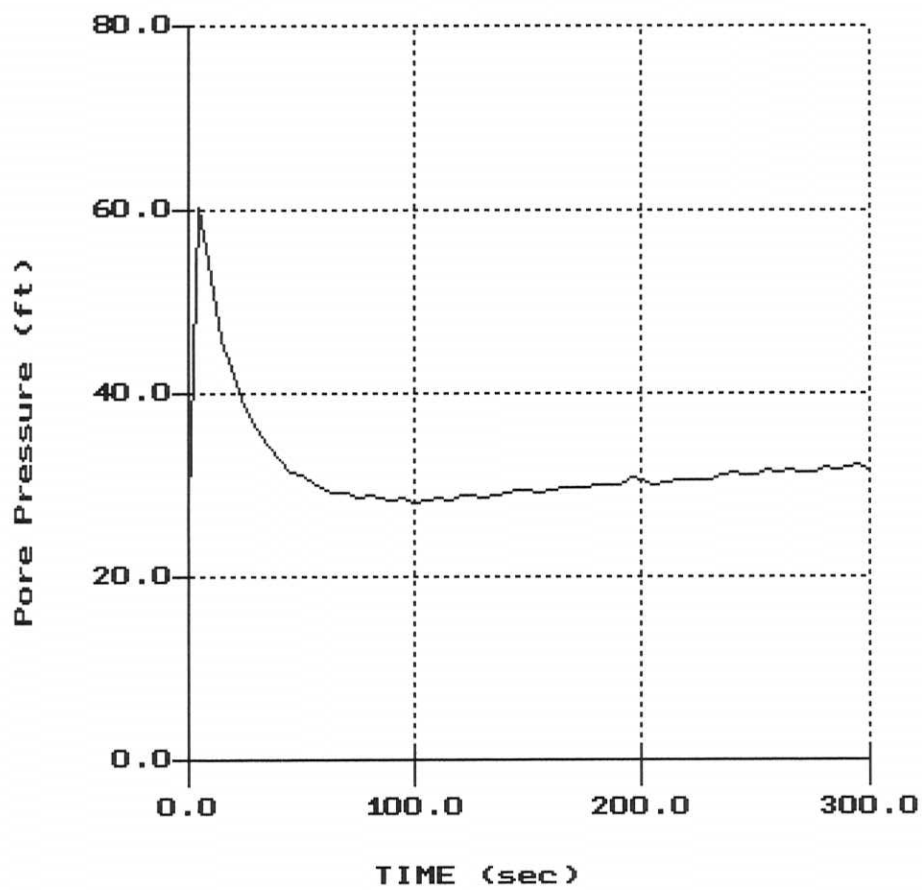
AMEC

Sounding: CPT-02
Site:USU USTAR

Cone:STD 20T AD183
Date:01:18:08 14:08

File: 310CP02.PPD
Depth (m): 15.75
(ft): 51.67
Duration : 300.0s
U-min: 26.70 0.0s
U-max: 60.21 5.0s

PORE PRESSURE DISSIPATION RECORD



CONETEC

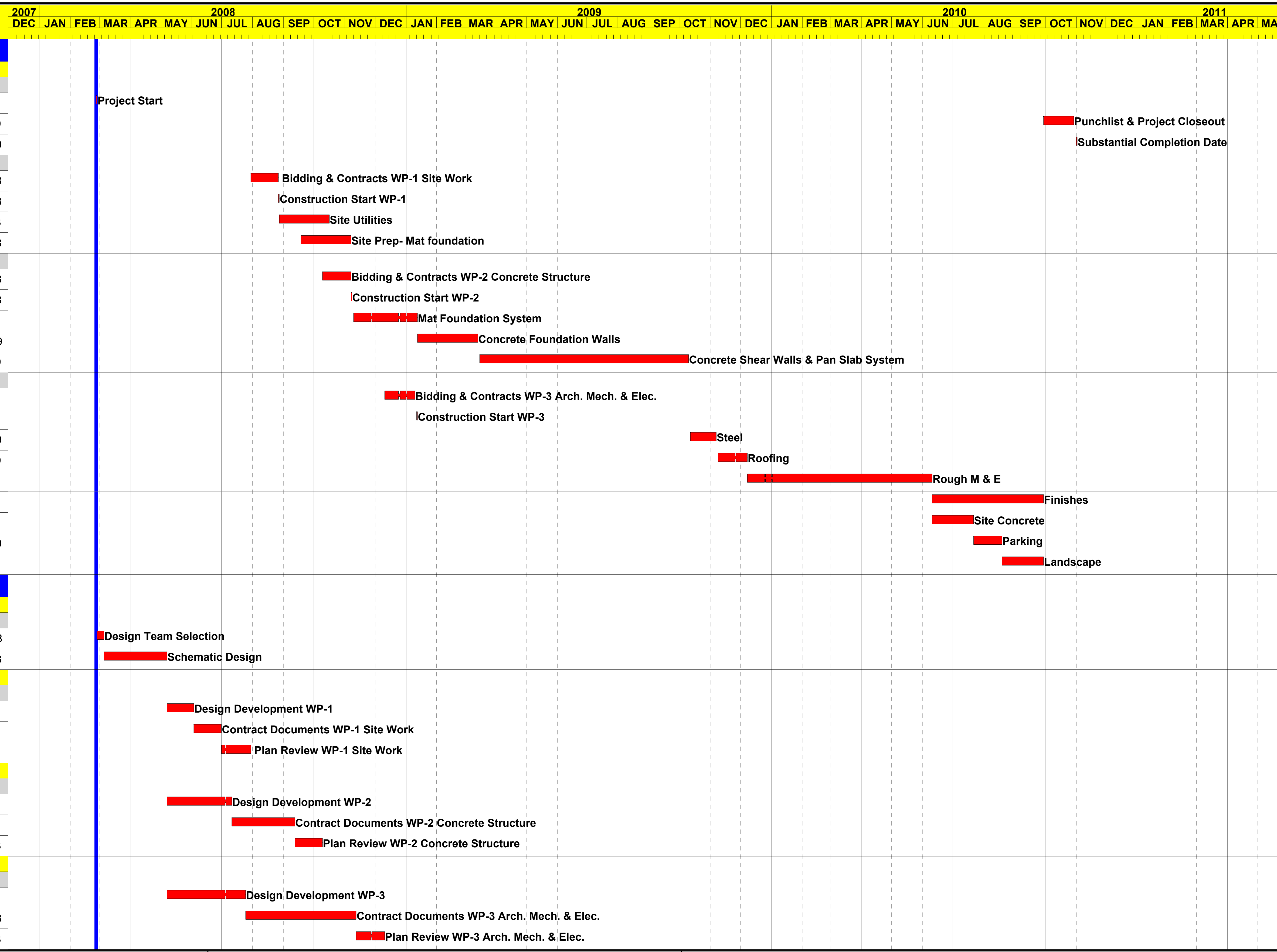
Shear Wave Velocity Calculations

Job No.: 08-310
Client: AMEC
CPT No.: CPT-01
Location USU USTAR Building
Date: 1/18/08

Geophone Offset (m): 0.20
Source Offset (18") (m): 0.46

Test Depth (m)	Geophone Depth (m)	Ray Path (m)	Incremental Distance (m)	Time Interval (ms)	Interval Velocity (m/s)	Interval Depth (m)	Interval Velocity (ft/s)	Interval Depth (ft)
0.80	0.60	0.76						
3.80	3.60	3.63	2.87	11.19	257	2.10	842	6.9
6.80	6.60	6.62	2.99	16.07	186	5.10	610	16.7
9.80	9.60	9.61	3.00	14.49	207	8.10	678	26.6
12.80	12.60	12.61	3.00	13.21	227	11.10	744	36.4
15.80	15.60	15.61	3.00	10.15	295	14.10	969	46.2

Activity ID	Activity Description	Orig Dur	Early Start	Early Finish
CM/GC				
Project				
100	Project Start	1	27FEB08	27FEB08
410	Punchlist & Project Closeout	22	30SEP10	29OCT10
430	Substantial Completion Date	1	01NOV10	01NOV10
Work Package 1 Site work				
142	Bidding & Contracts WP-1 Site Work	20	30JUL08	26AUG08
200	Construction Start WP-1	1	27AUG08	27AUG08
210	Site Utilities	36	28AUG08	16OCT08
220	Site Prep- Mat foundation	36	18SEP08	06NOV08
Work Package 2 Concrete Stucture				
152	Bidding & Contracts WP-2 Concrete	20	10OCT08	06NOV08
250	Construction Start WP-2	1	07NOV08	07NOV08
260	Mat Foundation System	43	10NOV08	12JAN09
310	Concrete Foundation Walls	44	13JAN09	13MAR09
320	Concrete Shear Walls & Pan Slab System	150	16MAR09	09OCT09
Work Package 3 Arch. Mech. & Elec.				
162	Bidding & Contracts WP-3 Arch. Mech. &	20	11DEC08	09JAN09
300	Construction Start WP-3	1	12JAN09	12JAN09
340	Steel	20	12OCT09	06NOV09
350	Roofing	20	09NOV09	07DEC09
360	Rough M & E	130	08DEC09	09JUN10
370	Finishes	80	10JUN10	29SEP10
380	Site Concrete	30	10JUN10	21JUL10
390	Parking	20	22JUL10	18AUG10
400	Landscape	30	19AUG10	29SEP10
Design Team				
Design				
110	Design Team Selection	5	28FEB08	05MAR08
120	Schematic Design	45	06MAR08	07MAY08
Work Package 1 Site Work				
130	Design Development WP-1	18	08MAY08	02JUN08
140	Contract Documents WP-1 Site Work	20	03JUN08	30JUN08
141	Plan Review WP-1 Site Work	20	01JUL08	29JUL08
Work Package 2 Concrete Structure				
131	Design Development WP-2	45	08MAY08	10JUL08
150	Contract Documents WP-2 Concrete	45	11JUL08	11SEP08
151	Plan Review WP-2 Concrete Structure	20	12SEP08	09OCT08
Work Package 3 Arch. Mech. & Elec.				
132	Design Development WP-3	55	08MAY08	24JUL08
160	Contract Documents WP-3 Arch. Mech. &	78	25JUL08	11NOV08
161	Plan Review WP-3 Arch. Mech. & Elec.	20	12NOV08	10DEC08



Start Date	27FEB08
Finish Date	01NOV10
Must Finish Date	01NOV10
Data Date	27FEB08
Run Date	17FEB08 16:07

Early Bar
Float Bar
Progress Bar
Critical Activity

Life Science Research Center-USTAR
Utah State University
Gramoll Construction CM / GC