GORDON
GEOTECHNICAL
ENGINEERING, INC.

REPORT
GEOTECHNICAL STUDY AND SLOPE STABILITY ANALYSIS
PROPOSED WASATCH ROCK DEVELOPMENT
6695 WASATCH BOULEVARD
COTTONWOOD HEIGHTS, UTAH

May 13, 2020

Job No. 528-005-20

Prepared for:
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Rockworth Companies
4655 South 2300 East, Suite 205
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Attention: **Mr. Josh Cowley**

Ladies and Gentlemen:

Re: Report
Geotechnical Study and Slope Stability Analysis
Proposed Wasatch Rock Development
6695 Wasatch Boulevard
Cottonwood Heights, Utah

1. INTRODUCTION

1.1 GENERAL

This report presents the results of our geotechnical study and slope stability analysis performed at the site of the proposed Wasatch Rock development which is located at 6695 Wasatch Boulevard in Cottonwood Heights, Utah. The general location of the site with respect to major topographic features and existing facilities, as of 1998, is presented on Figure 1, Vicinity Map. A detailed location of the site showing existing roadways and surrounding facilities, on an air photograph base, is presented on Figure 2, Area Map. The locations and alignments of photographs taken of the site during the field portion of study are also shown on Figure 2. A more detailed layout of the site showing the existing topography and proposed structure locations based on the conceptual site plan by McNeil Engineering dated April 24, 2020 is presented on Figure 3, Site Plan. The locations of the exploration test pits and borings excavated in conjunction with this study as well as a previous study are also presented on Figure 3.

An updated Geologic Hazards Evaluation was performed in conjunction with this geotechnical study and slope stability analysis and is included with Appendix A of this report. The locations of the fault trenches are shown within the Geologic Hazards report.

1.2 BACKGROUND

Engineers at Gordon Geotechnical Engineering, Inc. (G^2) previously completed a geotechnical study for the site summarized in letters dated September 11, 2009[1] and February 3, 2012[2]. In concurrence with these studies, a surface fault rupture hazard study was performed for the site dated June 10, 2009[3]. G^2 has also reviewed a previous geotechnical study for the site, by others, dated July 29, 2016[4].

This report also incorporates data collected by G^2 in 2018 while investigating potential sources of fill material for Rockworth Companies. The logs of test pits and laboratory data was never summarized in a report. The conclusions were transmitted verbally to the client.

1.3 OBJECTIVES AND SCOPE

The objectives and scope of our study were planned in discussions between Mr. Josh Cowley of Rockworth Companies and Mr. Patrick Emery of G^2.

In general, the objectives of this study were to:

1. Accurately define and evaluate the general subsurface soil and groundwater conditions across the site.
2. Provide foundation, earthwork, floor slab, pavement, drainage, slope stability, and geoseismic recommendations and parameters to be utilized in the design and construction of the proposed facilities.

In accomplishing these objectives, our scope has included the following:

1. A field program consisting of the excavating and drilling, logging, and sampling of eight test pits and three borings, respectively.

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[1] "Summary Letter, Geotechnical/Geoseismic Study, A.J. Dean Property, East Side of Wasatch Boulevard at Approximately 6700 South, Cottonwood Heights, Salt Lake County, Utah," GSH Job No. 0883-001-09.

[2] "Supplemental Discussions and Recommendations, Earthwork and Initial Aspects of Proposed Commercial Development, A.J. Dean Property, East Side of Wasatch Boulevard at Approximately 6700 South, Cottonwood Heights, Salt Lake County, Utah," G^2 Job No. 028-001-12.

[3] "Report, Surface Fault Rupture Hazard Study, A.J. Dean Property, East Side of Wasatch Boulevard at Approximately 6700 South, Cottonwood Heights, Salt Lake County, Utah," Western Geologic Job No. 2381.

[4] "Geotechnical Study, Liberty Mountain, 6695 South Wasatch Boulevard, Salt Lake City, Utah", GSH Job No.: 0283-015-16, Dated July 29, 2016.

2. ReMi Survey to develop a shear wave velocity profile to 100 feet for IBC Site Class determination.

3. A laboratory testing program.

4. An office program consisting of the correlation of available data, engineering analyses, and the preparation of this summary report.

1.4 AUTHORIZATION

Authorization was provided by returning a signed copy of our Professional Services Agreement No. 20-0205-rev1 dated February 13, 2020.

1.5 PROFESSIONAL STATEMENTS

Supporting data upon which our recommendations are based are presented in subsequent sections of this report. Recommendations presented herein are governed by the physical properties of the soils encountered in the exploration borings, test pits, and trenches, measured and projected groundwater conditions, and the layout and design data discussed in Section 2., Proposed Construction, of this report. If subsurface conditions other than those described in this report are encountered and/or if design and layout changes are implemented, G^2 must be informed so that our recommendations can be reviewed and amended, if necessary.

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

2. PROPOSED CONSTRUCTION

Development plans for the site have changed since the previous geotechnical reports for the site. Development at the site is complicated by the existence of several fault lines and a buried aqueduct which render significant portions of the site as “un-buildable” for habitable structures. These fault lines and buried aqueduct have been considered from the onset when designing the latest development plans. The proposed structures have been strategically located in the “buildable” areas defined in the surface fault rupture hazard report.

Currently, a hotel, an apartment, condominium, senior living center, three mixed-use pads, and three retail pads are planned for the site. Additionally, Wasatch Boulevard along the western boundary of the site will be re-aligned to bi-sect the site in a generally north-south direction.

<u>Apartment Structure</u>

An apartment structure is planned for the eastern portion of the site. The proposed apartment structure will consist of a two-level reinforced concrete parking structure with four- to five-levels of wood-frame apartments on top. The lowest level of the parking structure will be established at an elevation of 4,480 feet. Due to the extremely variable topography in the area, creating a pad for this structure will require cuts on the order of 15 to 20 feet and fills on the order of 15 to 20 feet.

Maximum column and wall loads for the apartment structure are anticipated to be on the order of 120 to 500 kips and 5 to 15 kips per lineal foot, respectively.

Condominium Structure

A condominium structure is planned for the north side of the site. The proposed condominium structure will consist of a 5-level reinforced concrete parking structure with 10-levels of steel-frame residential space on top. The lowest level of the parking structure will be established at an elevation of 4,893.2 feet. The mass excavation for this structure will need to extend to depths ranging from 25 to 50 feet below existing grades. The final site grading around the structure will require fills on the order of 10 to 15 feet.

Maximum column and wall loads for the condominium structure are anticipated to be very large, on the order of 1,000 to 1,500 kips and 15 to 30 kips per lineal foot, respectively. Detailed structural loads will be needed to finalize geotechnical recommendations for this structure.

Due the high loads, we anticipate that the condominium may be supported upon a continuous mat. If an average real uniform load of 145 pounds per square foot (bearing load plus frequently applied load) is imposed by each floor, an average uniform load of 75 pounds per square foot is imposed by the roof and the structure is supported upon a three and one-half foot thick mat, the pressure imposed by the base of the mat will be on the order of 2,500 to 2,800 pounds per square foot.

Senior Living Center

A senior living center is planned for the southeast corner of the site. The proposed senior living structure will consist of a one-level reinforced concrete parking with two- to three-levels of wood-frame residential space on top. The lowest level of the parking structure will be established at an elevation of 4,853 feet. Creating a pad at the bottom of footing elevation for this structure will require fills on the order of 1 to 15 feet.

Maximum column and wall loads for the senior living structure are anticipated to be on the order of 120 to 250 kips and 5 to 10 kips per lineal foot, respectively.

Hotel

A hotel is planned for the northwest corner of the site. The proposed hotel structure will consist of four-levels of wood-framing established slab-on-grade. The lowest level of the parking structure will be established at an elevation of 4,843 feet. The pad for this structure will be established near existing grade; however, the contour maps indicate fills up to 10 or 15 feet in height may need to be removed from the area. It should be noted that the existing topography has likely changed due to the gravel pit operations.

Maximum column and wall loads for the hotel structure are anticipated to be on the order of 90 to 180 kips and 5 to 10 kips per lineal foot, respectively.

Mixed-Use and Retail Structures

Mixed-use structures are planned for Pads B and C and along the western portion of the site. Retail structures are planned for Pads A, D, and E in the central portion of the site. The mixed-use structures will be three levels in height and the retail structures will be one level in height. These structures will be of wood-frame construction established slab-on-grade. Generally these pads will require cuts and fills on the order of 5 to 10 feet.

Maximum column and wall loads for the mixed-use structures are anticipated to be on the order of 90 to 180 kips and 5 to 7 kips per lineal foot, respectively.

Final site grading will require cuts up to 30 feet in the northern portions of the site and fills on the order of 5 to 10 feet in the southwest portion of the site. Fills up to 35 feet in height are planned for the eastern portion of the site and will buttress the existing gravel pit cut slope. Final site grading slopes are generally not anticipated to not exceed 50 percent or 2.0:1.0 (H:V) with localized areas of up to 56 percent or 1.8:1.0 (H:V).

3. INVESTIGATIONS

3.1 FIELD PROGRAM

In order to define and evaluate the subsurface soil and groundwater conditions across the site, during this study, 3 borings were drilled to depths of 41 to 101 feet below existing grade utilizing a truck-mounted drill rig equipped ODEX percussion drilling methods. Additionally, 8 test pits were excavated to depths of 7 to 23 feet below existing grade. One trench approximately 210 feet in length and 5 to 15 feet below existing grade was excavated for the updated Geologic Hazards Evaluation. The approximate locations of the borings and test pits are presented on Figure 3. Additionally, the locations of borings drilled in conjunction with previous studies are also shown on Figure 3.

The field portion of our study was under the direct control and continual supervision of an experienced member of our geotechnical staff. During the course of the excavation operations, a continuous log of the subsurface conditions encountered was maintained. In addition, samples of the typical soils encountered were obtained for subsequent laboratory testing and examination. The soils were classified in the field based upon visual and textural examination. These classifications have been supplemented by subsequent inspection and testing in our laboratory. Detailed graphical representation of the subsurface conditions encountered is presented on Figures 4A through 4K, Log of Borings and Test Pits. Soils were classified in accordance with the nomenclature described on Figure 5, Unified Soil Classification System.

A 3.25-inch outside diameter, 2.42-inch inside diameter drive sampler (Dames & Moore) was utilized in the majority of the subsurface sampling at the site. Additionally, a 2.0-inch outside diameter, 1.38-inch inside diameter drive sampler (SPT) was utilized at select locations and depths. The blow counts recorded on the boring logs were those required to drive the sampler 12 inches with a 140-pound hammer dropping 30 inches.

Disturbed bag samples were collected from the soils brought up by the backhoe bucket. Additionally, relatively undisturbed samples of the finer-grained sand type soils encountered in the continuous trench excavated in conjunction with the geologic hazards study were obtained using 2.42-inch inside diameter hand sampling equipment.

Following completion of excavating and logging, each test pit and continuous trench was backfilled. Although an effort was made to compact the backfill with the backhoe, backfill was not placed in uniform lifts and compacted to a specific density. Consequently, settlement of the backfill with time is likely to occur.

3.2 ReMi SURVEY

3.2.1 General

Refraction microtremor (ReMi) is a geophysical survey developed by Dr. John N. Louie and explained in detail in his document “Faster, Better: Shear-Wave Velocity to 100 Meters Depth from Microtremor Arrays”[5].

One survey line, measuring 88 meters (288.7 feet) in length, running approximately east-to-west in the central portion of the site was performed at the site. An array of 12, 4.5 hertz geophones spaced at 8-meter intervals was attached to the line. The geophones, attached to metal spikes, were firmly planted in the surficial soils and connected, via the line, to a DAQ 3 – 24 Channel Seismograph. This system was used to digitally record the seismic wave vibrations at each geophone position with 32-bit analog to digital conversion

3.2.2 Data Evaluation

Following the acquisition of the data, using *SeisOpt ReMi* software, a wavefield transformation of the records reveals the shear-wave dispersion curve (Appendix B). This dispersion curve plots frequency, in hertz, against slowness, in seconds per meter. The shear-wave dispersion

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[5] Louie, J.N., 2001, “Faster, Better: Shear-Wave Velocity to 100 Meters Depth from Refraction Microtremor Arrays”, Bulletin of Seismological Society of America, Vol 91, 347-364.

curve from the wavefield transformation is then manually picked and the picks modeled to determine the subsurface shear-wave velocity profile (see Appendix B).

The results of the tests are tabulated below:

Line	V_{s100} (ft/s)
1	1549.0

3.3 LABORATORY TESTING

3.3.1 General

In order to provide data necessary for our engineering analyses, a laboratory testing program was performed. The program included moisture and density, partial gradation, Atterberg limits, compaction, consolidation, and direct shear tests. The following paragraphs describe the tests and summarize the test data.

3.3.2 Moisture and Density Tests

To aid in classifying the soils and to help correlate other test data, moisture and density tests were performed on selected undisturbed samples. The results of these tests are presented on the boring and test pit logs, Figures 4A through 4K.

3.3.3 Partial Gradation Test

To aid in classifying the soils and to provide general index parameters, a partial gradation test was performed upon representative samples of potential sources of fill as well as the soils encountered in the exploration borings. The results of the tests are tabulated on the following page.

Potential Fill Sources

Test Pit/ Sample No.	Depth (feet)	Percent Passing (Sieve Size)							Soil Classification
		(1 1/2")	(3/4")	(1/2")	(#4)	(#10)	(#40)	(#200)	
TP-3	11.0	-	-	-	96.7	95.2	91.5	67.8	Sandy CL (Red Clay)
TP-5	2.0	-	-	-	-	85.7	81.6	59.3	CL/SC (Washout Fines)
TP-5	4.0	-	-	-	-	97.7	96.5	70.6	CL (Washout Fines)
TP-5	9.0	-	-	-	-	10.4	5.5	4.6	SP ("Squeegee")
TP-5	BULK	-	-	-	-	88.3	85.3	65.3	Sandy CL (Washout Fines)
TP-6	3.0	-	-	-	-	95.7	95.1	92.7	CL (Washout Fines)
TP-6	6.0	-	-	-	-	94.4	93.9	69.3	Sandy CL (Washout Fines)
TP-6	12.0	-	-	-	-	15.2	7.5	4.9	SP ("Squeegee")
TP-6	BULK	-	-	-	-	89.5	87.7	68.7	Sandy CL (Washout Fines)
TP-7	5.0	-	-	-	-	92.4	91.1	62.7	Sandy CL (Washout Fines)
TP-7	BULK	-	-	-	-	98.2	96.6	67.9	Sandy CL (Washout Fines)
TP-8	2.0	-	-	-	-	75.5	70.4	51.9	CL/SC (Washout Fines)
TP-8	6.0	-	-	-	-	82.9	76.5	53.7	CL/SC (Washout Fines)
Shelby Stockpile (1A)	-	87.6	58.7	47.2	38.5	32.7	18.9	3.3	SP/GP (Bank Run)
Shelby Stockpile (1B)	-	73.7	73.7	72.7	71.1	69.0	52.7	17.2	SM (Bank Run)
Shelby Stockpile (Composite)	-	-	-	89.9	82.3	76.0	48.5	15.3	SM (Bank Run)

Borings (This Study)

Boring No.	Depth (feet)	Percent Passing No. 4 Sieve	Percent Passing No. 200 Sieve	Soil Classification
B-1	7.5	-	35.2	SM
B-1	10.0	78.2	3.0	SP
B-1	30.0	60.3	10.7	SP-SM
B-1	40.0	75.2	18.5	SM
B-2	2.5	34.5	9.8	GP-GM
B-2	10.0	34.2	4.6	GP
B-2	15.0	41.5	10.2	GP-GM
B-2	25.0	39.0	5.9	GP-GM
B-2	45.0	75.2	66.3	CL
B-3	36.5	28.5	3.9	GP
B-3	55.0	48.2	4.6	SP/GP
B-3	60.0	44.7	6.5	GP-GM
B-3	80.0	74.9	18.2	SM
B-3	85.0	49.0	10.4	GP-GM
B-3	95.0	3.2	0.9	SP

3.3.4 Atterberg Limits Tests

An Atterberg limits test was performed upon representative samples of the potential fill sources and soils encountered in the exploration borings. Results are tabulated on the following page.

Potential Fill Sources

Test Pit No.	Depth (feet)	Liquid Limit (percent)	Plastic Limit (percent)	Plasticity Index (percent)	Unified Soils Classification
TP-3	11.0	24	15	9	CL (Red Clay)
TP-5	BULK	30	20	10	CL (Washout Fines)

Borings (This Study)

Boring No.	Depth (feet)	Liquid Limit (percent)	Plastic Limit (percent)	Plasticity Index (percent)	Unified Soils Classification
B-2	45.0	35	21	14	CL

3.3.5 Compaction Tests

To determine the compaction properties of the potential fill sources, a Modified Proctor compaction test was performed on several bulk samples. Results of the tests are tabulated below:

Test Pit/Sample No.	Depth (feet)	Soil Classification	Maximum Dry Density (pcf)	Optimum Moisture Content (percent)
1A and 1B Composite	-	SM	126.8	6.5
Washout Fines Composite	-	CL	124.3	10.8
Red Clay Composite	-	CL	123.3	10.7

3.3.6 Consolidation Tests

To determine the load deformation and consolidation characteristics of the typical fine-grained soils encountered for our settlement analysis, a consolidation test was performed on one relatively undisturbed sample obtained during our field program. A consolidation test was also run a sample of laboratory compacted washout fines.

The test results are tabulated on the following page.

Boring/ Sample No.	Depth (feet)	Soil Classification	Dry Density (pcf)	Moisture Content (percent)	Preconsolidation Pressure (psf)
B-1	20.0	CL	97.8	24.3	2,400*
Washout Fines Composite**	-	CL	117.7	11.1	13,000*

* Determined by the Casagrande Graphical Method

** Recompacted to 95 percent of the Modified Proctor Density

Data available indicates that the fine-grained cohesive soils are lightly to moderately over-consolidated. When loaded below the preconsolidation pressure, the soils will exhibit moderate compressibility characteristics. Detailed results of the tests are maintained within our files and can be transmitted to you, at your request.

3.3.7 Direct Shear Tests

Direct shear tests were performed on representative laboratory compacted samples of potential fill sources as well as undisturbed samples of the soils encountered in the exploration borings. The results of the direct shear test are tabulated below:

Boring/ Sample No.	Depth (feet)	Soil Classification	Friction Angle	Cohesion (psf)
Shelby Stockpile (Composite)	-	SM	38	499
Washout Fines (Composite)	-	CL	33	356
B-2	35.0	CL	27	166
B-2	40.0	SM	35	524
B-3	75.0	SM	33	0

A detailed report of the direct shear test results is provided in Appendix C.

4. SITE CONDITIONS

4.1 SURFACE

The site consists of a 21.56-acre parcel located on the southeast corner of the intersection of Wasatch Boulevard and 6200 South Street. The site has been used as a gravel pit since the 1960's. Significant cuts for mining of sand and gravel have occurred over the years as the operation progressed into the hillside. Additionally significant amounts of waste material (washout fines, concrete washout, and "squeegee" material) have been placed around the site. A severely over-steeped slope is present on the eastern portion of the site due to the significant cuts in the area.

Several at-grade structures associated with the gravel pit operations as well as stockpiles of material are present on the site. Vegetation is limited to a sparse growth of ankle- to knee-high weeds and grasses and occasional small trees.

The topography across the site is quite variable and has changed over the years due to the on-going gravel pit operations. The overall topography in the area generally slopes down to the southwest. Overall total topographic relief across the site is on the order of 90 to 160 feet.

Representative photographs of the site area are shown on Figure 6, Photographs.

4.2 SUBSURFACE SOIL

Subsurface soil conditions encountered in the exploration borings, test pits, and trenches were relatively consistent. The dominant soil type at the site is fine to coarse sands and gravels with varying amounts of silt. The sand and gravel soils are generally dense to very dense, slightly moist, light brown to tan in color, and will exhibit high strength and low compressibility characteristics.

The sand and gravel sequence contains interbedded layers of thinly layered silty fine sand encountered within Boring B-1 between 33.5 and 41.0 feet below existing grade, Boring B-2 between 38.0 and 43.0 feet below existing grade, and Boring B-3 between 74.0 and 77.5 feet below existing grade at the boring locations. The silty sand soils are medium dense to dense, slightly moist to saturated, light brown to tan in color, and will exhibit moderate strength and compressibility characteristics.

The sand and gravel sequence also contains interbedded layers of silty clay and fine sandy clay encountered within Boring B-1 between 17.5 and 24.5 feet below existing grade, and Boring B-2 between 32.5 and 38.0 feet, and between 43.0 and 51.5 feet below existing grade at the boring locations. The silty/sandy clay soils are stiff to very stiff, moist to saturated, tan to gray in color, and will exhibit moderate strength and compressibility characteristics.

The lines designating the interface between soil types on the borings and test pit logs generally represent approximate boundaries. In-situ, the transition between soil types may be gradual.

4.3 GROUNDWATER

Immediately following drilling operations, groundwater was measured in the exploration borings. On March 25, 2020 we returned to the site and measured the groundwater within the piezometers placed in the borings. Groundwater measurements are tabulated below:

	Groundwater Depth (feet)		
Boring No.	**March 2, 2020**	**March 13, 2020**	**March 25, 2020**
B-1	32.5*		36.0
B-2	29.0*		30.8
B-3	--	Not Encountered	Not Encountered

* Measured at the end of drilling operations, not yet stabilized.

Seasonal and longer-term groundwater fluctuations on the order of one to two feet are projected, with the highest seasonal levels generally occurring during the late spring and early summer months.

5. DISCUSSIONS AND RECOMMENDATIONS

5.1 SUMMARY OF FINDINGS

<u>The results of our geotechnical and geologic hazard study indicate that the site is suitable for the proposed development provided the recommendations in this report are followed:</u>

The most significant geotechnical aspects of the site are:

1. Active Normal Faulting observed in fault study trenches.

   Several splays of the Wasatch Fault were encountered in the fault study trenches. The building layout has been designed to account for the setback recommendations outlined in the geologic hazard report for the site. The geologic hazard report is included in Appendix A

2. Stability of the proposed slopes.

   The results of the slope stability analysis indicate that the proposed slopes will meet the required factors of safety provided that the following recommendations are followed:

   - The condominium structure at Section A-A’ incorporates a deep cut for below-grade parking. A structural element must extend a minimum of 15 feet below the bottom of footings to assure an adequate factor of safety. This may consist of deep foundations, soil improvement, or a permanent shoring solution such as soil nails.

   - Section B-B’ indicates that concrete washout material may remain in place provided that any loose or raveling material is removed and the concrete washout is competent.

   - The slope at Section C-C’ is the steepest with an average grade of approximately 53 percent. The stability analysis indicates that compacted bank run sand and gravel fill material will be required for slopes that exceed 50 percent or 2:1 (H:V).

   - Compacted washout fines may be utilized for slopes that do not exceed 50 percent or 2:1 (H:V).

   - It is recommended that all fill slopes on the northern and eastern portions of the site incorporate subdrains near the toe of the existing slopes to intercept seepage from up-gradient runoff.

   - Fill slopes must be benched into the existing slope as fill placement progresses to avoid a planar interface at the base of the fill. Individual benches may be on the order of five feet in height.

   - Fill materials must be compacted to a minimum of 90 percent of the Modified Proctor dry density.

3. Non-engineered fills encountered to depths of 2.0 to 6.5 feet in the majority of the building areas and up to 15.0 feet in the area of the proposed condominium.

   Non-engineered fills are not suitable for building support and must be completely removed from below the building footprint and rigid pavement areas.

4. Deep cuts required for the condominium structure.

   The condominium structure will require cuts up to 50 feet in depth. Due to the adjacent roadways this will likely require shoring to maintain stability of the sideslopes. Temporary shoring may potentially consist of a soil nail wall provided that permission is granted to extend nails below adjacent properties (if needed). As an alternative, a permanent shoring wall consisting of a solider pile wall with tiebacks may be considered. A permanent shoring wall would have the added benefit of significantly reducing the lateral pressures on the below-grade walls. Additionally, our slope stability analysis indicates that at Cross-section A-A' a structural element must extend a minimum of 15 feet below the bottom of footings to force the potential failure plane deeper and assure an adequate factor of safety. This requirement should be accounted for in the shoring design.

5. Potential for "perched" groundwater conditions and groundwater seepage through the hillside.

   Due to the potential for perched groundwater conditions, subdrains are behind all below-grade structural elements.

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Detailed discussions pertaining to slope stability, earthwork, foundations, floor slabs, lateral resistance, pavement, and the geoseismic setting of the site are discussed in the following sections.

5.2 SLOPE STABILITY

5.2.1 General

In order to evaluate the stability of the proposed slopes at the site, a slope stability analysis was performed with the computer program, SLIDE (Version 6.0), utilizing the modified Bishops method for a circular failure surfaces. The analysis included both long-term static and seismic conditions of the proposed site grading and development.

5.2.2 Geometry

The geometry for the slope stability models was developed from the geologic cross-sections provided with the concurrent Geologic Hazards Evaluation report. Topography was obtained from 2013 lidar data with 0.5-meter resolution. Three cross-sections (A-A', B-B', and C-C') for slope stability analysis were selected based on the locations of the proposed developments and the most adverse topographic and geologic conditions.

The locations and elevations of the proposed structures were obtained from the site grading plans by McNeil Engineering. The topography from the 2013 LiDAR data set was modified to show the proposed cuts for the structures and proposed site grading fills.

The subsurface profile was developed utilizing stratigraphic information obtained from numerous borings, test pits, and trenches.

5.2.3 Soil Strength

The soil parameters were selected for analysis based upon direct shear test results performed on undisturbed and laboratory recompacted samples. Strength parameters for the more coarse-grained granular soils were selected based upon our experience with similar soils in the area. These coarse-grained sand and gravel soils are projected to exhibit relatively high strengths based on their performance history in gravel pit cut slopes which have been known to stand near-vertical for extended periods of time. The cohesive characteristic of these granular soils may be explained by a slight cementation and interlocking of particles. Parameters of concrete washout are estimated as a hybrid between high strength soil and low-grade concrete.

The following table summarizes the soil strength values utilized for static and seismic conditions:

Soil Type	Soil Parameter	Parameter Units
Lacustrine Sand and Gravel	Cohesion	200 (psf)
	Friction Angle	36
	Unit Weight	120 (pcf)
Laminated Silty Fine Sand Beds	Cohesion	0 (psf)
	Friction Angle	33
	Unit Weight	120 (pcf)
Lacustrine Fines	Cohesion	150 (psf)
	Friction Angle	27
	Unit Weight	120 (pcf)
Site Grading Fill (Compacted Washout Fines)	Cohesion	350 (psf)
	Friction Angle	33
	Unit Weight	120 (pcf)
Site Grading Fill (Compacted Sand and Gravel)	Cohesion	250 (psf)
	Friction Angle	38
	Unit Weight	120 (pcf)
Concrete Washout	Cohesion	500 (psf)
	Friction Angle	37
	Unit Weight	130 (pcf)

5.2.4 Analysis Results

The results of the stability analyses are tabulated below:

Profile	Condition	Seismic Coefficient	Lowest Factor of Safety	Recommended Minimum Allowable Factor of Safety
A-A’	Static	--	1.70	1.5
A-A’	Seismic	0.3*	1.00	1.0
B-B’	Static	--	2.04	1.5
B-B’	Seismic	0.3*	1.14	1.0
C-C’	Static	--	1.90	1.5
C-C’	Seismic	0.3*	1.07	1.0

* Approximately one-half of the geometric mean PGA.

The results of the slope stability analysis indicate that the proposed slopes will meet the required factors of safety provided that the following recommendations are followed:

- The condominium structure at Section A-A’ incorporates a deep cut for below-grade parking. A structural element must extend a minimum of 15 feet below the bottom of footings to assure an adequate factor of safety. This may consist of deep foundations, soil improvement, or a permanent shoring solution such as soil nails.

- Section B-B’ indicates that concrete washout material may remain in place provided that any loose or raveling material is removed and the concrete washout is competent.

- The slope at Section C-C’ is the steepest with an average grade of approximately 53 percent. The stability analysis indicates that compacted bank-run sand and gravel fill material will be required for slopes that exceed 50 percent or 2:1 (H:V).

- Compacted washout fines may be utilized for slopes that do not exceed 50 percent or 2:1 (H:V).

- It is recommended that all fill slopes on the northern and eastern portions of the site incorporate subdrains near the toe of the existing slopes to intercept seepage from up-gradient runoff.

- Fill slopes must be benched into the existing slope as fill placement progresses to avoid a planar interface at the base of the fill. Individual benches may be on the order of five feet in height.

- Fill materials must be compacted to a minimum of 90 percent of the Modified Proctor dry density.

5.2.5 Surficial Stability Analysis Results

Slope stability analysis results are presented in graphical form and have been enclosed in Appendix D, Slope Stability Analysis Results.

1. Surficial Stability Analysis Results

Considering the long-term performance the proposed slopes, and to account for periods of high snow melt and rainfall, a surficial stability analysis of the proposed slopes was performed. The analysis assumes an infinite slope with seepage parallel to the slope. The assumed depth of saturation is four feet.

For slopes with an average grade up to 53 percent constructed of bank-run sand and gravel, considering a slight reduction in cohesion due to low confinement pressure, the surficial factor of safety is 1.52.

For slopes with an average grade up to 50 percent constructed of compacted washout fines, considering a slight reduction in cohesion due to low confinement pressure, the surficial factor of safety is 1.87.

For additional long-term protection from erosion, we recommend that erosion control measures be implemented, such as seeding, erosion control mats, terraces, tracking, or other erosion control measures.

5.3 EARTHWORK

5.3.1 Site Preparation

Preparation of the site must consist of the removal of all non-engineered fills, loose surficial soils, topsoil, debris, and other deleterious materials from beneath an area extending at least five feet beyond the perimeter of the proposed building, rigid pavement, and exterior flatwork areas.

The non-engineered fills may remain in flexible pavement areas as long as they are properly prepared. Proper preparation will consist of scarifying and moisture conditioning the upper eight inches and recompacting to the requirements of structural fill. However, it should be noted that compaction of fine-grained soils (if encountered) as structural site grading fill will be very difficult, if not impossible, during wet and cold periods of the year. As an option for proper preparation and recompaction, the upper eight inches of the non-engineered fills may be removed and replaced with granular subbase over proofrolled subgrade. Even with proper preparation, flexible pavements established on non-engineered fills may experience some long-term movements. If the possibility of these movements is not acceptable, these non-engineered fills must be completely removed.

Subsequent to the above operations and prior to the placement of footings, structural site grading fill or floor slabs, the exposed natural subgrade must be proofrolled by passing moderate-weight rubber tire-mounted construction equipment over the surface at least twice. If any loose, soft, or disturbed zones are encountered, they must be completely removed in footing and floor slab areas and replaced with granular structural fill. If removal depth required is greater than two feet, G^2 must be notified to provide further recommendations. In pavement areas, unsuitable soils encountered during recompaction and proofrolling must be removed to a maximum depth of two feet and replaced with compacted granular structural fill.

5.3.2 Temporary Excavations

Temporary construction excavations through granular soil, not exceeding four feet in depth, above or below the groundwater table, may be constructed with near-vertical sideslopes. Deeper excavations in granular soils, not exceeding 12 feet above or below the water table, should be constructed with sideslopes no steeper than one horizontal to one vertical (1.0H:1.0V). Excavations in granular soils not exceeding 30 feet, should be constructed with sideslopes no steeper than one and one-half horizontal to one vertical (1.5H:1.0V). Excavations deeper than 30 feet will require shoring. If clean granular soils are encountered, or if excessive sloughing occurs, the sideslopes must be flattened. Loose and raveling soils are anticipated.

All excavations must be inspected periodically by qualified personnel. If any signs of instability or excessive sloughing are noted, immediate remedial action must be initiated.

5.3.3 Structural Fill

Structural fill is defined as all fill which will ultimately be subjected to structural loadings, such as imposed by footings, floor slabs, pavements, etc. Structural fill will be required as backfill over foundations and utilities, as site grading fill, and in some areas, as replacement fill below footings. All structural fill must be free of sod, rubbish, topsoil, frozen soil, and other deleterious materials. Structural site grading fill is defined as fill placed over fairly large open areas to raise the overall site grade. For structural site grading fill, the maximum particle size should generally not exceed four inches; although, occasional larger particles, not exceeding six inches in diameter may be incorporated if placed randomly in a manner such that "honeycombing" does not occur and the desired degree of compaction can be achieved. The maximum particle size within structural fill placed within confined areas should generally be restricted to two inches.

The on-site non-engineered fills and natural granular soils may be utilized as structural site grading fill. It should be noted that unless moisture control is maintained, utilization clayey soils as structural site grading fill will be very difficult, if not impossible, during wet and cold periods of the year. Only granular soils are recommended as structural fill in confined areas, such as around foundations and within utility trenches.

Non-structural site grading fill is defined as all fill material not designated as structural fill and may consist of any cohesive or granular soils not containing excessive amounts of degradable material.

5.3.4 Fill Placement and Compaction

Coarse gravel and cobble mixtures (stabilizing fill), if utilized, shall be end-dumped, spread to a maximum loose lift thickness of 15 inches, and compacted by dropping a backhoe bucket onto the surface continuously at least twice. As an alternative, the fill may be compacted by passing moderately heavy construction equipment or large self-propelled compaction equipment over the area at least twice. Subsequent fill material placed over the coarse gravels and cobbles shall be adequately placed so that the "fines" are "worked into" the voids in the underlying coarser gravels and cobbles.

All other structural fill shall be placed in lifts not exceeding eight inches in loose thickness. Structural fills shall be compacted in accordance with the percent of the maximum dry density as determined by the AASHTO[6] T-180 (ASTM D-1557) compaction criteria in accordance with the table below:

Location	Total Fill Thickness (feet)	Minimum Percentage of Maximum Dry Density
Beneath an area extending at least 3 feet beyond the perimeter of the structure	0 to 8	95
Beneath an area extending at least 5 feet beyond the perimeter of the structure	8 to 15	98
Outside area defined above	0 to 15	90
Slope Buttressing Fill	0 to 35	90
Road base	-	96

---

[6] American Association of State Highway and Transportation Officials

Subsequent to stripping and prior to the placement of structural site grading fill, the subgrade must be prepared as discussed in Section 5.3.1, Site Preparation, of this report. In confined areas, subgrade preparation should consist of the removal of all loose or disturbed soils.

Non-structural fill may be placed in lifts not exceeding 12 inches in loose thickness and compacted by passing construction, spreading, or hauling equipment over the surface at least twice.

5.3.5 Utility Trenches

All utility trench backfill material below structurally loaded facilities (flatwork, floor slabs, roads, etc.) should be placed at the same density requirements established for structural fill. If the surface of the backfill becomes disturbed during the course of construction, the backfill should be proofrolled and/or properly compacted prior to the construction of any exterior flatwork over a backfilled trench. Proofrolling may be performed by passing moderately loaded rubber tire-mounted construction equipment uniformly over the surface at least twice. If excessively loose or soft areas are encountered during proofrolling, they should be removed to a maximum depth of two feet below design finish grade and replaced with structural fill.

Most utility companies and City-County governments are now requiring that Type A-1 or A-1-a (AASHTO Designation – basically granular soils with limited fines) soils 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.

Fine-grained cohesive soils are not recommended for use as trench backfill. The natural sand and gravel may be suitable for use as trench backfill provided it meets the requirements of A-1 or A-1-a material.

5.3.6 Areal Settlements

Areal settlements associated with up to 10 feet of structural site grading fill will be minimal. These settlements are in addition to settlements induced by foundation and floor slab loads. The majority of this settlement will occur during placement.

5.4 FOUNDATIONS

5.4.1 Spread and Continuous Wall Foundations

5.4.1.1 Design Data

The proposed apartment, hotel, mixed-use, and retail structures may be supported upon conventional spread and continuous wall foundations established upon suitable natural soils and/or structural fill extending to suitable natural soils. Under no circumstances shall footings be placed overlying non-engineered fills.

Minimum Recommended Depth of Embedment for Frost Protection	- 30 inches
Minimum Recommended Depth of Embedment for Non-frost Conditions	- 15 inches
Recommended Minimum Width for Continuous Wall Footings	- 18 inches
Minimum Recommended Width for Isolated Spread Footings	- 24 inches
Recommended Net Bearing Pressure for Real Load Conditions	
Footings having minimum depth of embedment and width	- 3,000 pounds per square foot*
Footings having a minimum depth of embedment and a minimum plan dimension of four feet or greater	- 6,000 pounds per square foot*
Bearing Pressure Increase for Seismic Loading – Vertical Downward	- 50 percent

* For intermediate-sized footings, the appropriate bearing pressure may be interpolated on a straightline basis from these values.

** Do not apply to edge bearing loading conditions.

The term "net bearing pressure" refers to the pressure imposed by the portion of the structure located above lowest adjacent final grade. Therefore, the weight of the footing and backfill to lowest adjacent final grade need not be considered. Real loads are defined as the total of all dead plus frequently applied live loads. Total load includes all dead and live loads, including seismic and wind.

5.4.1.2 Installation

Under no circumstances shall the footings be established upon non-engineered fills, loose or disturbed soils, rubbish, construction debris, other deleterious materials, frozen soils, or within ponded water. If unsuitable soils are encountered, they must be completely removed and replaced with compacted structural fill.

The width of structural replacement fill below footings should be equal to the width of the footing plus one foot for each foot of fill thickness.

5.4.1.3 Settlements

Settlements of conventional shallow foundations designed and installed in accordance with the above recommendations and supported upon a sequence of natural granular soils and/or granular structural fill are projected to be on the order of one inch or less.

Settlements will occur rapidly with approximately 50 to 70 percent occurring during construction.

5.4.2 Reinforced Continuous Mat

As stated previously, the condominium structure will likely need to be established on a continuous mat due to overlap of the large footings necessary. The net pressure imposed by the base of a mat established a minimum of 25 feet deep will be negligible; therefore, the settlements would be mostly elastic and occur almost instantaneously with application of the load. The projected settlement varies depending on the depth and thickness of the mat. A mat established at a greater depth would impose less net load and therefore experience less settlement.

A reinforced mat with a negligible net load is projected to experience elastic settlements on the order of one to one and one-half inches with the greatest settlement at the center of the mat. At the edges and corners, mat settlements would be approximately 50 to 60 percent of the center settlements. The mat must be underlain by a minimum of 18 inches of granular structural fill extending to suitable natural soils.

For a mat established on a minimum of 18 inches of granular structural fill and with a minimum embedment depth of 25 feet, we recommend that a modulus of reaction of 25 pounds per cubic inch be used for preliminary design. We request that a bearing pressure distribution plan be provided to our office for review, when available.

5.5 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 on granular soils, a coefficient of 0.45 should be utilized. 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. Below the water table, this granular soil should be considered equivalent to a fluid with a density of 150 pounds per cubic foot.

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

5.6 LATERAL PRESSURES

The lateral pressure parameters as presented within this section, assume that the backfill will consist of a drained granular soil placed and compacted in accordance with the recommendations presented herein. Subdrains around below-grade levels will be an essential part of construction.

The lateral pressures imposed upon subgrade facilities will, therefore, be basically dependent upon the relative rigidity and movement of the backfilled structure. For active walls, such as retaining walls which can move outward (away from the backfill), granular backfill may be considered equivalent to a fluid with a density of 35 pounds per cubic foot in computing lateral pressures. For more rigid basement walls that are not more than 10 inches thick and 12 feet or less in height, granular backfill may be considered equivalent to a fluid with a density of 45 pounds per cubic foot. For very rigid non-yielding walls, granular backfill should be considered equivalent to a fluid with a density of at least 60 pounds per cubic foot. The above values assume that the surface of the soils slope behind the wall is horizontal, that the granular fill has been placed and lightly compacted, not as a structural fill. If the fill is placed as a structural fill, the values should be increased to 45 pounds per cubic foot, 60 pounds per cubic foot, and 120 pounds per cubic foot, respectively. If the slope behind the wall is two horizontal to one vertical the values for purely active walls and basement walls should increase to 57 pounds per cubic foot and 67 pounds per cubic foot, respectively.

In addition to the static pressures, seismic loadings must be considered. Recommended average lateral uniform pressure for various height walls are tabulated on the following page and assume a granular wall backfill with a horizontal grade above the wall:

Wall Height (feet)	Uniform Seismic Lateral Pressure*, ** (psf)
5	106
10	213
15	319
20	426
40	851

* Maximum short-term pressures, they are not sustained loads.

** For intermediate height wall, the lateral pressure will be developed based upon a straightline interpolated between the pressures at the specific height.

Note that the pressures presented in the section do not include surcharge loadings, such as floor slabs, adjacent footings, etc.

5.7 FLOOR SLABS

Floor slabs may be established upon suitable undisturbed natural soils, and/or upon structural fill extending to suitable natural soils or properly prepared existing surface soils. Non-engineered fills and topsoil are not considered suitable. To provide a capillary break, it is recommended that floor slabs be directly underlain by at least four inches of “free-draining” fill, such as “pea” gravel or three-quarters- to one-inch minus clean gap-graded gravel. Settlements of lightly to moderately loaded floor slabs are anticipated to be minor.

5.8 SUBDRAINS

Due to the potential for infiltration from the adjacent slope, and to provide additional protection, we recommend that a foundation subdrain be installed along the up-gradient and side-gradient subgrade walls.

Foundation subdrains should consist of a four-inch diameter perforated or slotted plastic or PVC pipe enclosed in clean gravel. The invert of a subdrain should be at least two feet below the top of the lowest adjacent floor slab. The gravel portion of the drain should extend two inches laterally and below the perforated pipe and at least one foot above the top of the lowest adjacent floor slab. The gravel zone must be installed immediately adjacent to the perimeter footings and the foundation walls. To reduce the possibility of plugging, the gravel must be wrapped with a geotextile, such as Mirafi 140N or equivalent.

Above the subdrain, a minimum four-inch-wide zone of "free-draining" sand and gravel should be placed adjacent to the foundation walls and extend to within two feet of final grade. The upper two feet of soils should consist of a compacted clayey cap to reduce surface water infiltration into the drain. As an alternative to the zone of permeable sand and a prefabricated "drainage board," such as Miradrain or equivalent, may be placed adjacent to the exterior below grade walls. Prior to the installation of the footing subdrain, the below-grade walls should be dampproofed. The slope of the subdrain should be at least 0.3 percent. The gravel placed around the drainpipe should be clean three-quarters- to one-inch minus gap-graded gravel and/or "pea" gravel. The foundation subdrains can be discharged into the area subdrains, storm drains, or other suitable down-gradient location.

5.9 PAVEMENTS

The properly prepared non-engineered fills will exhibit poor engineering characteristics when saturated or nearly saturated. Non-engineered fills may remain in flexible pavement areas if properly prepared, as stated previously in this report. Rigid pavements shall not be placed overlying non-engineered fills, even if properly prepared. A pavement section recommendation for the re-alignment of Wasatch Boulevard is not provided in this report. We recommend that the section for Wasatch Boulevard match the existing Wasatch Boulevard section where it joins with 6200 South Street along the west side of the project. A pavement section recommendation for Wasatch Boulevard can be provided if detailed traffic loading is available.

Considering the existing non-engineered soils as the subgrade soils and the projected traffic, the following pavement sections are recommended:

Roadway Areas

(Moderate Volume of Automobiles and Light Trucks
with Light Volume of Medium- and Heavy-Weight Trucks)
[5 to 10 equivalent 18-kip axle loads per day]

Flexible Pavements:
(Asphalt Concrete)

4.0 inches	Asphalt concrete
10.0 inches	Aggregate base course
Over	Minimum of 12 inches of suitable granular soil (natural and/or fill). This layer can also be considered as a subbase component.

Rigid Pavements:
(Non-reinforced Concrete)

6.0 inches	Portland cement concrete (non-reinforced)
5.0 inches	Aggregate base course
Over	Minimum of 12 inches of suitable granular soil (natural and/or fill). This layer can also be considered as a subbase component.

Parking Areas

(Moderate Volume of Automobiles and Light Trucks
with Light Volume of Medium- and Heavy-Weight Trucks)
[5 to 7 equivalent 18-kip axle loads per day]

Flexible Pavements:
(Asphalt Concrete)

3.0 inches	Asphalt concrete
8.0 inches	Aggregate base course
Over	Minimum of 12 inches of suitable granular soil (natural and/or fill). This layer can also be considered as a subbase component.

Rigid Pavements:
(Non-reinforced Concrete)

5.0 inches	Portland cement concrete (non-reinforced)
6.0 inches	Aggregate base course
Over	Minimum of 12 inches of suitable granular soil (natural and/or fill). This layer can also be considered as a subbase component.

Rigid pavements over non-engineered fill will be subjected to settlement which will result in cracking of the concrete surface. Asphalt concrete pavements would also settle but are much more tolerant to movement. In critical areas, it is our recommendation that rigid pavements over non-engineered fills be reinforced with No. 4 rebar on 18-inch centers.

For dumpster pads, we recommend a pavement section consisting of six and one-half inches of Portland cement concrete, four inches of aggregate base course, over properly prepared natural subgrade or site grading structural fills.

Granular structural site grading fill, if sufficiently “clean,” will satisfy the requirements for granular subbase.

Asphalt concrete and base course components should meet the requirements and be placed in accordance with the Utah Department of Transportation specifications.

The above rigid pavement sections are for reinforced and non-reinforced Portland cement concrete. Construction of the rigid pavement should be in sections 10 to 12 feet in width with construction or expansion joints or one-quarter depth saw-cuts on no more than 12-foot centers. Saw-cuts must be completed within 24 hours of the “initial set” of the concrete and should be performed under the direction of the concrete paving contractor. The concrete should have a minimum 28-day unconfined compressive strength of 4,000 pounds per square inch and contain 6 percent ±1 percent air-entrainment.

5.10 GEOSEISMIC SETTING

5.10.1 General

As of July 2019, the State of Utah has adopted the International Building Code (IBC) 2018 and International Residential Code (IRC) 2015. The IBC 2018 code determines the seismic hazard for a site based upon 2008 mapping of bedrock accelerations prepared by the United States Geologic Survey (USGS) and the soil site class. The USGS values are presented on maps incorporated into the IBC code and are also available based on latitude and longitude coordinates (grid points).

The structures must be designed in accordance with the procedure presented in Section 1613, Earthquake Loads, of the IBC 2018 edition.

5.10.2 Faulting

The results of the fault study indicate that active normal faulting was observed in the continuous trench. Study details and fault setbacks are presented in The Surface Fault Rupture Hazard Evaluation is enclosed with this report; see Appendix A.

5.10.3 Soil Class

As stated earlier, a Vs_{100} value of 1549.0 ft/sec was calculated from the ReMi survey performed in the lower portion of the site. Based on the shear wave velocity profile and on the soils encountered in our exploration borings, we recommend that "Site Class C – Very Dense Soil and Soft Rock" be utilized for the design of structures at the site. The average shear-wave velocity profile can be seen in Appendix B.

5.10.4 Ground Motions

The IBC 2018 code is based on 2014 USGS mapping, which provides values of short and long period accelerations for the Site Class B boundary for the Maximum Considered Earthquake (MCE). This Site Class B boundary represents a hypothetical sandstone bedrock surface and must be corrected for local soil conditions. The following table summarizes the peak ground and short and long period accelerations for a MCE event and incorporates a soil amplification factor for a Site Class C soil profile in the second column. Based on the site latitude and longitude (40.6299 degrees north and -111.7979 degrees west, respectively), the values for this site are tabulated below:

Spectral Acceleration Value, T Seconds	**Site Class B-C Boundary [mapped values] (% g)**	**Site Class C [adjusted for site class effects] (% g)**
Peak Ground Acceleration (Geo-Mean)	60.9	73.1
0.2 Seconds (Short Period Acceleration)	$S_S = 134.2$	$S_{MS} = 161.0$
1.0 Seconds (Long Period Acceleration)	$S_1 = 49.8$	$S_{M1} = 74.7$

The IBC 2018 code design accelerations (S_{DS} and S_{D1}) are based on multiplying the above accelerations (adjusted for site class effects) for the MCE event by two-thirds.

5.10.5 Liquefaction

As shown on the Cottonwood Heights City Ordinance Chapter 19.72 (SLEDS) liquefaction potential map, the site is mapped within an area having "very low" liquefaction potential during the design seismic event.

The site is located on a boundary that has been identified by the Utah Geological Survey as having "moderate" liquefaction potential. Liquefaction is defined as the condition when saturated, loose, finer-grained sand-type soils lose their support capabilities because of excessive pore water pressure which develops during a seismic event.

Due to the medium dense nature of the saturated granular soils encountered, our analysis indicates liquefaction is not anticipated during the design seismic event.

Calculations were performed using the procedures described in the 2008 Soil Liquefaction During Earthquakes Monograph by Idriss and Boulanger[7].

5.11 SITE OBSERVATIONS

As stated previously, due to the variable nature of the non-engineered fills encountered, a qualified geotechnical engineer must aid in verifying that all non-engineered fills have been completely removed prior to the placement of structural site grading fills, footings, or foundations.

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[7] Idriss, I. M., and Boulanger, R. W. (2008), Soil liquefaction during earthquakes: Monograph MNO-12, Earthquake Engineering Research Institute, Oakland, CA, 261 pp.

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,

Gordon Geotechnical Engineering, Inc.

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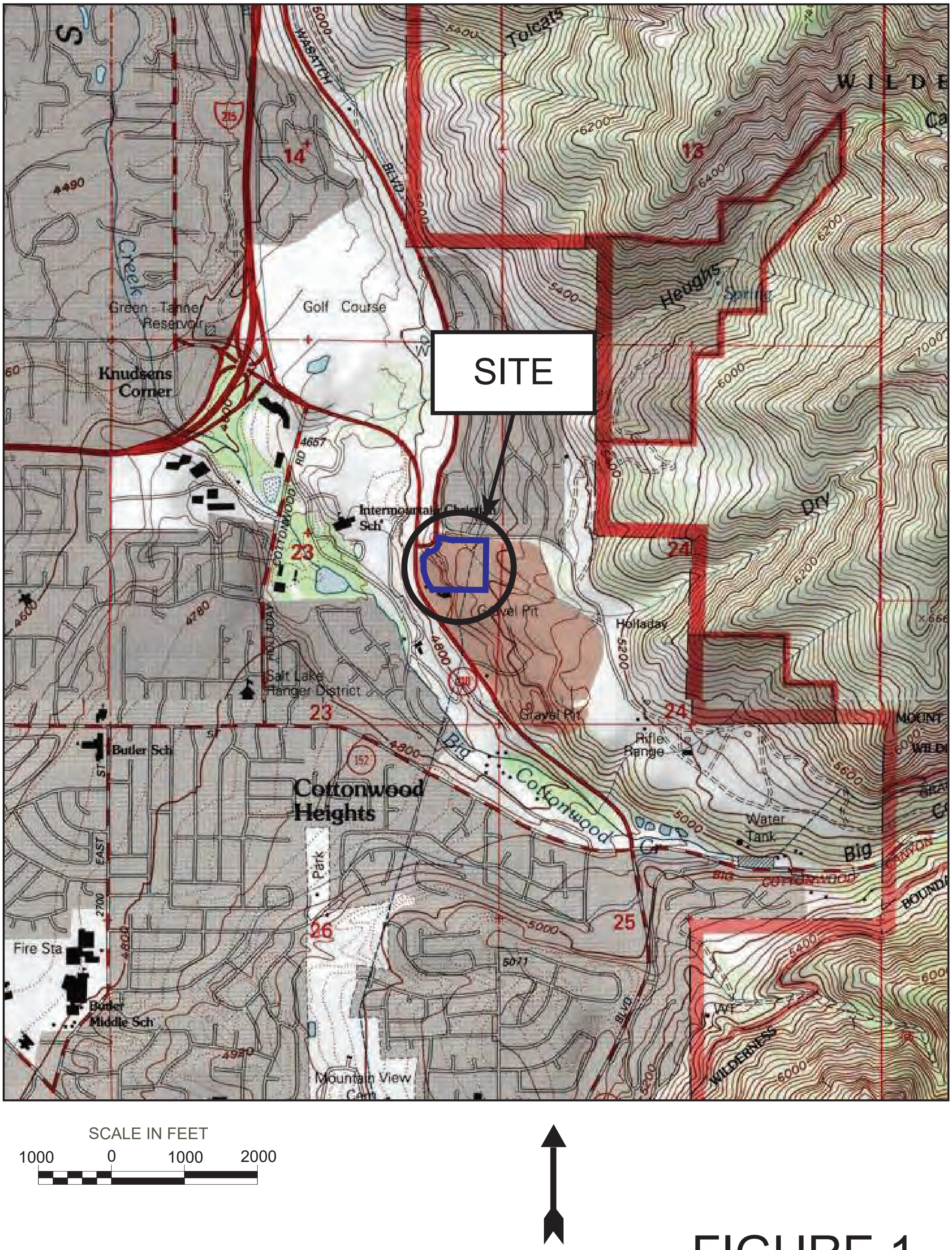
Encl. Figure 1, Vicinity Map
Figure 2, Area Map
Figure 3, Site Plan
Figures 4A through 4C, Log of Borings
Figures 4D through 4K, Log of Test Pits
Figure 5, Unified Soil Classification System
Figure 6, Photographs
Appendix A, Geologic Hazards Study Report
Appendix B, ReMi Survey Results
Appendix C, Direct Shear Test Results
Appendix D, Slope Stability Analysis Results

Addressee (3 + email)

ROCKWORTH COMPANIES
JOB NO. 528-005-20

GORDON GEOTECHNICAL ENGINEERING, INC.

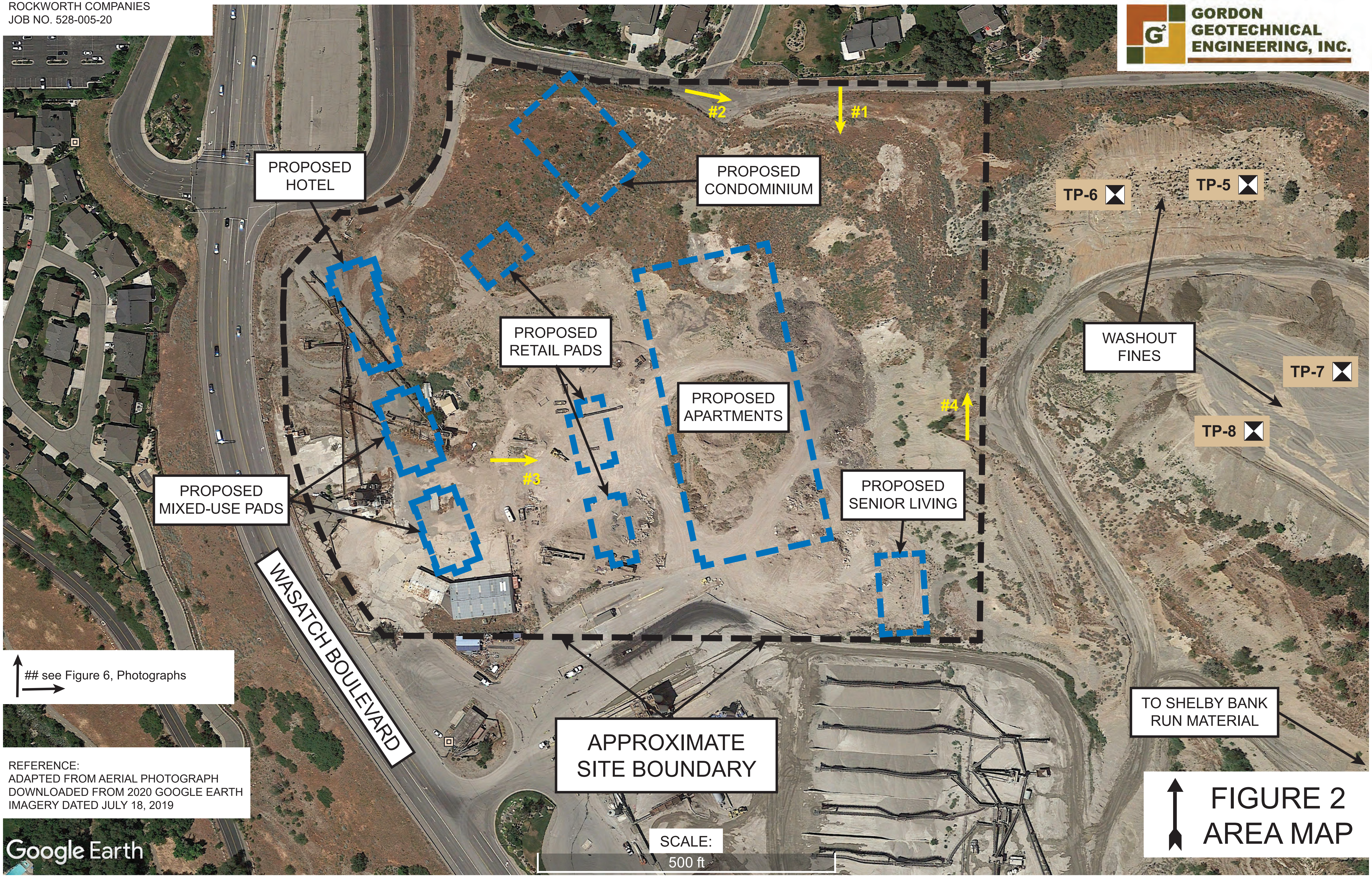


REFERENCE:
USGS 7.5 MINUTE TOPOGRAPHIC QUADRANGLE MAPS TITLED "SUGAR HOUSE, UTAH", AND "DRAPER, UTAH", BOTH DATED 1998

FIGURE 1
VICINITY MAP

ROCKWORTH COMPANIES
JOB NO. 528-005-20

GORDON GEOTECHNICAL ENGINEERING, INC.

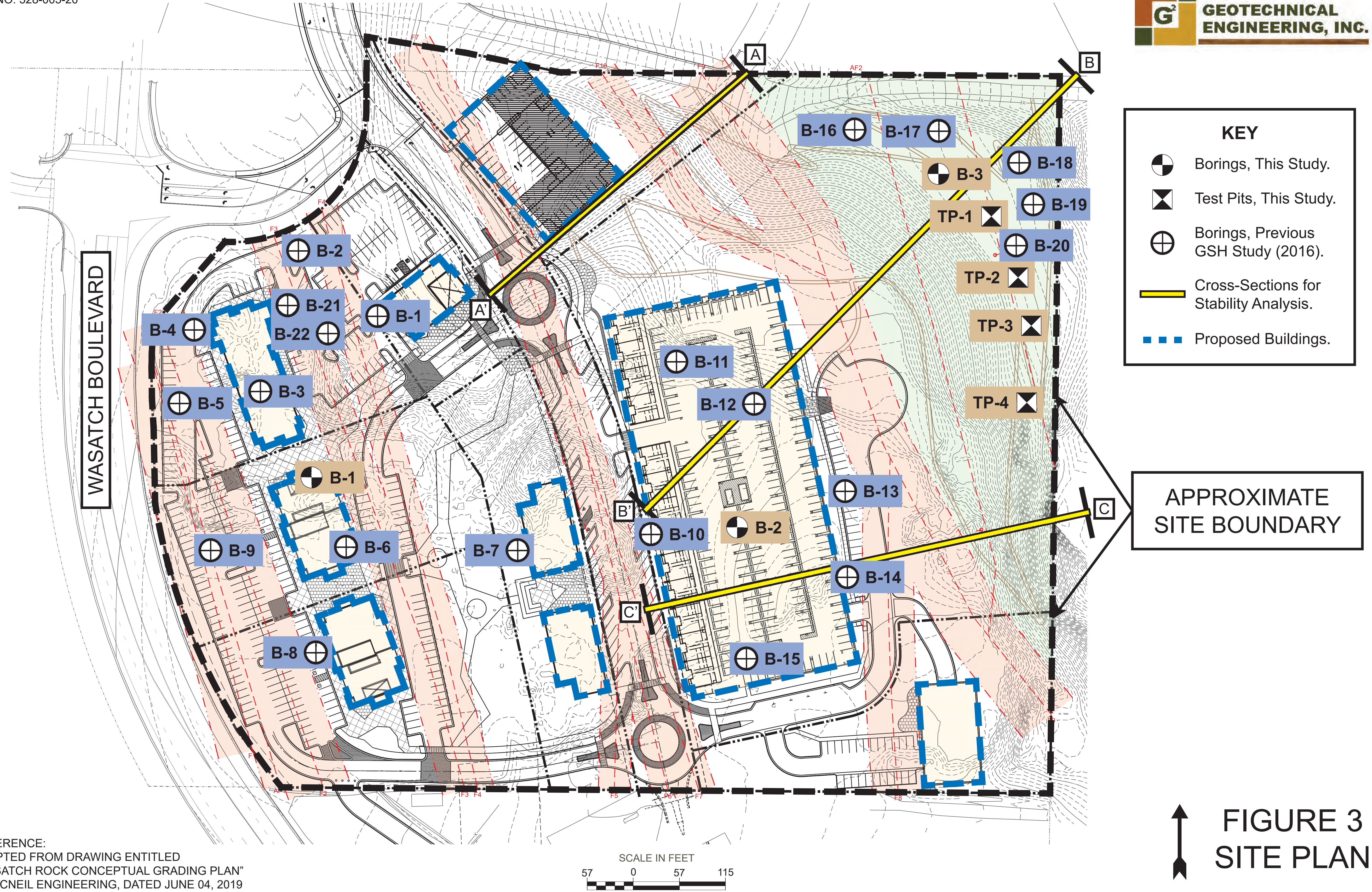


REFERENCE:
ADAPTED FROM AERIAL PHOTOGRAPH
DOWNLOADED FROM 2020 GOOGLE EARTH
IMAGERY DATED JULY 18, 2019

FIGURE 2
AREA MAP

ROCKWORTH COMPANIES
JOB NO. 528-005-20

GORDON
GEOTECHNICAL
ENGINEERING, INC.



REFERENCE:
ADAPTED FROM DRAWING ENTITLED
"WASATCH ROCK CONCEPTUAL GRADING PLAN"
BY MCNEIL ENGINEERING, DATED JUNE 04, 2019

FIGURE 3
SITE PLAN

Gordon Geotechnical Engineering, Inc.
4426 South Century Drive, Suite 100
Salt Lake City, Utah 84123

BOREHOLE B-1

Project Name: Proposed Gravel Pit Development

Location: 6695 Wasatch Boulevard, Cottonwood Heights, Utah

Drilling Method: 4" Case ODEX

Elevation: ---

Remarks:

Project No.: 528-005-20

Client: Rockworth Companies

Date Drilled: 03-02-20

Water Level: 32.5' (03-02-20), 36.0' (03-25-20)

DESCRIPTION	GRAPHIC LOG	WATER LEVEL	DEPTH (FT.)	SAMPLE SYMBOL	SAMPLE TYPE	BLOWS/FT.	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	REMARKS
12.0" AGGREGATE BASE												
FINE AND COARSE GRAVEL with trace silt and fine sand; brown (GP)												slightly moist very dense
				▮	SPT	50 5"						
FINE TO COARSE SAND with some silt and occasional silty fine sand layers 4" thick; tan (SP-SM)			5	▮	SPT	66						slightly moist very dense
				▮	SPT	31	9.5		35.2			
												dense
grades with trace fine and coarse gravel			10	▮	SPT	26	8.3		3.0			moist medium dense
grades grayish-tan			15	⧗	D	90	16.5	112				dense
SILTY CLAY with trace fine sand and occasional fine sand seams; tan (CL)												moist very stiff
			20	⧗	D	31	24.3	98				
												saturated
SILTY FINE AND COARSE GRAVEL with some fine to coarse sand and frequent cobbles; brown (GM)			25	⧗		32						saturated

The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary for a proper understanding of the nature of the subsurface material.

BOREHOLE B-1

Project Name: Proposed Gravel Pit Development
Project No.: 528-005-20
Location: 6695 Wasatch Boulevard, Cottonwood Heights, Utah
Client: Rockworth Companies
Drilling Method: 4" Case ODEX
Date Drilled: 03-02-20
Elevation: ---
Water Level: 32.5' (03-02-20), 36.0' (03-25-20)
Remarks:

DESCRIPTION	GRAPHIC LOG	WATER LEVEL	DEPTH (FT.)	SAMPLE SYMBOL	SAMPLE TYPE	BLOWS/FT.	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	REMARKS
				⧗	D	32						medium dense
FINE TO COARSE SAND with some silt and gravel; tan (SP-SM)												slightly moist medium dense
			30	‖	SPT	22	2.8		10.7			
												saturated
SILTY FINE SAND with frequent fine sandy clay layers 6" thick ; tan (SM)												saturated medium dense
		▼	35	‖	SPT	22						
grades with occasional silty clay layers 1/2" thick			40	⧗	D	40	23.2	99	18.5			
Stopped drilling at 39.5'. Stopped sampling at 41.0'. Installed slotted PVC pipe to 40.0'.			45									
			50									

The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary for a proper understanding of the nature of the subsurface material.

FIGURE 4A
(con't)

BOREHOLE B-2

Project Name: Proposed Gravel Pit Development
Location: 6695 Wasatch Boulevard, Cottonwood Heights, Utah
Drilling Method: 4" Case ODEX
Elevation: ---
Remarks:

Project No.: 528-005-20
Client: Rockworth Companies
Date Drilled: 03-02-20
Water Level: 29.0' (03-02-20), 30.8' (03-25-20)

DESCRIPTION	GRAPHIC LOG	WATER LEVEL	DEPTH (FT.)	SAMPLE SYMBOL	SAMPLE TYPE	BLOWS/FT.	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	REMARKS
FINE AND COARSE GRAVEL with some silt and sand and occasional small boulders; tan (GP-GM)	▨											slightly moist very dense
	▨			▮	SPT	50 5"	0.9		9.8			
	▨		5	▮	SPT	74						
	▨		10	▮	SPT	36	1.0		4.6			dense
grades with occasional cobbles and boulders	▨		15	▮	SPT	69	2.8		10.2			very dense
	▨		20	▮	SPT	14						
	▨		25	▮								

The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary for a proper understanding of the nature of the subsurface material.

BOREHOLE B-2

Project Name: Proposed Gravel Pit Development
Project No.: 528-005-20
Location: 6695 Wasatch Boulevard, Cottonwood Heights, Utah
Client: Rockworth Companies
Drilling Method: 4" Case ODEX
Date Drilled: 03-02-20
Elevation: ---
Water Level: 29.0' (03-02-20), 30.8' (03-25-20)
Remarks:

DESCRIPTION	GRAPHIC LOG	WATER LEVEL	DEPTH (FT.)	SAMPLE SYMBOL	SAMPLE TYPE	BLOWS/FT.	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	REMARKS
FINE AND COARSE GRAVEL with some silt and sand and occasional small boulders; tan (GP-GM)					SPT	39	1.2		5.9			dense
												saturated
			30		SPT	8						
		▼										
SILTY CLAY with trace fine sand; gray (CL)												saturated very stiff
			35		D	30	29.8	94				
SILTY FINE SAND gray (SM)												saturated dense
			40		D	93	21.6	104				
FINE SANDY CLAY gray (CL)												saturated very stiff
			45		SPT	27	25.0		66.3	35	21	
			50									

The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary for a proper understanding of the nature of the subsurface material.

FIGURE 4B
(con't)

Project Name: Proposed Gravel Pit Development
Project No.: 528-005-20
Location: 6695 Wasatch Boulevard, Cottonwood Heights, Utah
Client: Rockworth Companies
Drilling Method: 4" Case ODEX
Date Drilled: 03-02-20
Elevation: ---
Water Level: 29.0' (03-02-20), 30.8' (03-25-20)
Remarks:

DESCRIPTION	GRAPHIC LOG	WATER LEVEL	DEPTH (FT.)	SAMPLE SYMBOL	SAMPLE TYPE	BLOWS/FT.	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	REMARKS
	▨			‖	SPT	18						
Stopped drilling at 50.0'. Stopped sampling at 51.5'. Installed slotted PVC pipe to 50.0'.			55									
			60									
			65									
			70									
			75									

The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary for a proper understanding of the nature of the subsurface material.

FIGURE 4B
(con't)

BOREHOLE B-3

Project Name: Proposed Gravel Pit Development
Project No.: 528-005-20
Location: 6695 Wasatch Boulevard, Cottonwood Heights, Utah
Client: Rockworth Companies
Drilling Method: 4" Case ODEX
Date Drilled: 03-13-20
Elevation: ---
Water Level: No groundwater encountered.
Remarks:

DESCRIPTION	GRAPHIC LOG	WATER LEVEL	DEPTH (FT.)	SAMPLE SYMBOL	SAMPLE TYPE	BLOWS/FT.	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	REMARKS
SILTY CLAY, FILL with some fine to coarse sand and fine gravel; reddish-brown (CL-FILL)	▨											moist soft
CONCRETE WASHOUT FILL white (CONCRETE)	▒		5 10									dry very dense
grades with occasional layers of sand and gravel 6" to 24"	▒		15	⧗	D							sampler bouncing
sampled because soil was encountered at 14.0', no recovery, sampler bouncing on concrete	▒											
layers of soil on the order of 6" to 2' between layers of concrete	▒		20	‖	SPT	50 4"						sampler bouncing
	▒		25	‖		50 2"						

The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary for a proper understanding of the nature of the subsurface material.

Project Name: Proposed Gravel Pit Development
Location: 6695 Wasatch Boulevard, Cottonwood Heights, Utah
Drilling Method: 4" Case ODEX
Elevation: ---
Remarks:

Project No.: 528-005-20
Client: Rockworth Companies
Date Drilled: 03-13-20
Water Level: No groundwater encountered.

DESCRIPTION	DEPTH (FT.)	SAMPLE TYPE	BLOWS/FT.	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	REMARKS
		SPT	2"						
	30								
FINE AND COARSE GRAVEL, FILL with some sand; light brown (GP-FILL)	35								moist very dense
		D	100 3"	5.0		3.9			
	40	SPT	50 3"						
CONCRETE WASHOUT FILL white (CONCRETE)									dry very dense
	45								
	50								

The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary for a proper understanding of the nature of the subsurface material.

Project Name: Proposed Gravel Pit Development
Project No.: 528-005-20
Location: 6695 Wasatch Boulevard, Cottonwood Heights, Utah
Client: Rockworth Companies
Drilling Method: 4" Case ODEX
Date Drilled: 03-13-20
Elevation: ---
Water Level: No groundwater encountered.
Remarks:

DESCRIPTION	GRAPHIC LOG	WATER LEVEL	DEPTH (FT.)	SAMPLE SYMBOL	SAMPLE TYPE	BLOWS/FT.	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	REMARKS
FINE TO COARSE SAND AND FINE AND COARSE GRAVEL with trace silt; grayish-brown (SP/GP)			55	⧗	D	103	1.8	123	4.6			sligthly moist very dense
			60	▮	SPT	64	1.1		6.5			
			65	⧗	D	119	1.8	91				
			70	▮	SPT	62						
drilling indicates cobbles and boulders												
SILTY FINE SAND thinly layered; light brown (SM)			75	⧗		51						slightly moist medium dense

The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary for a proper understanding of the nature of the subsurface material.

Project Name: Proposed Gravel Pit Development

Project No.: 528-005-20

Location: 6695 Wasatch Boulevard, Cottonwood Heights, Utah

Client: Rockworth Companies

Drilling Method: 4" Case ODEX

Date Drilled: 03-13-20

Elevation: ---

Water Level: No groundwater encountered.

Remarks:

DESCRIPTION	GRAPHIC LOG	WATER LEVEL	DEPTH (FT.)	SAMPLE SYMBOL	SAMPLE TYPE	BLOWS/FT.	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	REMARKS
	▥			⧗	D	51	11.1	101				
SILTY FINE TO MEDIUM SAND with some fine gravel; light brown (SM)	▥		80	▮	SPT	65	3.6		18.2			slightly moist medium dense
FINE AND COARSE GRAVEL with some fine to coarse sand and silt; likely with cobbles and boulders; light yellowish-brown (GP-GM)	▨		85	▮	SPT	50 2"	2.2		10.4			slightly moist very dense
	▨		90	▮	SPT	50 5"						
	▨		95	▮	SPT	50 3"	0.2		0.9			
	▨		100	▮		50 5"						

The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary for a proper understanding of the nature of the subsurface material.

BOREHOLE B-3

Project Name: Proposed Gravel Pit Development
Location: 6695 Wasatch Boulevard, Cottonwood Heights, Utah
Drilling Method: 4" Case ODEX
Elevation: ---
Remarks:

Project No.: 528-005-20
Client: Rockworth Companies
Date Drilled: 03-13-20
Water Level: No groundwater encountered.

DESCRIPTION	GRAPHIC LOG	WATER LEVEL	DEPTH (FT.)	SAMPLE SYMBOL	SAMPLE TYPE	BLOWS/FT.	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	REMARKS
	▨			▮	SPT	5"						
Stopped drilling at 99.5'. Stopped sampling at 101.0'. Installed slotted PVC pipe to 101.0'. No groundwater encountered at time of drilling.			105 110 115 120 125									

The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary for a proper understanding of the nature of the subsurface material.

FIGURE 4C
(con't)

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TEST PIT TP-1

Page: 1 of 1

Project Name: Proposed View 62 Development
Location: 6200 S Wasatch Boulevard, Cottonwood Heights, Utah
Excavating Method: Hitachi Trackhoe
Elevation: ---
Remarks:

Project No.: 528-002-18
Client: Rockworth Companies
Date Excavated: 02-01-18
Water Level: No groundwater encountered.

DESCRIPTION	GRAPHIC LOG	WATER LEVEL	DEPTH (FT.)	SAMPLE SYMBOL	SAMPLE TYPE	BLOWS/FT.	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	REMARKS
SILTY AND CLAYEY FINE TO COARSE SAND, FILL with fine and coarse gravel; reddish-brown (SC-SM-FILL)	▨											moist "medium dense"
			5									
				◣	B							
Excavation refusal at 7.0' on concrete washout material. Stopped sampling at 7.0'. No groundwater encountered at time of excavation. No signficant sidewall caving. Trenched northwest approximately 40.0' at refusal.			10									
			15									
			20									
			25									

The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary for a proper understanding of the nature of the subsurface material.

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Salt Lake City, Utah 84123

TEST PIT TP-2

Page: 1 of 1

Project Name: Proposed View 62 Development
Project No.: 528-002-18
Location: 6200 S Wasatch Boulevard, Cottonwood Heights, Utah
Client: Rockworth Companies
Excavating Method: Hitachi Trackhoe
Date Excavated: 02-01-18
Elevation: ---
Water Level: No groundwater encountered.
Remarks:

DESCRIPTION	GRAPHIC LOG	WATER LEVEL	DEPTH (FT.)	SAMPLE SYMBOL	SAMPLE TYPE	BLOWS/FT.	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	REMARKS
CLAYEY FINE TO COARSE SAND AND FINE AND COARSE GRAVEL, FILL reddish-brown (SC/GC-FILL)	▨		5 10									moist "loose"
Excavation refusal at 10.0' on concrete washout material. No groundwater encountered at time of excavation. No signficant sidewall caving. Trenched southeast approximately 20.0' at refusal.			15 20 25									

The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary for a proper understanding of the nature of the subsurface material.

FIGURE 4E

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TEST PIT TP-3

Page: 1 of 1

Project Name: Proposed View 62 Development
Project No.: 528-002-18
Location: 6200 S Wasatch Boulevard, Cottonwood Heights, Utah
Client: Rockworth Companies
Excavating Method: Hitachi Trackhoe
Date Excavated: 02-01-18
Elevation: ---
Water Level: No groundwater encountered.
Remarks:

DESCRIPTION	GRAPHIC LOG	WATER LEVEL	DEPTH (FT.)	SAMPLE SYMBOL	SAMPLE TYPE	BLOWS/FT.	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	REMARKS
SILTY AND CLAYEY FINE TO COARSE SAND, FILL with fine and coarse gravel; reddish-brown (SC-SM-FILL)	▨		5									
SANDY CLAY, FILL red (CL-FILL) [RED CLAY]	▨		10									moist "stiff"
				◢	B				67.8			
grades dark brown			15 20									
				◢	B							
Excavation refusal at 23.0' due to maximum reach. Stopped sampling at 22.5'. No groundwater encountered at time of excavation. No signficant sidewall caving.			25									

The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary for a proper understanding of the nature of the subsurface material.

FIGURE 4F

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TEST PIT TP-4

Page: 1 of 1

Project Name: Proposed View 62 Development
Project No.: 528-002-18
Location: 6200 S Wasatch Boulevard, Cottonwood Heights, Utah
Client: Rockworth Companies
Excavating Method: Hitachi Trackhoe
Date Excavated: 02-01-18
Elevation: ---
Water Level: No groundwater encountered.
Remarks:

DESCRIPTION	GRAPHIC LOG	WATER LEVEL	DEPTH (FT.)	SAMPLE SYMBOL	SAMPLE TYPE	BLOWS/FT.	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	REMARKS
CLAYEY FINE TO COARSE SAND AND FINE AND COARSE GRAVEL, FILL with occasional slabs of concrete and asphalt concrete, cobbles, and small boulders; reddish-brown (SC/GC-FILL)	▨		5 10									
FINE TO COARSE SAND AND GRAVEL light brown (SP-GP) [NATIVE]	▨		15	◣	B							slightly moist "dense"
Stopped excavation at 16.0'. Stopped sampling at 15.5'. No groundwater encountered at time of excavation. No signficant sidewall caving.			20 25									

The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary for a proper understanding of the nature of the subsurface material.

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TEST PIT TP-5

Page: 1 of 1

Project Name: Proposed View 62 Development
Location: 6200 S Wasatch Boulevard, Cottonwood Heights, Utah
Excavating Method: Hitachi Trackhoe
Elevation: ---
Remarks:

Project No.: 528-002-18
Client: Rockworth Companies
Date Excavated: 02-01-18
Water Level: No groundwater encountered.

DESCRIPTION	GRAPHIC LOG	WATER LEVEL	DEPTH (FT.)	SAMPLE SYMBOL	SAMPLE TYPE	BLOWS/FT.	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	REMARKS
FINE SANDY CLAY/CLAYEY FINE SAND, FILL brown (CL/SC-FILL) [WASHOUT FINES]	▨ ░			◣	B		15.0		59.3			
MEDIUM TO COARSE SAND, FILL brown (SP-FILL)	░											
SANDY CLAY, FILL brown (CL-FILL) [WASHOUT FINES]	▨		5	◣	B		20.4		70.6			slightly moist "medium stiff"
MEDIUM TO COARSE SAND, FILL brown (SP-FILL)	░			◣	B		1.9		4.6			
SANDY CLAY, FILL brown (CL-FILL) [WASHOUT FINES]	▨		10									
MEDIUM TO COARSE SAND, FILL brown (SP-FILL)	░											
SANDY CLAY, FILL brown (CL-FILL) [WASHOUT FINES]	▨		15									
Stopped excavation at 16.0'. Stopped sampling at 15.5'. No groundwater encountered at time of excavation. No signficant sidewall caving.			20 25									

The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary for a proper understanding of the nature of the subsurface material.

FIGURE 4H

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TEST PIT TP-6

Project Name: Proposed View 62 Development
Project No.: 528-002-18
Location: 6200 S Wasatch Boulevard, Cottonwood Heights, Utah
Client: Rockworth Companies
Excavating Method: Hitachi Trackhoe
Date Excavated: 02-01-18
Elevation: ---
Water Level: No groundwater encountered.
Remarks:

DESCRIPTION	GRAPHIC LOG	WATER LEVEL	DEPTH (FT.)	SAMPLE SYMBOL	SAMPLE TYPE	BLOWS/FT.	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	REMARKS
SILTY CLAY, FILL with medium to coarse sand; brown (CL-FILL) [WASHOUT FINES]	▨			◢	B		17.5		92.7			slightly moist "medium stiff"
	▨		5									
	▨			◢	B		21.8		69.3			
MEDIUM TO COARSE SAND, FILL brown (SP-FILL)	░											
SANDY CLAY, FILL brown (CL-FILL) [WASHOUT FINES]	▨		10									
MEDIUM TO COARSE SAND, FILL brown (SP-FILL)	░			◢	B		2.8		4.9			
SANDY CLAY, FILL brown (CL-FILL) [WASHOUT FINES]	▨		15									
MEDIUM TO COARSE SAND, FILL brown (SP-FILL)	░											
Stopped excavation at 18.0'. Stopped sampling at 12.5'. No groundwater encountered at time of excavation. No signficant sidewall caving.			20 25									

The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary for a proper understanding of the nature of the subsurface material.

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TEST PIT TP-7

Page: 1 of 1

Project Name: Proposed View 62 Development

Project No.: 528-002-18

Location: 6200 S Wasatch Boulevard, Cottonwood Heights, Utah

Client: Rockworth Companies

Excavating Method: Hitachi Trackhoe

Date Excavated: 02-01-18

Elevation: ---

Water Level: No groundwater encountered.

Remarks:

DESCRIPTION	GRAPHIC LOG	WATER LEVEL	DEPTH (FT.)	SAMPLE SYMBOL	SAMPLE TYPE	BLOWS/FT.	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	REMARKS
FINE SANDY CLAY, FILL brown (CL-FILL) [WASHOUT FINES]	▨											
MEDIUM TO COARSE SAND, FILL brown (SP-FILL)	░											
FINE SANDY CLAY, FILL brown (CL-FILL) "perched" moisure at 4.0' [WASHOUT FINES]	▨											very moist "medium stiff"
	▨		5	◢	B		32.0		62.7			
	▨		10									
Stopped excavation at 14.0'. Stopped sampling at 5.5'. No groundwater encountered at time of excavation. Collapsing sidewalls.			15									
			20									
			25									

The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary for a proper understanding of the nature of the subsurface material.

FIGURE 4J

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TEST PIT TP-8

Page: 1 of 1

Project Name: Proposed View 62 Development

Project No.: 528-002-18

Location: 6200 S Wasatch Boulevard, Cottonwood Heights, Utah

Client: Rockworth Companies

Excavating Method: Hitachi Trackhoe

Date Excavated: 02-01-18

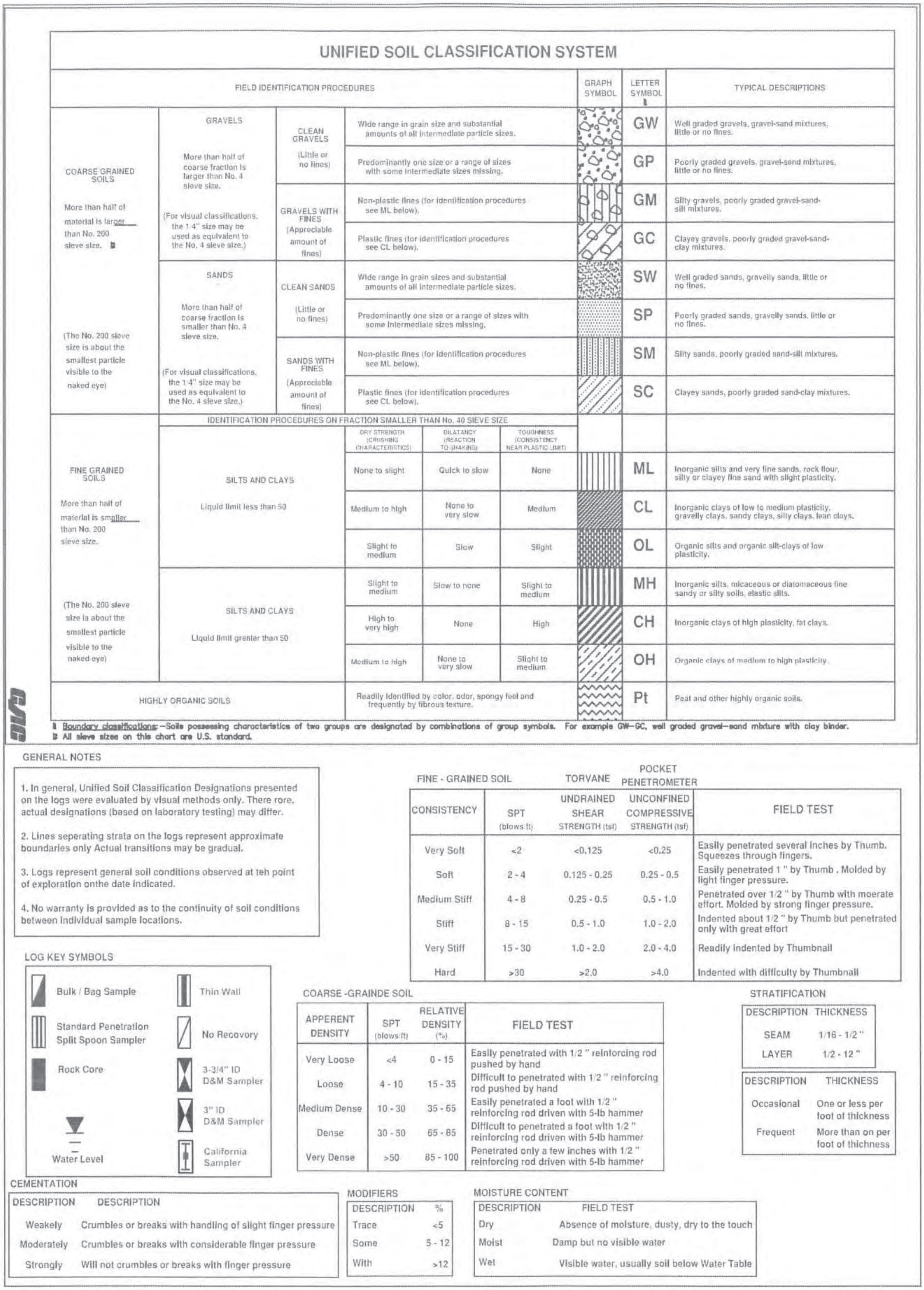
Elevation: ---

Water Level: No groundwater encountered.

Remarks:

DESCRIPTION	GRAPHIC LOG	WATER LEVEL	DEPTH (FT.)	SAMPLE SYMBOL	SAMPLE TYPE	BLOWS/FT.	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	REMARKS
CLAYEY FINE SAND/FINE SANDY CLAY, FILL brown (CL/SC-FILL) [WASHOUT FINES]	▨											very moist "medium dense"
	▨			◢	B		24.3		51.9			
"perched" moisture at 4.5'	▨		5									very moist "medium stiff"
	▨			◢	B		21.5		53.7			
grades with occasional cobbles at 9.0'	▨		10									
			15									
Stopped excavation at 14.0'. Stopped sampling at 6.5'. No groundwater encountered at time of excavation. Collapsing sidewalls.												
			20									
			25									

The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary for a proper understanding of the nature of the subsurface material.



UNIFIED SOIL CLASSIFICATION SYSTEM

FIELD IDENTIFICATION PROCEDURES						GRAPH SYMBOL	LETTER SYMBOL [1]	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS More than half of material is larger than No. 200 sieve size. [2]	GRAVELS More than half of coarse fraction is larger than No. 4 sieve size. (For visual classifications, the 1 4" size may be used as equivalent to the No. 4 sieve size.)	CLEAN GRAVELS (Little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes.			▨	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			Predominantly one size or a range of sizes with some intermediate sizes missing.			▨	GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below).			▨	GM	Silty gravels, poorly graded gravel-sand-silt mixtures.
			Plastic fines (for identification procedures see CL below).			▨	GC	Clayey gravels, poorly graded gravel-sand-clay mixtures.
(The No. 200 sieve size is about the smallest particle visible to the naked eye)	SANDS More than half of coarse fraction is smaller than No. 4 sieve size. (For visual classifications, the 1 4" size may be used as equivalent to the No. 4 sieve size.)	CLEAN SANDS (Little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.			▨	SW	Well graded sands, gravelly sands, little or no fines.
			Predominantly one size or a range of sizes with some intermediate sizes missing.			▨	SP	Poorly graded sands, gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below).			▨	SM	Silty sands, poorly graded sand-silt mixtures.
			Plastic fines (for identification procedures see CL below).			▨	SC	Clayey sands, poorly graded sand-clay mixtures.
FINE GRAINED SOILS More than half of material is smaller than No. 200 sieve size.	IDENTIFICATION PROCEDURES ON FRACTION SMALLER THAN No. 40 SIEVE SIZE							
			DRY STRENGTH (CRUSHING CHARACTERISTICS)	DILATANCY (REACTION TO SHAKING)	TOUGHNESS (CONSISTENCY NEAR PLASTIC LIMIT)			
	SILTS AND CLAYS Liquid limit less than 50		None to slight	Quick to slow	None	▨	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sand with slight plasticity.
			Medium to high	None to very slow	Medium	▨	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			Slight to medium	Slow	Slight	▨	OL	Organic silts and organic silt-clays of low plasticity.
(The No. 200 sieve size is about the smallest particle visible to the naked eye)	SILTS AND CLAYS Liquid limit greater than 50		Slight to medium	Slow to none	Slight to medium	▨	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			High to very high	None	High	▨	CH	Inorganic clays of high plasticity, fat clays.
			Medium to high	None to very slow	Slight to medium	▨	OH	Organic clays of medium to high plasticity.
HIGHLY ORGANIC SOILS			Readily identified by color, odor, spongy feel and frequently by fibrous texture.			▨	Pt	Peat and other highly organic soils.

[1] Boundary classifications:—Soils possessing characteristics of two groups are designated by combinations of group symbols. For example GW—GC, well graded gravel—sand mixture with clay binder.
[2] All sieve sizes on this chart are U.S. standard.

GENERAL NOTES

1. In general, Unified Soil Classification Designations presented on the logs were evaluated by visual methods only. There rore, actual designations (based on laboratory testing) may differ.

2. Lines seperating strata on the logs represent approximate boundaries only Actual transitions may be gradual.

3. Logs represent general soil conditions observed at teh point of exploration onthe date indicated.

4. No warranty is provided as to the continuity of soil conditions between individual sample locations.

FINE - GRAINED SOIL

CONSISTENCY	SPT (blows ft)	TORVANE UNDRAINED SHEAR STRENGTH (tsf)	POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH (tsf)	FIELD TEST
Very Soft	<2	<0.125	<0.25	Easily penetrated several inches by Thumb. Squeezes through fingers.
Soft	2 - 4	0.125 - 0.25	0.25 - 0.5	Easily penetrated 1 " by Thumb . Molded by light finger pressure.
Medium Stiff	4 - 8	0.25 - 0.5	0.5 - 1.0	Penetrated over 1/2 " by Thumb with moerate effort. Molded by strong finger pressure.
Stiff	8 - 15	0.5 - 1.0	1.0 - 2.0	Indented about 1/2 " by Thumb but penetrated only with great effort
Very Stiff	15 - 30	1.0 - 2.0	2.0 - 4.0	Readily indented by Thumbnail
Hard	>30	>2.0	>4.0	Indented with difficulty by Thumbnail

LOG KEY SYMBOLS

Bulk / Bag Sample	Thin Wall
Standard Penetration Split Spoon Sampler	No Recovery
Rock Core	3-3/4" ID D&M Sampler
	3" ID D&M Sampler
Water Level	California Sampler

COARSE -GRAINDE SOIL

APPERENT DENSITY	SPT (blows ft)	RELATIVE DENSITY (%)	FIELD TEST
Very Loose	<4	0 - 15	Easily penetrated with 1/2 " reinforcing rod pushed by hand
Loose	4 - 10	15 - 35	Difficult to penetrated with 1/2 " reinforcing rod pushed by hand
Medium Dense	10 - 30	35 - 65	Easily penetrated a foot with 1/2 " reinforcing rod driven with 5-lb hammer
Dense	30 - 50	65 - 85	Difficult to penetrated a foot with 1/2 " reinforcing rod driven with 5-lb hammer
Very Dense	>50	85 - 100	Penetrated only a few inches with 1/2 " reinforcing rod driven with 5-lb hammer

STRATIFICATION

DESCRIPTION	THICKNESS
SEAM	1/16 - 1/2 "
LAYER	1/2 - 12 "

DESCRIPTION	THICKNESS
Occasional	One or less per foot of thickness
Frequent	More than on per foot of thichness

CEMENTATION

DESCRIPTION	DESCRIPTION
Weakely	Crumbles or breaks with handling of slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumbles or breaks with finger pressure

MODIFIERS

DESCRIPTION	%
Trace	<5
Some	5 - 12
With	>12

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible water, usually soil below Water Table

FIGURE 5

#1 Looking south.

#2 Looking east/southeast.

#3 Looking east.

#4 Looking north.

FIGURE 6
PHOTOGRAPHS

Locations and direction, see Figure 2, Area Map

APPENDIX A

Geologic Hazards Study Report

REPORT

GEOLOGIC HAZARDS EVALUATION
AJ ROCK LLC PROPERTY
6695 SOUTH WASATCH BOULEVARD
COTTONWOOD HEIGHTS, UTAH

Prepared for

Gordon Geotechnical Engineering, Inc.
4426 South Century Drive, Suite 100
Salt Lake City, Utah 84123

May 11, 2020

Prepared by

Western Geologic & Environmental LLC
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May 11, 2020

Patrick R. Emery – Principal Engineer
Gordon Geotechnical Engineering, Inc.
4426 South Century Drive, Suite 100
Salt Lake City, Utah 84123

Letter of Transmittal: REPORT
Geologic Hazards Evaluation
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

Dear Mr. Emery:

Western Geologic & Environmental has completed a Geologic Hazards Evaluation for the AJ Rock LLC Property at 6695 South Wasatch Boulevard in Cottonwood Heights, Utah and submits the attached report for your review.

If you have any questions regarding this report, please contact us at (801) 359-7222.

Sincerely,
Western Geologic & Environmental LLC

05/11/2020
BILL DUANE BLACK
NO. 5224898-2250

Bill. D. Black, P.G.
Subcontract Geologist

Reviewed By:

05/11/2020
KEVIN J THOMAS
11297638-2250
Kevin Thomas

Kevin J. Thomas, P.G.
Principal Geologist

C:\Users\GLENDA\Documents\WG&E\PROJECTS\Gordon Geotechnical Engineering, Inc\Cottonwood Height, UT - Geo Haz Eval - 6695 South Wasatch Boulevard #5342\Geo Haz Eval - AJ Rock LLC Property - 6695 S Wasatch Blvd - Cottonwood Heights, UT.docx

WG&E Project No. 5342

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FIGURES

1.0 INTRODUCTION

This report presents the results of a geology and geologic hazards review and evaluation conducted by Western Geologic & Environmental LLC (Western Geologic) for the AJ Rock LLC property at roughly 6695 South Wasatch Boulevard in Cottonwood Heights City, Utah (Figure 1 – Project Location). The site is in eastern Salt Lake Valley at the western base of the Wasatch Range north of Big Cottonwood Canyon, in the SE¼ Section 23, Township 2 South, Range 1 East (Salt Lake Base Line and Meridian). Elevation of the site is about 4,820 to 5,010 feet above sea level. The site has been an active gravel mining operation since the mid-1950s and considerable material (up to 100 feet or more in the eastern part) has been removed. Much of the site is mantled by fill of varying thicknesses from gravel mining operations. Based on an April 24, 2020 McNeil Engineering conceptual grading plan, the site is currently proposed for mixed-used development by five commercial buildings, a hotel, a large apartment building, a condominium tower, a senior living center, various ancillary parking areas and re-alignment of Wasatch Boulevard.

2.0 PURPOSE AND SCOPE

The purpose and scope of this investigation is to identify and interpret surficial geologic conditions at the site to identify potential risk from geologic hazards to the Project. This investigation is intended to: (1) provide preliminary geologic information and assessment of geologic conditions at the site; (2) identify potential geologic hazards that may be present and qualitatively assess their risk to the intended site use; and (3) provide recommendations for additional site- and hazard-specific studies or mitigation measures, as may be needed based on our findings. Such recommendations could require further multi-disciplinary evaluations, and/or may need design criteria that are beyond our professional scope. Our investigation was conducted concurrently with a geotechnical engineering study performed at the Project by Gordon Geotechnical.

Maps included in Appendix A of the Cottonwood Heights City's Sensitive Lands Evaluation & Development Standards (SLEDS; Cottonwood Heights City Municipal Code Title 19, Chapter 19.72) show the property is located in Surface Fault Rupture Study Area (Map 1), a "High" Slope Stability Hazard Area (Map 2), a "Very Low" Liquefaction Hazard Area (Map 3), a "Low" Debris Flow Hazard Area (Map 4), and a "Moderate" Rock Fall Hazard Area (Map 5). Appendix A, Map 10, provides a surficial geologic map based on U.S. Geological Survey Map I-2106 (Personius and Scott, 1992), which is incorporated into Personius and Scott (2009).

2.1 Methodology

The following services were performed in accordance with the above-stated purpose and scope:

- A site reconnaissance conducted by an experienced certified engineering geologist to assess the site setting and look for adverse geologic conditions;

- Review of readily-available geologic maps, reports, and air photos;
- Logging of eight exploratory trenches at the site in 2009 and one trench in 2020 to identify the presence and location of any active faults, assess zones of fault-related deformation, and recommend appropriate fault set-back distances and safe "buildable" areas should faults be discovered;
- Preparation of three cross section profiles based on site-specific subsurface data and inferred conditions; and
- Evaluation of available data and preparation of this report, which presents the results of our study.

The engineering geology section of this report has been prepared in accordance with Bowman and Lund (2016), current generally accepted professional engineering geologic principles and practice in Utah, and the Cottonwood Heights City SLEDS. However, we do not include discussion of radon hazard potential, as recommended in Bowman and Lund (2016), because radon gas poses an environmental health hazard and indoor levels are heavily influenced by several post-construction, non-geologic factors. The hazard from radon should be evaluated by long-term testing following construction.

2.2 Limitations and Exceptions

This investigation was performed at the request of the Client using the methods and procedures consistent with good commercial and customary practice designed to conform to acceptable industry standards. The analysis and recommendations submitted in this report are based upon the data obtained from site-specific observations and compilation of known geologic information. This information and the conclusions of this report should not be interpolated to adjacent properties without additional site-specific information. In the event that any changes are later made in the location of the proposed site, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report modified or approved in writing by the engineering geologist.

This report has been prepared by the staff of Western Geologic for the Client under the professional supervision of the principal and/or senior staff whose seal(s) and signatures appear hereon. Neither Western Geologic, nor any staff member assigned to this investigation has any interest or contemplated interest, financial or otherwise, in the subject or surrounding properties, or in any entity which owns, leases, or occupies the subject or surrounding properties or which may be responsible for environmental issues identified during the course of this investigation, and has no personal bias with respect to the parties involved.

The information contained in this report has received appropriate technical review and approval. The conclusions represent professional judgment and are founded upon the findings of the investigations identified in the report and the interpretation of such data based on our experience and expertise according to the existing standard of care. No other warranty or limitation exists, either expressed or implied.

The investigation was prepared in accordance with the approved scope of work outlined in our proposal for the use and benefit of the Client; its successors, and assignees. It is based, in part, upon documents, writings, and information owned, possessed, or secured by the Client. Neither this report, nor any information contained herein shall be used or relied upon for any purpose by any other person or entity without the express written permission of the Client. This report is not for the use or benefit of, nor may it be relied upon by any other person or entity, for any purpose without the advance written consent of Western Geologic.

In expressing the opinions stated in this report, Western Geologic has exercised the degree of skill and care ordinarily exercised by a reasonable prudent environmental professional in the same community and in the same time frame given the same or similar facts and circumstances. Documentation and data provided by the Client, designated representatives of the Client or other interested third parties, or from the public domain, and referred to in the preparation of this assessment, have been used and referenced with the understanding that Western Geologic assumes no responsibility or liability for their accuracy. The independent conclusions represent our professional judgment based on information and data available to us during the course of this assignment. Factual information regarding operations, conditions, and test data provided by the Client or their representative has been assumed to be correct and complete. The conclusions presented are based on the data provided, observations, and conditions that existed at the time of the field exploration.

3.0 GEOLOGY

3.1 Surficial Geology

Utah Geological Survey Map 243DM (Surficial geologic map of the Salt Lake City segment and parts of adjacent segments of the Wasatch fault zone, Davis, Salt Lake, and Utah Counties, Utah; Personius and Scott, 2009) maps the site in an area underlain by Bells Canyon glacial outwash; and clay, silt, sand, and gravel related to the transgressive stage of Lake Bonneville. Two main, west-dipping traces of the active Salt Lake City section of the WFZ are mapped by Personius and Scott (2009) diverging from a single trace southeast of the site into two northwest-trending en-echelon traces that cross the Project. A third main west-dipping trace is mapped by Personius and Scott (2009) near the northeastern site corner.

More-recent, 1:24,000-scale mapping by McKean (2018) and McKean and Solomon (2018) is provided on Figure 2. McKean (2018) shows the Project straddles a broad zone of fault deformation bounded by two west-dipping main traces of the WFZ on the east and west, with a third partly concealed west-dipping trace crossing the site (Figure 2). All these fault traces diverge from a single trace further south similar to Personius and Scott (2009). McKean (2018) maps the site in an area underlain by Holocene to upper Pleistocene stream deposits, upper Pleistocene deltaic deposits related to the Bonneville shoreline and transgressive phase of Lake Bonneville, and lacustrine gravel and sand of similar provenance (units Qaly, Qdlb and Qlgb; Figure 2). McKean (2018) describes these units as follows:

***Qaly* –** Young stream deposits, undivided (Holocene to upper Pleistocene). Poorly to moderately sorted pebble and cobble gravel, locally bouldery, with a matrix of sand and silt; mapped in channels and active floodplains of Big Cottonwood, Parleys, Emigration, and Mill Creeks and small creeks; locally includes small alluvial-fan and colluvial deposits; includes level-2 stream deposits (Qal2) incised by active streams with level-1 stream deposits (Qal1); Qal1 and Qal2 deposits cannot be mapped separately, due to lack of bars and swales and because patches of deposits are too small to show separately at map scale; postdates regression of Lake Bonneville from the Provo shoreline and lower shorelines; thickness variable, probably less than 30 feet (10 m).

Deposits related to the Bonneville shoreline and transgressive phase of Lake Bonneville: Mapped between the Bonneville and Provo shorelines. The Bonneville shoreline is at elevations from about 5160 to 5230 feet (1570–1595 m) in the Sugar House quadrangle (table 1).

***Qldb* –** Deltaic deposits, undivided (upper Pleistocene). Moderately to well-sorted gravel and sand, locally including thin beds of silt and sandy silt; clasts subrounded to rounded; thin to thick planar and cross-bedded foreset beds; locally includes topset alluvial beds; locally weakly cemented with calcium carbonate; undivided (Qldb) where exposed in bluffs between streams or below terraces, subdivided into a gravelly unit (Qldbg) where delta contains clast-supported, pebble and cobble gravel in a matrix of sand and silt; present near the mouth of Big Cottonwood Canyon; previously mapped near the mouth of Big Cottonwood Canyon as outwash of Bells Canyon age by Personius and Scott (1992; gbco), but mapped here as deltaic based on the delta fan-shape and moderately to well-sorted, well-rounded gravel and sand in planar and foreset beds exposed in sand and gravel pits; exposed thickness less than 130 feet (40 m).

***Qlgb* –** Lacustrine gravel and sand (upper Pleistocene). Moderately to well-sorted, clast-supported, pebble to cobble gravel, with boulders near bedrock sources, with a matrix of sand and pebbly sand; locally interbedded with thin beds and lenses containing silt and clay; clasts commonly subrounded to rounded, but some deposits consist of poorly sorted, angular gravel derived from nearby bedrock outcrops; deposited between the Bonneville and Provo shorelines in planar and cross-bedded beds; typically overlies bedrock near the foot of the Wasatch Range; commonly covered by unmapped colluvium from adjacent steep slopes on erosional benches at the Bonneville shoreline; this colluvium is thin and does not cover the benches; exposed thickness less than 75 feet (25 m).

Citations, tables and/or figures referenced above, and descriptions of nearby surficial geologic units shown on Figure 2 are not provided herein but are in McKean (2018).

3.2 Seismotectonic Setting

The property is located in Salt Lake Valley at the western base of the Wasatch Range about 1.1 miles northwest of the mouth of Big Cottonwood Canyon. Salt Lake Valley is a deep, sediment-filled structural basin of Cenozoic age that is bounded by two uplifted range blocks, the Oquirrh Mountains and the Wasatch Range (to the west and east, respectively). The valley lies at the eastern edge of the Basin and Range physiographic province (Stokes, 1977, 1986). The Basin and Range province is characterized by a series of generally north-trending elongate mountain ranges, separated by predominately alluvial and lacustrine sediment-filled valleys and typically bounded on one or both sides by major normal faults (Stewart, 1978). The boundary between the Basin and Range and Middle Rocky Mountains provinces is the prominent, west-facing escarpment along the Wasatch fault zone at the western base of the Wasatch Range. Late Cenozoic normal faulting, a characteristic of the Basin and Range, began between about 17 and 10 Ma (million years ago) in the Nevada (Stewart, 1980) and Utah (Anderson, 1989) portions of the province. The faulting is a result of a roughly east-west directed, regional extensional stress regime that has continued to the present (Zoback and Zoback, 1989; Zoback, 1989).

The Wasatch fault zone (WFZ) is one of the longest and most active normal-slip faults in the world and extends for 213 miles along the western base of the Wasatch Range from southeastern Idaho to north-central Utah (Machette and others, 1992). The fault zone generally trends north-south and, at the surface, can form a zone of deformation up to several hundred feet wide containing many subparallel west-dipping main faults and east-dipping antithetic faults. Previous studies divided the fault zone into 10 sections, each of which rupture independently and are capable of generating large-magnitude surface-faulting earthquakes (Machette and others, 1992). The central five sections of the fault (Brigham City, Weber, Salt Lake, Provo, and Nephi) have each produced two or more surface-faulting earthquakes in the past 6,000 years (Black and others, 2003).

The site is located along the active Salt Lake City section of the WFZ, which trends across the heavily populated east side of Salt Lake Valley. The Salt Lake City section is further divided into three subsections (from north to south): Warm Springs, East Bench, and Cottonwood. The site is located at the northern end of the Cottonwood (southernmost) subsection.

Personius and Scott (2009) and McKean (2018; Figure 2) map three main traces of the WFZ that bound and/or cross the site and trend generally northward. The faults form a broad zone of en-echelon, down-to-the-west faulting from 700 to 1,200 feet wide on which the site is situated. The Working Group on Utah Earthquake Probabilities (2016; Table 4.1-1) indicates mean timing (+ 2σ) for the last four surface-faulting earthquakes on the Salt Lake City section is: (1) 1,300 + 200 years, (2) 2,200 + 200 years, (3) 4,100 + 300 years, and (4) 5,300 + 200 years. The Working Group on Utah Earthquake Probabilities (2016; Table 4.1-2) indicates a closed mean recurrence interval for the Salt Lake City section, based on timing for the last four surface-faulting earthquakes, of 1,300 + 100 years.

The site is also in the central portion of the Intermountain Seismic Belt (ISB), a generally north-south trending zone of historical seismicity along the eastern margin of the Basin and Range province extending from northern Arizona to northwestern Montana (Sbar and others, 1972; Smith and Sbar, 1974). At least 16 earthquakes of magnitude 6.0 or greater have occurred within the ISB since 1850; the largest of these earthquakes was a M 7.5 event in 1959 near Hebgen Lake, Montana. None of these earthquakes occurred along the Wasatch fault or other known late Quaternary faults (Arabasz and others, 1992; Smith and Arabasz, 1991). The closest event was the 1934 Hansel Valley (M 6.6) event north of the Great Salt Lake. The March 18, 2020 M 5.7 earthquake north of Magna, Utah reportedly showed a style, location, and slip depth consistent with an earthquake on the Wasatch fault system (https://earthquake.usgs.gov/earthquakes/eventpage/uu60363602/executive). Despite being moderate in size (less than magnitude 6.0), this earthquake was felt from southern Idaho to south-central Utah and caused serious damage to multiple buildings (https://www.ksl.com/article/46731630/).

3.3 Lake Bonneville History

Lakes occupied nearly 100 basins in the western United States during late-Quaternary time, the largest of which was Lake Bonneville in northwestern Utah. The Bonneville basin consists of several topographically closed basins created by regional extension in the Basin and Range (Gwynn, 1980; Miller, 1990), and has been an area of internal drainage for much of the past 15 million years. Lake Bonneville consisted of numerous topographically closed basins, including the Salt Lake and Cache Valleys (Oviatt and others, 1992). Sediments from Lake Bonneville comprise much of the unconsolidated deposits in the site vicinity.

Timing of events related to the transgression and regression of Lake Bonneville are indicated in Oviatt (2015). Approximately 30,000 years ago, Lake Bonneville began a slow transgression (rise) to its highest level of 5,160 to 5,200 feet above mean sea level. The lake rise eventually slowed as water levels approached an external basin threshold in northern Cache Valley at Red Rock Pass near Zenda, Idaho. Lake Bonneville reached the Red Rock Pass threshold and occupied its highest shoreline, termed the Bonneville beach, around 18,000 years ago. Headward erosion of the Snake River-Bonneville basin drainage divide, possibly combined with landsliding in the threshold area, then caused a catastrophic incision that caused the lake level to lower by about 425 feet in less than a year (Jarrett and Malde, 1987; O’Conner, 1993). Following the Bonneville flood, the lake stabilized and formed a lower shoreline referred to as the Provo shoreline up to about 16,000 years ago. Climatic factors then caused the lake to regress rapidly from the Provo shoreline, and by about 13,000 years ago the lake had eventually dropped below historic levels of Great Salt Lake. Oviatt and others (1992) deem this low stage the end of the Bonneville lake cycle. Great Salt Lake then experienced a brief transgression between 12,800 and 11,600 years ago to the Gilbert level at about 4,250 feet before receding to and remaining within about 20 feet of its historic average level (Lund, 1990; Oviatt, 2015). The site is located slightly above the Provo shoreline, but below the highest Bonneville shoreline.

Glaciers in Little Cottonwood and Bells Canyons advanced into eastern Salt Lake Valley from the Wasatch Range between 26,000 and 18,000 years ago (Personius and Scott, 1992, 2009). Lake Bonneville was in its transgressive stage during this time, but stood at an intermediate level prior to reaching its highest Bonneville shoreline. Till deposited by the glaciers formed prominent moraines extending into the valley, and meltwater from glaciers in Bells Canyon and Little and Big Cottonwood Canyons deposited gravelly outwash fans along the range front (Personius and Scott, 1992). The site is partly mapped in outwash deposits from Big Cottonwood Canyon (unit gbco, Figure 2). As Lake Bonneville continued rising, the glaciers retreated up their respective valleys, the outwash deposits were eventually inundated by the lake, and deposition continued in deltas extending into the lake. When Lake Bonneville receded, the deltas and outwash deposits were downcut and eroded by Big and Little Cottonwood Creeks.

4.0 SITE CHARACTERIZATION

4.1 Empirical Observations

On March 25-26, 2020, Mr. Bill D. Black of Western Geologic conducted a brief reconnaissance of the property to observe geomorphic and surficial conditions. The reconnaissance was conducted in conjunction with additional subsurface investigation in the northern part of the site. Weather on March 25 was cloudy with snow and temperatures in the 30's (°F). The site is in eastern Salt Lake Valley at the western base of the Wasatch Range north of Big Cottonwood Canyon and has been an active gravel mining operation since the mid-1950s. Considerable material has been removed in the fault zone by gravel mining, with subvertical cut slopes a hundred feet high or more in this area. Much of the site is mantled by fill of varying thicknesses from gravel mining operations. The gravel pit was not in operation at the time of our reconnaissance, but was active when most of the trenching was conducted at the site in 2009. No springs or seeps were observed at the Project and no evidence for characteristic debris-flow features, landslides, recent or ongoing slope instability, rock fall source areas, or other geologic hazards was observed to the extent that surface areas at the Project could be accessed and viewed.

4.2 Air Photo Observations

Figure 3A shows a 1938 pre-gravel mining air photo of the site from historical photography flown for the Salt Lake Aqueduct Project (frames sla1-20 and sla1-21, original scale 1:20,000; Bowman and Beisner, 2008). The air photo center was approximately registered to the UTM NAD83 grid system by Bowman and Beisner (2008). However, we further adjusted the photo scale, rotation, and placement to correspond to range front bedrock exposures evident on the 1938 photos and 2006 U.S. Geological Survey digital orthophotography available from Utah AGRC. The 1938 photos were then enlarged and overlaid with the site boundary for stereo viewing. Several northwest-trending escarpments were observed on Figure 3A crossing the site, which correspond to locations of significant faults encountered in the trenches (discussed below and shown in red, with bar and ball on downthrown side). Except for some broad correlations, surficial faulting evidence is obscured by gravel mining disturbance on 1977 and later air photos and a 2013 geoprocessed LIDAR image available from the Utah AGRC (Figures 3B through 3E). No evidence for other geologic hazards was observed on the air photos at the site or in the area.

4.3 Subsurface Investigation

Eight trenches were excavated and logged at the Project in 2009 and one trench was excavated and logged in 2020 to evaluate subsurface geologic conditions and assess the potential hazard from surface faulting. The 2009 investigation was conducted prior to formalization of the current Cottonwood Heights City SLEDS, based on an estimated timeline provided by Tim Thompson of GeoStrata. No work plan was prepared for the 2009 investigation, and no Project scoping or field reviews were conducted. A work plan dated March 2, 2020 was prepared for the 2020 investigation that was approved by Tim Thompson of GeoStrata on March 17, 2020. A field review for trench T-9 was conducted with Mr. Thompson on March 27, 2020.

Trench excavation and logging in 2009 was performed on weekends to facilitate backfilling and restoration and allow for unrestricted construction vehicle access prior to active operations each following Monday. Subsurface exploration was limited to accessible areas not mantled by large gravel piles, such as along roads, and further restricted by the easement for the aqueduct crossing the site. No exploration was conducted in steep areas of the eastern part of the Project (east of the steep escarpments from gravel mining) and no long continuous trench exposures were feasible. The trenches were excavated to a safe depth sufficient to expose lacustrine sediments from Lake Bonneville capable of displaying active faulting and providing good chronostratigraphic markers. Deep fill materials were encountered in places that complicated excavation and logging, such as from active and inactive utility lines, old pit excavations, backfilled settling ponds, and past grading activities. Although native sediments were exposed, excavation in some areas could not extend deep enough to expose correlative stratigraphy across exposed faults. The trenches also exposed bedded fills that appeared similar to native sediments, which we do not consider unusual given the site use as a gravel pit operation; in general, we interpreted fills where sediments contained anomalous materials or had an abnormal appearance from soil organics inclusion, conservatively erring on the side of a fill interpretation.

Figure 4 is a site plan at a scale of 1:1,200 (1 inch equals 100 feet) showing the site boundary, current development plan, locations for the trenches conducted for our study, locations of three Gordon Geotechnical borings conducted in March 2020, and exposed faults in the trenches (shown by small red lines, with bar and ball on the downthrown side). Trench locations were measured in the field using a handheld GPS and by trend and distance methods, and subsequently surveyed to provide positional accuracy. The 2009 trenches were surveyed by Benchmark Engineering and the 2020 trench (T-9) was surveyed by McNeil Engineering. Surveyed elevations for significant faults are tagged in blue on Figure 4 to show the highest point of the fault in the trench exposure. Fault elevation is shown because the site has been and will be subject to significant surface modification, which may change the fault location depending on dip direction, angle, and amount of surface material removed. The trenches generally provide good overlapping coverage given a presumed overall fault trend of about N15°W.

The trenches at the Project were excavated in three general alignments: (1) a southern alignment formed by T-1, T-2, T-7, and T-8; (2) a middle alignment consisting of T-3, T-4, T-5, and T-6; and (3) a northern alignment consisting of T-9 (Figure 4). T-1 extended an overall S87°W for a total distance of 125 feet (stations -5.0 feet to 120.0 feet, east to west, Figures 4 and 5). T-2 extended an overall S64°W for a total distance of 280 feet (stations -5.0 feet to 275.0 feet, east to west, Figures 4 and 6). T-3 extended an overall S85°E for a total distance 192.5 feet (stations -1.5 feet to 191.0 feet, west to east, Figures 4 and 7). T-4 extended an overall S73°E for a total distance of 211.4 feet (stations -4.0 to 207.4 feet, west to east, Figures 4 and 8). T-5 extended sinuously an overall S82°E for a total distance of 339 feet (stations -5.0 feet to 334.0 feet, west to east, Figures 4 and 9). T-6 extended an overall S77°W for a total distance of 171 feet (stations -5.0 feet to 166.0 feet, east to west, Figures 4 and 10). T-7 extended an overall N79°E for a total distance of 146.3 feet (stations -3.4 feet to 143.0 feet, west to east, Figures 4 and 11). T-8 extended an overall S77°W for a total distance of 373.1 feet (stations -3.1 feet to 370 feet, east to west, Figures 4 and 12). T-9 extended an overall N87°W for a total distance of 218 feet (stations -5.0 feet to 218.0 feet, east to west, Figures 4 and 13).

Figures 5 through 13 are detailed logs of the trenches at a scale of 1:60 (1 inch equals 5 feet). Due to space restraints and the scale of the logs, all of the trenches cover multiple sheets. Except for some schematic clasts and within the defined log scale, the logs generally accurately depict notable bedding and texture observed in the trenches. With the exception of trench T-2, which was excavated west from the east end and then east from west end to mate up, original stations and logging direction are preserved on the logs. Trench locations are shown on Figure 4 with stations denoted on the logged wall. The trenches were digitally photographed to document the exposures at either 5- or 10-foot intervals. These photos are not provided herein, but are available on request. Trench logging generally followed methodology in McCalpin (1996), with the exception that soil horizons were not logged due to surficial disturbance.

The trenches at the site mainly exposed a well-bedded sequence of lacustrine-deltaic fine sand and silt that coarsened eastward to sandy, cobbly, and bouldery crossbedded gravels. Alluvium may have been present at the surface overlying the lacustrine-deltaic sediments, but was stripped off during early gravel pit activities and was not observed in the trenches. The depositional sequence exposed in the trenches at the site consisted of (from oldest to youngest): (1) a lower, strongly east-dipping, crossbedded sandy gravel below an intra-unit angular unconformity and an overlying west-dipping cobbly to bouldery gravel (exposed as unit 1a in T-5 and T-7, and unit 2 in T-6; Figures 9 through 11); (2) a thin unit of deformed sand to silt, likely from a low-energy landslide that occurred subaqueously in Lake Bonneville shortly after its transgression across the site (exposed as unit 1a in T-3, and 1b in T-1, T-2, T-4, and T-5; Figures 5-9); and (3) a sequence of interbedded and crossbedded sand and gravel deposits with lesser silt (exposed as unit 1c in T-1, T-5, and T-7; units 1c and 1d in T-2 and T-4; unit 1b in T-3; and unit 1g in T-8; Figures 5 through 9, 11, and 12). Interbedded and interfingering sand and gravel was exposed in trench T-9 that we infer corresponds to the latter. The sediments likely represent glacial outwash from Big Cottonwood Canyon accumulating in the delta emanating from the canyon mouth, followed by Lake Bonneville inundation and subsequent deltaic deposition in the lake. Trench T-8 also exposed lacustrine clay and gravelly clay likely deposited in a lagoon behind a longshore barrier bar (units 1e an 1f, Figure 12) , and T-6 exposed a pre-Lake Bonneville or near-shore landslide deposit comprised of lean to fat blue-gray clay with mineralized wood debris and bone fragments in the footwall of fault F9 (unit 1, Figure 10).

All of the trenches at the site, except for T-5, exposed one or more faults that displace the Lake Bonneville stratigraphic sequence. No evidence for faulting was observed in T-5. Major faults showing more than 4 feet of displacement were observed in trenches T-3, T-6, T-7, and T-8, corresponding to three main, west-dipping, en-echelon traces (from west to east): (1) a trace formed by F1 and F2 on the west side of the project, which appears to converge northward; (2) fault F7 in the central part of the project; and (3) faults F8 and F9 in the eastern part of the project, which also converge northward (Figure 4). The faults correspond to visible west-facing escarpments on Figure 3A that form a series of steps from west to east across the site. Minor faults with between 0.3 and 4.0 feet of displacement were observed in T-2, T-4, and T-7, corresponding to faults F2 through F6. These faults converge northward with F1 and F7 (Figure 3A). Two antithetic faults were also exposed in the trenches: (1) AF1 on the west side of the project in the westernmost escarpment on Figure 4, and (2) AF2 on the east side, which forms a graben bounded by a fourth major west-dipping fault to the east on Figure 3A. Small displacement faults with less than 0.3 feet of displacement were observed in T-1, T-2, and T-4. The small displacement faults in T-2 and T-4 are in the F3/F4 zone, whereas the faults in T-1 appear to be unrelated and do not correspond to any surficial features on Figure 3A. Trench T-9 exposed a fault with 0.5 feet of down-to-the-west offset that displaces and is overlain by lacustrine sand and gravel (which is suggestive an intra-lacustrine event), a younger fault with 0.7 feet of down-to-the-west offset that displaces the lacustrine sequence to the fill base, and a narrow zone of crack fill in the lacustrine sequence that displayed no net displacement. A lineament is on the 1938 air photo that suggests this fault (F10, Figure 4) trends to the southeast and then bends eastward to converge with F9. No trenching could be conducted to confirm the fault location southeast of trench T-9 due to the aqueduct easement.

4.4 Cross Sections

Figures 14 through 16 shows three cross sections (A-A’, B-B’, and C-C’) across the site as located on Figure 4. Figure 14 is at a scale of 1 inch equals 25 feet, Figure 15 is at a scale of 1 inch equals 50 feet, and Figure 16 is at a scale of 1 inch equals 40 feet with no vertical exaggeration. The topographic profiles are based on geoprocessed 2013 LIDAR data. The LIDAR data provides a snapshot of topographic conditions at the time it was acquired; past, present and future topographic conditions may vary. Cross Section A-A’ is based on the stratigraphic sequence in trenches T-5, T-6, and T-9. Cross Section B-B’ is based on the stratigraphic sequence in trenches T-5 and T-6 and subsurface data from Gordon Geotechnical boring B-3. Cross section C-C’ is based on the stratigraphic sequence in trenches T-6, T-7, and T-8, as well as subsurface data from Gordon Geotechnical boring B-2. Units and contacts should be considered approximate and inferred, and variations should be expected at depth and laterally. We caution that some portions of the cross sections have limited or no subsurface data.

No groundwater was encountered in Gordon Geotechnical boring B-3 or any of the trenches at the site, except for in the base of the old tank excavation in trench T-2 (Figure 6C, stations 155 feet to about 190 feet). Groundwater in trench T-2 was at a depth of about 13 feet below the ground surface (bgs), and was encountered at a depth of 22.5 feet bgs in boring B-1 and at a depth of 29 feet bgs in boring B-2 (Figure 4). Based on this, groundwater deepens between trench T-2 and boring B-2 from 13 to 29 feet deep, and deepens to more than 100 feet between borings B-2 and B-3. These data suggest a southwestward flow direction. Inferred groundwater levels are shown on the cross sections.

5.0 GEOLOGIC HAZARDS

Assessment of potential geologic hazards and the resulting risks imposed is critical in determining the suitability of the site for development. Table 1 below shows a summary of the geologic hazards reviewed at the site, as well as a relative (qualitative) assessment of risk to the Project for each hazard. A "high" hazard rating (H) indicates a hazard is present at the site (whether currently or in the geologic past) that is likely to pose significant risk and/or may require further study or mitigation techniques. A "moderate" hazard rating (M) indicates a hazard that poses an equivocal risk. Moderate-risk hazards may also require further studies or mitigation. A "low" hazard rating (L) indicates the hazard is not present, poses little or no risk, and/or is not likely to significantly impact the Project. Low-risk hazards typically require no additional studies or mitigation. We note that these hazard ratings represent a conservative assessment for the entire site and risk may vary in some areas. Careful selection of development areas can minimize risk by avoiding known hazard areas.

Table 1. *Geologic hazards summary.*

Hazard	H	M	L
Earthquake Ground Shaking	X		
Surface Fault Rupture	X		
Liquefaction and Lateral-spread Ground Failure		X	
Tectonic Deformation	X		
Seismic Seiche and Storm Surge			X
Stream Flooding			X
Shallow Groundwater		X	
Landslides and Slope Failures		X	
Debris Flows and Floods			X
Rock Fall			X
Problem Soil and Rock			X

5.1 Earthquake Ground Shaking

Ground shaking refers to the ground surface acceleration caused by seismic waves generated during an earthquake. Strong ground motion is likely to present a significant risk during moderate to large earthquakes located within a 60 mile radius of the Project area (Boore and others, 1993). Seismic sources include mapped active faults, as well as a random or "floating" earthquake source on faults not evident at the surface. The Utah Geological Survey Quaternary Fault Database (Black and others, 2003; January 2017 update) shows numerous class A faults within 60 miles of the Project that may pose potential seismic sources.

The extent of property damage and loss of life due to ground shaking depends on factors such as: (1) proximity of the earthquake and strength of seismic waves at the surface (horizontal motions are the most damaging); (2) amplitude, duration, and frequency of ground motions; (3) nature of foundation materials; and (4) building design. Based on 2018 IBC provisions, a site class of D (stiff soil), and a risk category of II, calculated seismic values for the site (centered on 40.6300° N, -111. 7979° W) are summarized below:

Table 2. *Seismic hazards summary.*

Type	Value
S_S	*1.341 g*
S_1	*0.498 g*
S_{MS} (F_a x S_S)	*1.341 g*
S_{M1} (F_v x S_1)	*See ASCE 7-16 Section 11.4.8*
S_{DS} (2/3 x S_{MS})	*0.894 g*
S_{D1} (2/3 x S_{M1})	*See ASCE 7-16 Section 11.4.8*
Site Coefficient, F_a	*= 1.000*
Site Coefficient, F_v	*See ASCE 7-16 Section 11.4.8*
Peak Ground Acceleration, PGA	*= 0.609 g*

Given the above information, earthquake ground shaking poses a high risk to the site. Earthquake ground shaking is a regional hazard common to all Wasatch Front areas. The hazard is mitigated by design and construction in accordance with the current adopted building code. We note that IBC 2018 provisions require calculation of the spectral acceleration value (S_{M1}), seismic design value (S_{D1}), and site coefficient (F_v) differently from IBC 2015. In municipalities where IBC 2018 has been adopted, the Project engineer or architect should determine these seismic values in accordance with ASCE 7-16 Section 11.4.8 guidelines.

5.2 Surface Fault Rupture

Movement along faults at depth generates earthquakes. During earthquakes larger than Richter magnitude 6.5, ruptures along normal faults in the intermountain region generally propagate to the surface (Smith and Arabasz, 1991) as one side of the fault is uplifted and the other side down dropped. The resulting fault scarp has a near-vertical slope. The surface rupture may be expressed as a large singular rupture or several smaller ruptures in a broad zone. Ground displacement from surface fault rupture can cause significant damage or even collapse to structures located on an active fault.

All of the trenches at the site, except for T-5, exposed one or more faults that displace the Lake Bonneville stratigraphic sequence. No evidence for faulting was observed in T-5. Faults displaying 0.3 feet or more of displacement in the trench exposures are correlated across the site on Figure 4 (bold red dashed lines) based on trend, displacement sense and air photo evidence (Section 4.2). The faults are labeled for reference purposes with “F” where west dipping and “AF” where east dipping, and appended with a number (1 through

10 for west-dipping faults, and 1 or 2 for east-dipping faults) to denote specific traces (Figure 4). Small displacement faults (less than 0.3 feet) are noted where they were encountered in a trench, but are not correlated. With the exception of trench T-1, all of the small displacement faults were observed in existing fault zones with larger displacements. Table 3A is a compilation of fault data from the trenches at the site, and shows the log station of each trenched fault, fault trends, and dip angles.

Given the above, the risk from surface faulting is high at the site. Based on our current understanding that surface fault rupture and deformation tend to follow past patterns, we recommend a non-buildable (setback) zone around the projected traces of the fault crossing the site as shown on Figure 4. Calculated setback distances based on the fault parameters and guidelines in Lund and others (2016) are also indicated on Table 3A. Recommended setback distances are shown on Table 3B. The fault setback for the downthrown side of active faults at the Project was calculated using:

$$\mathbf{S = U\ (2D + F/tan\theta)}$$

where:

S = Setback distance from active faults;

U = Criticality factor (2.0 for IBC class IIb structures);

D = Expected maximum fault displacement per event (assumed to be the measured vertical displacement or, if not measured or confidently determined, a maximum displacement of 8.5 feet is used; all displacements are conservatively assumed to be from a single event unless there is evidence otherwise);

F = Maximum depth of footing or subgrade portion of the building (assumed to be 8 feet); and

θ *= Dip of the fault.*

The fault setback for the upthrown side of the faults was calculated using the same parameters and:

$$\mathbf{S = U\ (2D)}$$

Small displacement faults (< 0.3 feet of offset) are not listed on Table 3A, but two such faults were observed in trench T-1 underlying Pad E (Figures 4 and 5A). These faults show no evidence for Holocene reactivation that would suggest a future larger displacement is likely. We believe the faults pose a low life-safety risk, but recommend the structure be designed to withstand up to 0.3 feet of vertical offset to reduce the risk of costly repairs. Utility lines that cross faults should also be engineered to withstand expected displacements and/or have design features to ensure life safety.

Table 3A. *Fault parameters and calculated setbacks; fault numbers correspond to Figure 4.*

TRENCH 2, east to west.										
								Setback Distance (S), F=8		Safety Factor[4]
Fault	Elevation[1]	Station[2]	Width (ft)[3]	Trend	Dip (°Θ)	D (ft)	TanΘ	UFS	DFS	
F4	4813.08	10.9	0	N9°W	65	1.0	2.1	4.0	11.5	0.9
F3	4813.57	35.7	0.6	N15°W	68	2.3	2.5	9.2	16.3	0.8
F2	4821.44	219.5-226.1	6.3	N10°W - N15°W	69	2.3	2.6	9.2	21.6	0.8
AF1	4819.36	271.7	0	N10°W	78	1.2	4.7	4.8	8.2	0.4

TRENCH 3, west to east.										
								Setback Distance (S), F=8		Safety Factor[4]
Fault	Elevation[1]	Station[2]	Width (ft)[3]	Trend	Dip (°Θ)	D (ft.)	TanΘ	UFS	DFS	
F1	4810.43	14.9	0	N15°W	59	8.5	1.7	34.0	43.6	1.2
AF1	4810.49	20.2	0	N12°W	47	4.8	1.1	19.2	34.1	1.9
F2	4818.12	48.8	0	N16°W	53	8.5	1.3	34.0	46.1	1.5

TRENCH 4, west to east.										
								Setback Distance (S), F=8		Safety Factor[4]
Fault	Elevation[1]	Station[2]	Width (ft)[3]	Trend	Dip (°Θ)	D (ft.)	TanΘ	UFS	DFS	
F3	4821.24	23	0	N15°W	51	3.9	1.2	15.6	28.6	1.6
F4	4820.32	92.5	0	N13°W	57	0.8	1.5	3.2	13.6	1.3

TRENCH 6, east to west.										
								Setback Distance (S), F=8		Safety Factor[4]
Fault	Elevation[1]	Station[2]	Width (ft)[3]	Trend	Dip (°Θ)	D (ft.)	TanΘ	UFS	DFS	
F9	4837.15	81.6	0	N33°W	75	8.5	3.7	34.0	38.3	0.5

TRENCH 7, west to east.										
								Setback Distance (S), F=8		Safety Factor[4]
Fault	Elevation[1]	Station[2]	Width (ft)[3]	Trend	Dip (°Θ)	D (ft.)	TanΘ	UFS	DFS	
F5	4816.65	30.8-32.0	0.5	N18°E - N27°E	77	1.2	4.3	4.8	9.0	0.5
F6	4816.47	37.6-39.1	1.2	N18°W - N10°W	66	0.6	2.2	2.4	10.7	0.9
F7	4814.01	56.6-62.3	4.6	N18°W	71	5.2	2.9	20.8	30.9	0.7

TRENCH 8, east to west.										
								Setback Distance (S), F=8		Safety Factor[4]
Fault	Elevation[1]	Station[2]	Width (ft)[3]	Trend	Dip (°Θ)	D (ft.)	TanΘ	UFS	DFS	
AF2	4835.04	11.4	0	N10°E	78	1.3	4.7	5.2	8.6	0.4
F9	4833.72	24.6-26.3	1.7	N48°W	57	8.5	1.5	34.0	46.1	1.3
F8	4838.44	139.3	0	N18°W	89	8.0	57.3	32.0	32.3	0.0

TRENCH 9, east to west.										
								Setback Distance (S), F=8		Safety Factor[4]
Fault	Elevation[1]	Station[2]	Width (ft)[3]	Trend	Dip (°Θ)	D (ft.)	TanΘ	UFS	DFS	
F10	4887.04	6.6	0	N35°W	75	0.7	3.7	2.8	7.1	0.5

[1] Surveyed elevation (in 2009 for trenches 1 through 8, in 2020 for trench 9) minus the distance from the ground surface to the highest fault point on the log.

[2] Distance in feet from 0 horizontal.

[3] Width of fault zone between correlative units, or highest fault points if no stratigraphic correlation.

[4] Setback adder per foot where footings are at depths exceeding 8 feet (see below).

Table 3B. *Recommended setbacks.*

		Setback distance in feet (F=8)			
Fault	**Dip**	**West**	**East**	**Safety Factor[4]**	**Notes**
F1	SW	43.6	34.0	1.2	*Based on T-3 above.*
F2	SW	46.1	34.0	1.5	*Based on T-3 above.*
F3	SW	28.6	20.0	1.6	*Based on T-4 above.*
F4	SW	20.0	20.0	1.3	*Based on T-4 above.*
F5	NW	20.0	20.0	0.5	*Based on T-7 above.*
F6	SW	20.0	20.0	0.9	*Based on T-7 above.*
F7	SW	30.9	20.8	0.7	*Based on T-7 above.*
F8	SW-NW	32.3	32.0	0.0	*Based on T-8 above.*
F9	SW	46.1	34.0	1.3	*Based on T-8 above.*
F10	SW	20.0	20.0	0.5	*Based on T-9 above.*
AF1	NE	20.0	20.0	1.9	*Based on T-3 above.*

The setback distances on Tables 3A-B and Figure 4 are calculated assuming an 8-foot footing depth from existing grade. However, the Project may require cuts to create level building pads that would have deeper footing depths than we assume. We therefore show a safety factor on Tables 3A and 3B that should be added to the calculated setback distance (S, Table 3A) per 1-foot difference between the surveyed fault elevation (or existing grade) and proposed grade elevation where the difference exceeds 6 feet (assuming footings are 2 feet below grade, for a total depth of 8 feet). The distance between the fault and nearest portion of the structure should be more than the sum. The minimum setback is 20 feet. This safety factor only applies to the downthrown fault sides. For upthrown fault sides, cuts would shift a fault and the corresponding UFS setback horizontally in the direction of dip, i.e. westward for west-dipping faults and eastward for east-dipping faults. The distance may be calculated as follows:

$$\mathbf{\Delta\ S = H/\tan\theta}$$

where:

ΔS = horizontal distance (shift in feet);

H = cut height (difference in feet between existing and proposed grade elevations); and

θ = Dip of the fault.

We recommend not modifying the defined setback areas on Figure 4 to avoid complexity and because development plans may change. Instead, the Project civil engineer should review the above on a case-by-case basis to ensure that structures are at a safe distance in areas where significant cuts are planned. This may be shown as a table on the grading plan. It is our understanding that minor adjustments will be made with regard to the condominium and Pad E structures on Figure 4. The most-recent grading plan should be submitted at the time our report and the geotechnical engineering report are submitted to Cottonwood Heights City. Though plans may change (and may differ from the base provided on Figure 4), CAD fault and setback delineations on Figure 4 have been confirmed to accurately coincide with those of the Project civil engineer.

5.3 Liquefaction and Lateral-Spread Ground Failure

Liquefaction occurs when saturated, loose, cohesionless, soils lose their support capabilities during a seismic event because of the development of excessive pore pressure. Earthquake-induced liquefaction can present a significant risk to structures from bearing-capacity failures to structural footings and foundations, and can damage structures and roadway embankments by triggering lateral spread landslides. Earthquakes of Richter magnitude 5 are generally regarded as the lower threshold for liquefaction. Liquefaction potential at the site is a combination of expected seismic accelerations (earthquake ground shaking), groundwater conditions, and presence of susceptible soils.

Given subsurface soil conditions observed in the trenches and Gordon Geotechnical borings, sandy soils possibly susceptible to liquefaction are present underlying the site. The site is also in an area subject to strong ground shaking, and areas west of boring B-2 have groundwater at a depth less than 30 feet. McCalpin (2002) notes that an event between 17,000 to 20,000 years ago on the Salt Lake City section of the WFZ (which he terms event S?) may have been responsible for a landslide into the lake, and most of the trenches at the site conducted for our investigation exposed evidence for a similar subaqueous failure that occurred during the Bonneville transgression. This landslide may be related to liquefaction lateral spreading, although this is unconfirmed.

Based on the above, we rate the existing risk from liquefaction as moderate. We conservatively recommend that the hazard from liquefaction be considered and discussed in the Project geotechnical engineering evaluation. Future liquefaction from a large-magnitude earthquake on the Salt Lake City section of the WFZ, if it occurs, could similarly manifest as lateral spreading given the site slopes.

5.4 Tectonic Deformation

Tectonic deformation refers to subsidence from warping, lowering, and tilting of a valley floor that accompanies surface-faulting earthquakes on normal faults. Large-scale tectonic subsidence may accompany earthquakes along large normal faults (Lund, 1990). Tectonic subsidence is believed to mainly impact those areas immediately adjacent to the downthrown side of active normal faults. The Project straddles a broad zone of faulting with multiple west-dipping main traces and at least two east-dipping antithetic traces. Backtilting was also observed in several of the trenches conducted for our investigation and is inferred on the cross sections shown on Figures 14 through 16.

Given the above, the Project is in an area at a high risk from tectonic deformation. Tectonic deformation is not typically a life-safety issue but can tilt building pads and alter sewer and water flow gradients, which may require expensive subsequent repairs. The owner and all future owners should understand and be willing to accept the risk. We recommend that the hazard from tectonic deformation be disclosed in all future real estate transactions.

5.5 Seismic Seiche and Storm Surge

Earthquake-induced seiche presents a risk to structures within the wave-oscillation zone along the edges of large bodies of water, such as the Great Salt Lake. Given the elevation of the subject property and distance from large bodies of water, we rate the risk from seismic seiches as low.

5.6 Stream Flooding

Stream flooding may be caused by direct precipitation, melting snow, or a combination of both. In much of Utah, floods are most common in April through June during spring snowmelt. High flows may be sustained from a few days to several weeks, and the potential for flooding depends on a variety of factors such as surface hydrology, site grading and drainage, and runoff. No active drainages were observed crossing the Project and Federal Emergency Management Agency flood insurance rate mapping (Map Number 49035C0318G, effective 09/25/2009) classifies the Project in "Zone X - Area of Minimal Flood Hazard". Given the above, we rate the risk from stream flooding as low. Care should be taken that proper surface drainage is maintained.

5.7 Shallow Groundwater

As discussed Section 4.4 above, groundwater deepens between trench T-2 and Gordon Geotechnical boring B-2 from 13 to 29 feet deep (Figure 4), and deepens to more than 100 feet between borings B-2 and B-3. Given this, the western half of the site has a moderate risk from shallow groundwater. Foundation and site subsurface drainage concerns should be considered and discussed in the Project geotechnical engineering evaluation. Care should be taken that proper subsurface drainage is maintained.

5.8 Landslides and Slope Failures

Slope stability hazards such as landslides, slumps, and other mass movements can develop along moderate to steep slopes where a slope has been disturbed, the head of a slope loaded, or where increased groundwater pore pressures result in driving forces within the slope exceeding restraining forces. Slopes exhibiting prior failures, and also deposits from large landslides, are particularly vulnerable to instability and reactivation.

No landslides are mapped or evident at the Project on Figure 2, but trench T-6 exposed evidence for a relict landslide that incorporated surficial debris and likely occurred prior to or contemporaneous with the Bonneville transgression. James Kirkland of the Utah

Geological Survey believed the bone fragments incorporated in the landslide (Figure 10A, stations 11-12 feet) belonged to a Pleistocene ungulate based on a brief, informal, visual examination in May 2009. This presumed age would match the stratigraphic provenance.

Given the above and the steep slopes at the site associated with prior gravel mining operations, as profiled on Figures 14 through 16, we rate the risk from landslides and slope instability as moderate. We conservatively recommend that slope stability be evaluated by the Project geotechnical engineer based on site-specific soil conditions and the data provided in this report. Recommendations should be provided to reduce the landslide hazard risk if factors of safety are determined to be unsuitable. Water, steep man-made cuts, and non-engineered fill materials are often major contributors to slope instability. Care should therefore also be taken to maintain proper site drainage, that site grading does not destabilize slopes at the site without prior geotechnical analysis and grading plans, and that water from man-made sources is minimized in potentially unstable slope areas.

5.9 Debris Flows

Debris flow hazards are typically associated with unconsolidated alluvial fan deposits at the mouths of large range-front drainages, such as those along the Wasatch Front. Debris flows have historically significant damage in the Wasatch Front area. The site is not in a mapped active alluvial fan, and no evidence for debris-flow channels, levees, or other debris-flow features was observed at the site on air photos or during our reconnaissance. Given the above, we rate the risk as low.

5.10 Rock Fall

No significant bedrock outcrops are at the site or in adjacent higher slopes that could present a source area for rock fall clasts, and no boulders likely from rock falls were observed at the site. Based on the above, we rate the hazard from rock falls as low.

5.11 Problem Soil and Rock

Surficial soils that contain certain clays can swell or collapse when wet. Soil conditions and specific recommendations for site grading, subgrade preparation, and footing and foundation design should be provided in the Project geotechnical engineering evaluation.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Earthquake ground shaking, surface fault rupture, and tectonic deformation are identified as posing a high relative risk to the proposed development. Liquefaction and lateral-spread ground failure, shallow groundwater, and landslides and slope failures are identified as posing a moderate risk. The following recommendations are provided with regard to the geologic characterizations in this report:

- ***Seismic Design* –** All habitable structures developed at the property should be constructed to current adopted seismic building codes to reduce the risk of damage, injury, or loss of life from earthquake ground shaking. The Project geotechnical engineer should confirm the ground-shaking hazard and provide appropriate seismic design parameters as needed. We note that earthquake ground shaking is a common hazard for all Wasatch Front areas and, although ground shaking and surface faulting are related earthquake hazards, they pose distinctly different risks.

- ***Geotechnical Evaluation* –** A design-level geotechnical engineering study should be conducted prior to construction to assess soil foundation conditions, assess the risk from shallow groundwater and liquefaction (and provide recommendations as needed), and evaluate slope stability. The stability evaluation should be based on geologic characterizations in this report and site-specific geotechnical data, and provide recommendations for reducing the risk of landsliding if the factors of safety are deemed unsuitable.

- ***Site Modifications and Drainage* –** No unplanned cuts should be made in the slopes at the site without prior geotechnical analyses, and proper surface and subsurface drainage should be maintained.

- ***Surface Fault Rupture Hazards* –** No structures intended for human occupancy should be located in the setback zones shaded in light red on Figure 4. It is generally accepted practice to allow streets, driveways, yards, and other non-occupied, non-attached structures to be constructed within these areas. No habitable structures should also be located in the unexplored area shaded in light green on Figure 4 without additional subsurface exploration to evaluate if active faults are present. The structure on Pad E, which overlies two small displacement faults observed in trench T-1 (Figures 4 and 5A), should be designed to withstand up to 0.3 feet of vertical offset. Utility lines that cross faults should also be engineered to withstand expected displacements and/or have design features to ensure life safety.

- ***Grading and Development Plan Review* –** Significant cuts could change fault locations and setback zone calculations. A safety factor and an upthrown fault side modifier are therefore provided in Section 5.2 to assist review of the grading and development plan by the Project civil engineer in areas where there such cuts may be planned. Care should be taken in these areas to ensure that proposed structures remain at a safe distance. Results of this review may be shown as a table on the grading plan. The most-recent grading plan should be included with our report and the geotechnical engineering report when the reports are submitted to Cottonwood Heights City. We have confirmed our fault and setback delineations on Figure 4 accurately coincide with CAD data of the Project civil engineer.

- ***Excavation Backfill Considerations*** **–** The trenches may be in areas where a structure could subsequently be placed. However, backfill may not have been replaced in the excavations in compacted layers. The fill could settle with time and upon saturation. Should structures be located in an excavated area, no footings or structure should be founded over the excavation unless the backfill has been removed and replaced with structural fill.

- ***Excavation Inspection*** **–** This report does not reflect subsurface variations that may occur laterally away from an exploration trench. Such variations may occur that could become evident during construction. Thus, it is important that we observe subsurface materials exposed in future excavations to take advantage of opportunities to recognize differing conditions that could affect the performance of a planned structure.

- ***Hazard Disclosures and Report Availability*** **–** All hazards identified as posing a high risk at the site should be disclosed to future buyers so that they may understand and be willing to accept any potential developmental challenges and/or risks posed by these hazards. This report should be made available to architects, building contractors, and in the event of a future property sale, real estate agents and potential buyers. The report should be referenced for information on technical data only as interpreted from observations and not as a warranty of conditions throughout the site. The report should be submitted in its entirety, or referenced appropriately, as part of any document submittal to a government agency responsible for planning decisions or geologic review. Incomplete submittals void the professional seals and signatures we provide herein. Although this report and the data herein are the property of the client, the report format is the intellectual property of Western Geologic and should not be copied, used, or modified without express permission of the authors.

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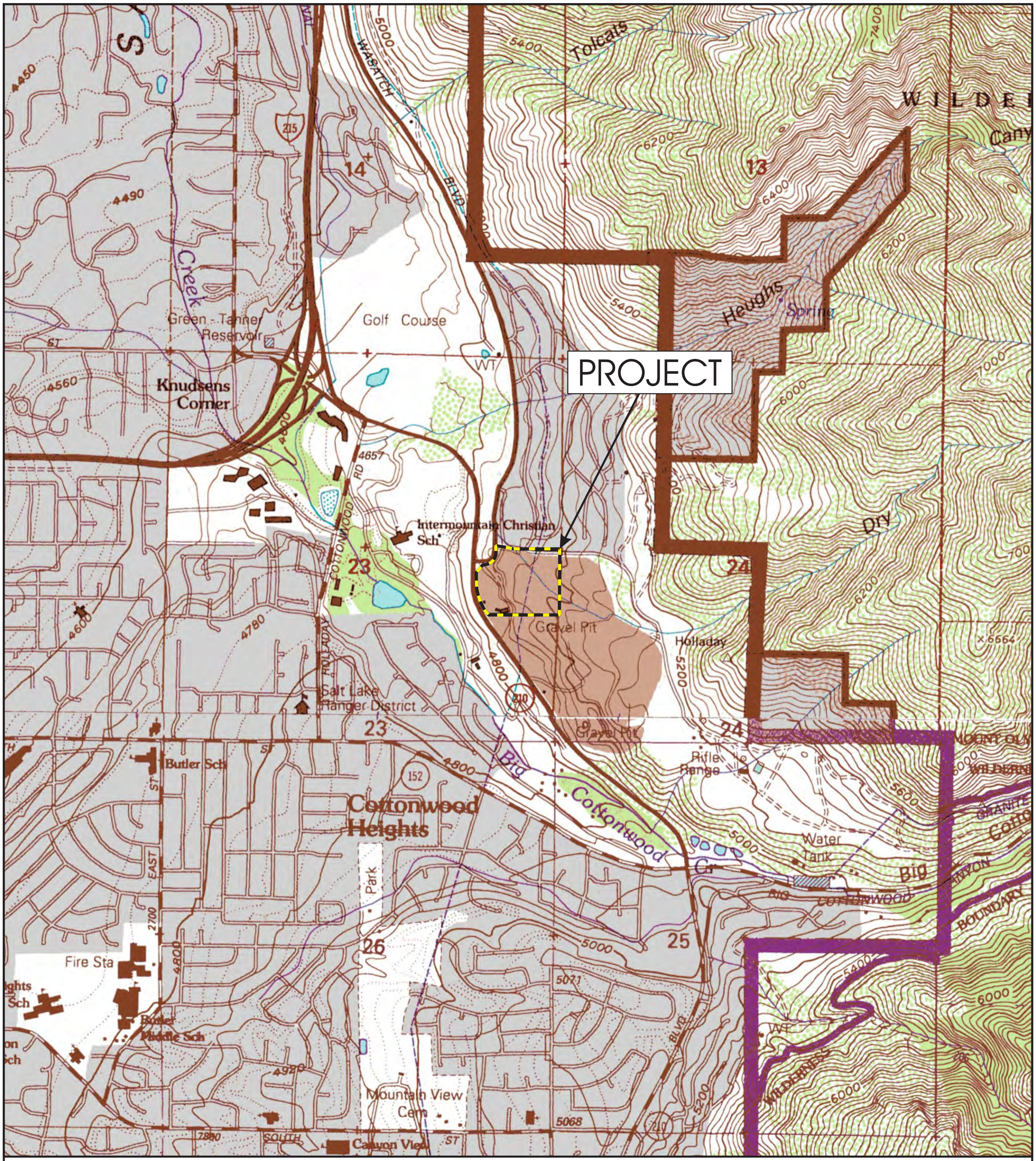
FIGURES



Source: U.S. Geological Survey 7.5 Minute Series Topographic Maps, Utah - Sugar House and Draper, 1998.

LOCATION MAP

GEOLOGIC HAZARDS EVALUATION

AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

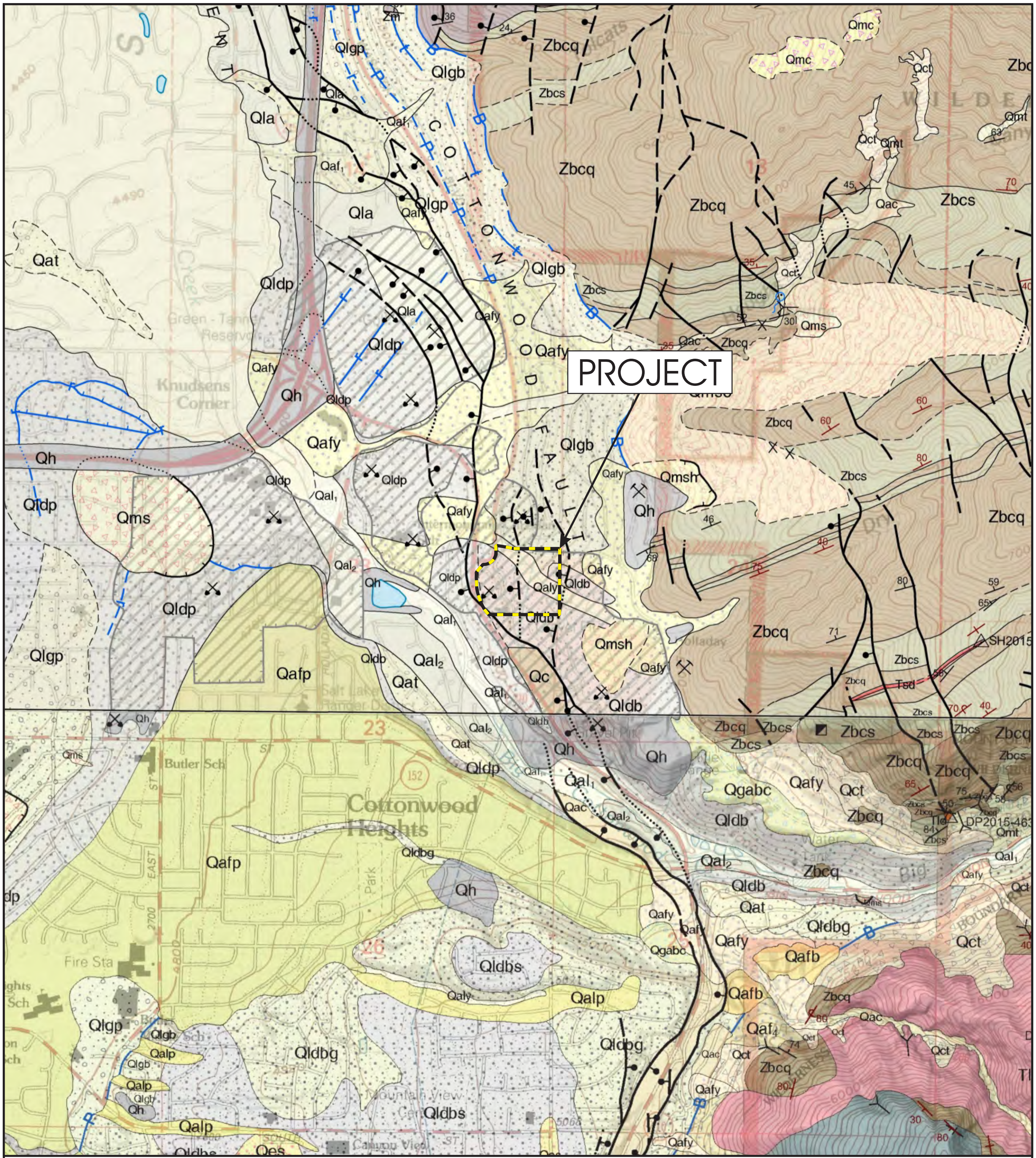
FIGURE 1

WESTERN GEOLOGIC

N
W E
S

0 1000 2000 feet

Scale 1:24,000
(1 inch = 2000 feet)



Source: McKean (2018; Sugarhouse Quadrangle) and McKean and Solomon (2018; Draper Quadrangle).
See text for description of surficial geologic units at Project.

GEOLOGIC MAP

GEOLOGIC HAZARDS EVALUATION

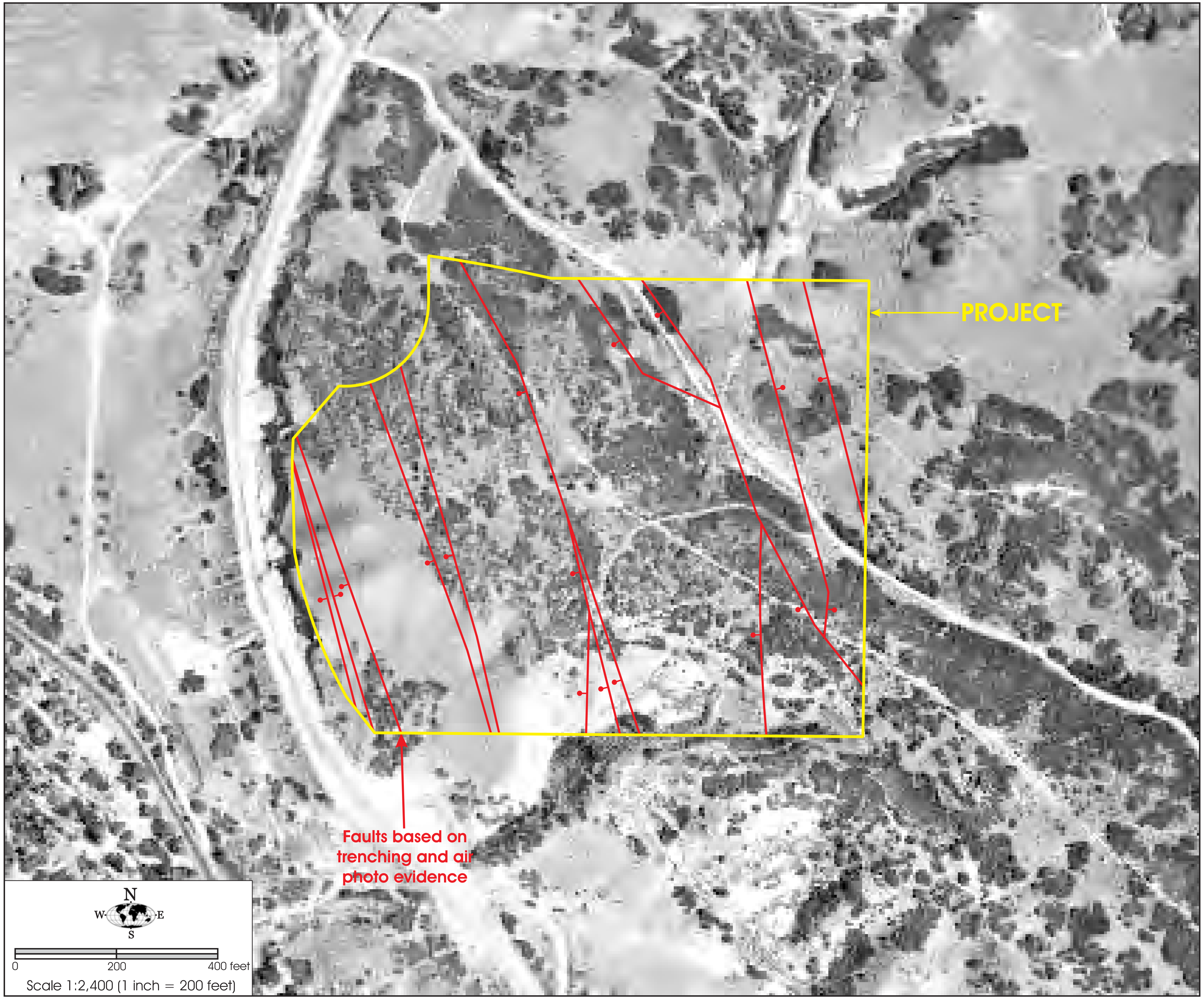
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 2

WESTERN GEOLOGIC

Scale 1:24,000
(1 inch = 2000 feet)

WESTERN GEOLOGIC



Air Photo Source: 1938 U.S. Bureau of Reclamation Salt Lake Aqueduct historical photography, reproduced and georeferenced in Bowman and Beisner (2008), frame sla1-20, approximate original scale 1:20,000.

1938 AIR PHOTO

GEOLOGIC HAZARDS EVALUATION

AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 3A

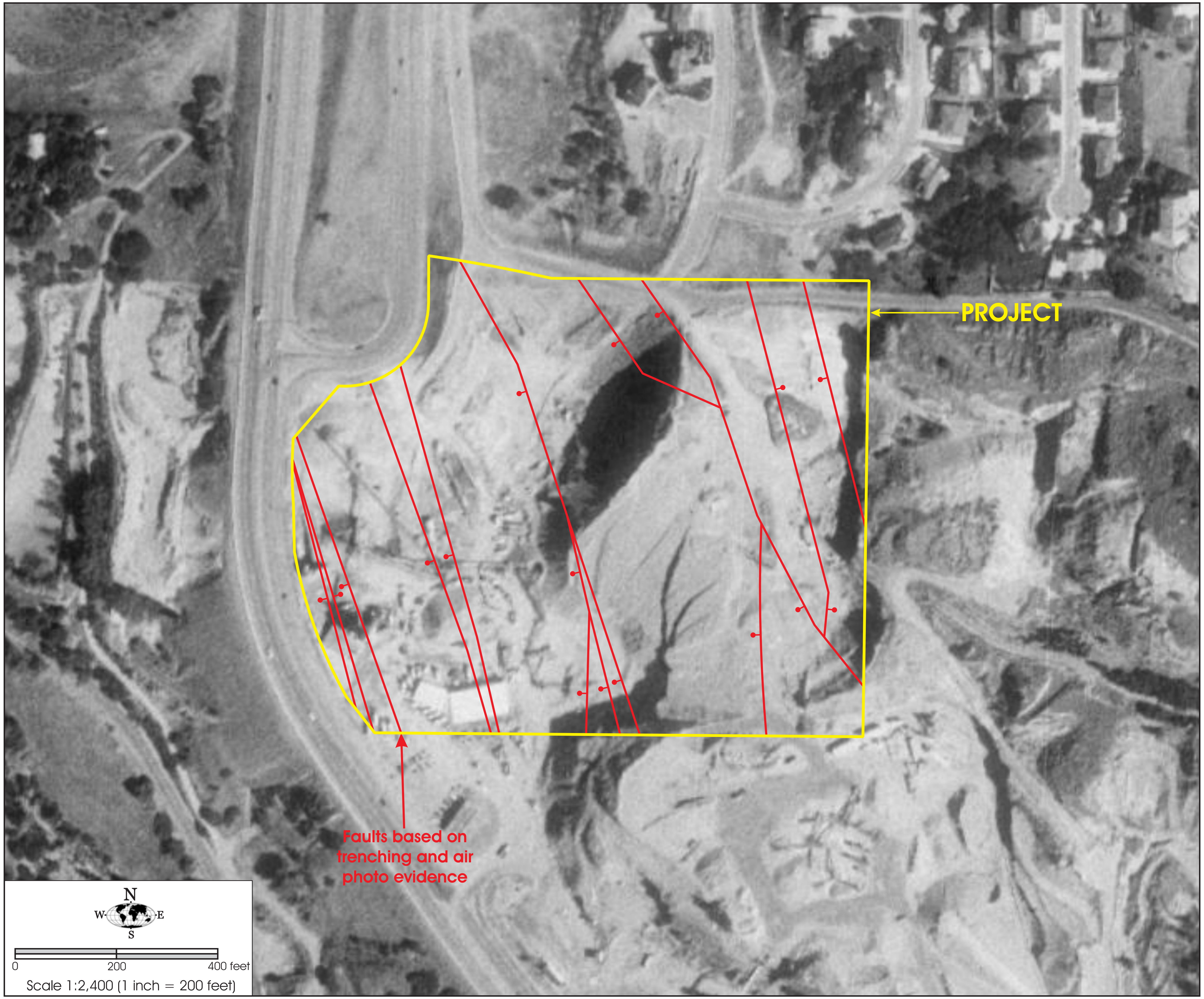
WESTERN GEOLOGIC

Air Photo Source: Utah AGRC, 1977 Digital Orthophoto Quadrangle, frame q1320-1977, 1 meter resolution.

1977 AIR PHOTO

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 3B



WESTERN GEOLOGIC

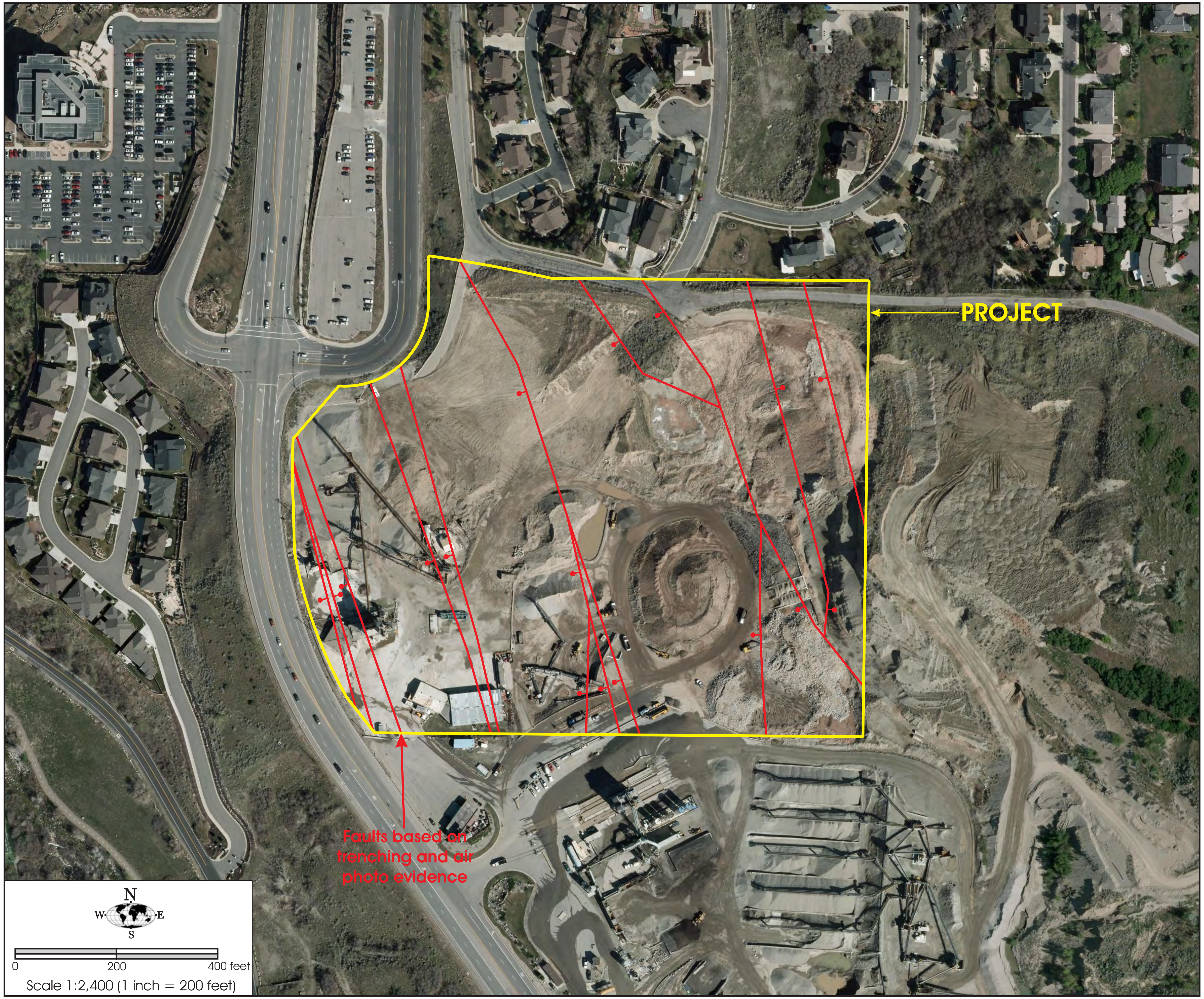
Air Photo Source: Utah AGRC, 1990s Digital Orthophoto Quadrangle, frame q1320-83, 1 meter resolution.

1993 AIR PHOTO

GEOLOGIC HAZARDS EVALUATION

AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 3C



Air Photo Source: Utah AGRC, 2012 High Resolution Orthophoto, frames 12TVK320960 and 12TVK320980, 12.5 centimeter resolution.

2012 AIR PHOTO

GEOLOGIC HAZARDS EVALUATION

AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 3D

WESTERN GEOLOGIC

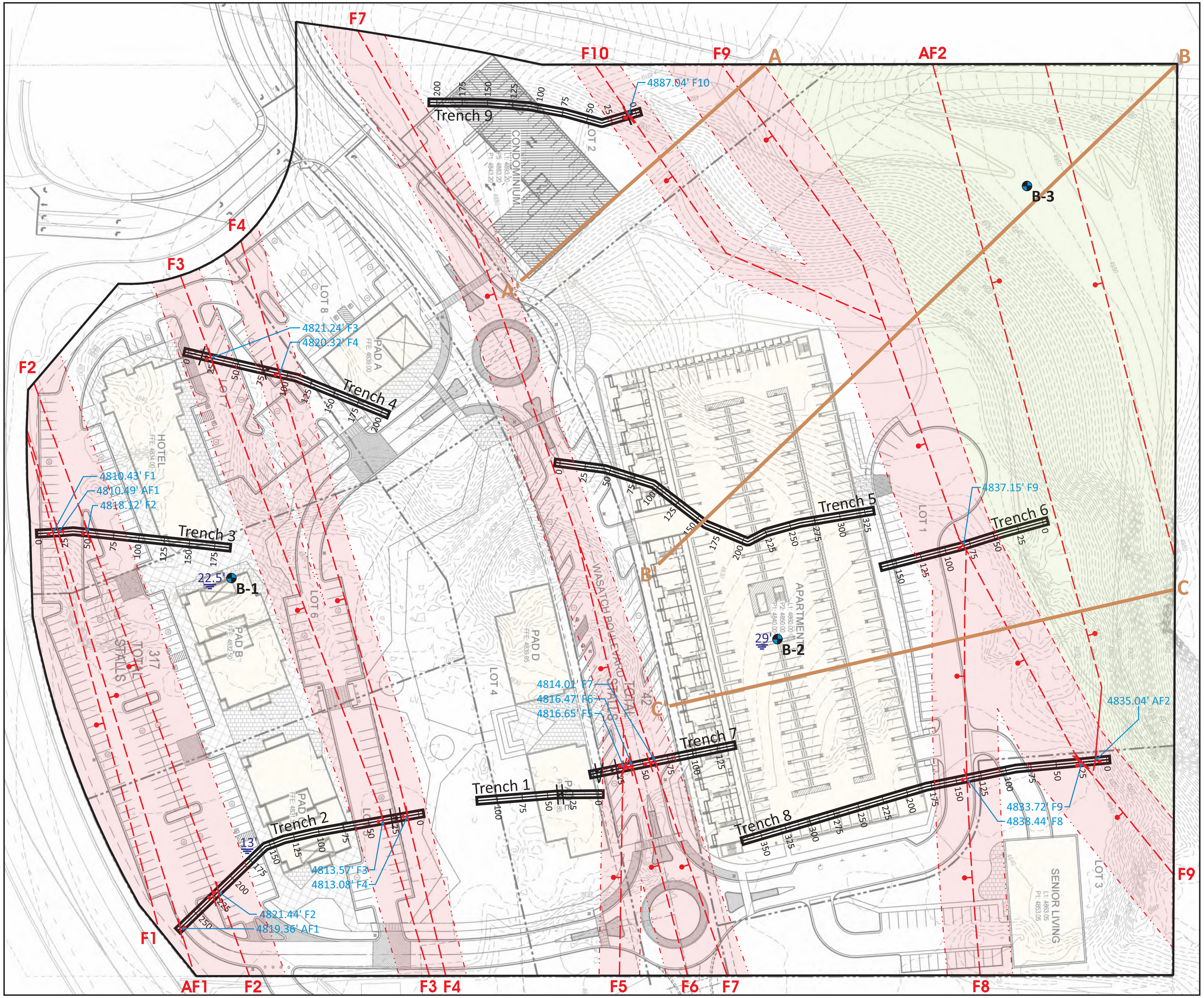
Air Photo Source: Utah AGRC, 2013 LIDAR Bare Earth DEM, frames BH12TVK3200096000 and BH12TVK3200098000, 50 centimeter resolution.

2013 LIDAR IMAGE

GEOLOGIC HAZARDS EVALUATION

AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 3E



WESTERN GEOLOGIC

EXPLANATION

Faults based on trenching and air photo evidence; bar and ball on downthrown side.

0 25 50 Trench for this study, stations denote distance in feet on logged wall; blue tags denote elevation of highest fault point in trench at time of surveying.

🟥 Setback zones (see text for explanation).

🟩 Unexplored area, no structures intended for human occupation should be placed in this area without additional exploration.

A' A Cross section location (Figures 14 to 16)

● B-2 Gordon Geotechnical boring

0 100 200 feet
Scale 1 inch equals 100 feet

N
W E
S

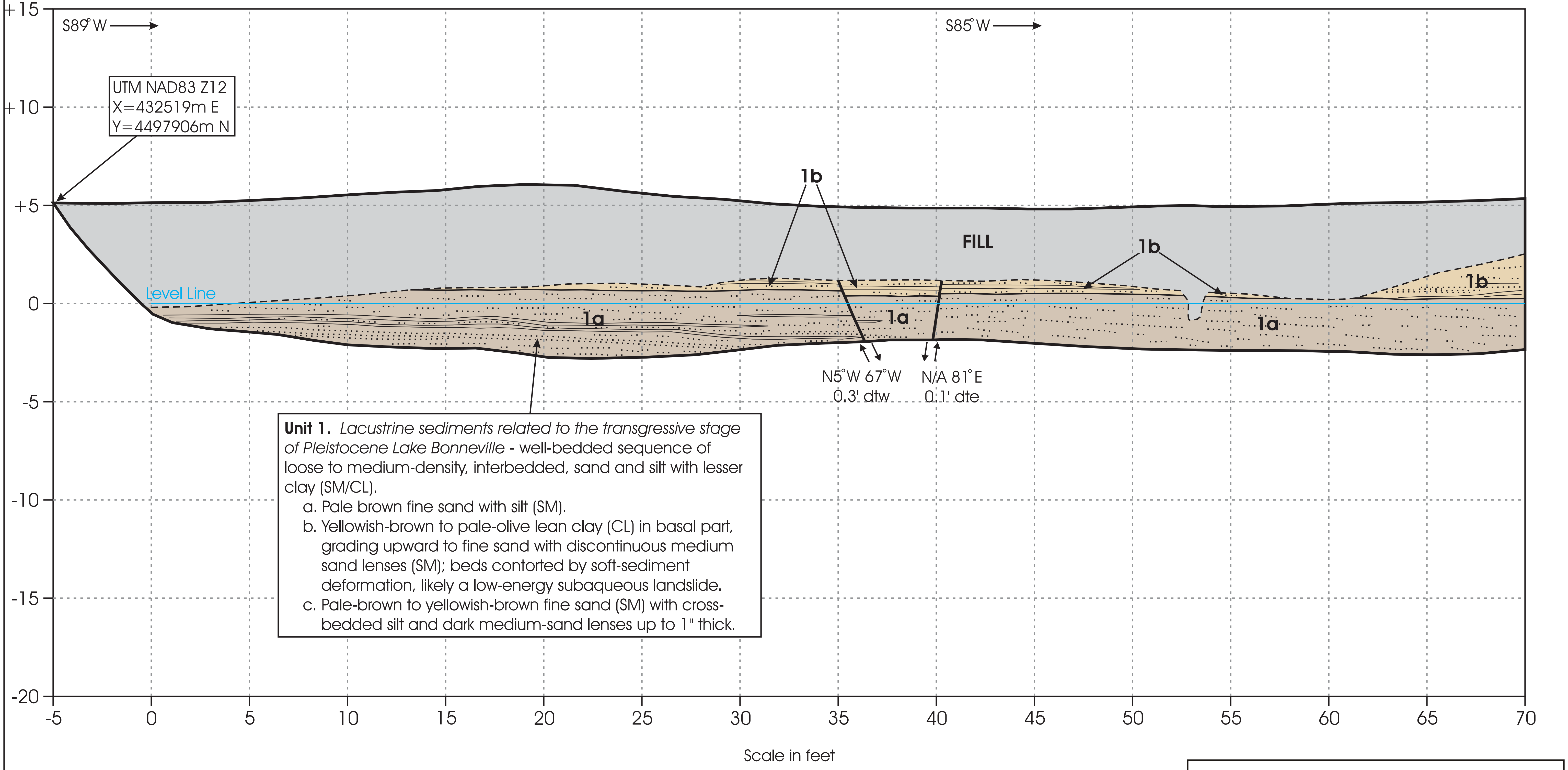
Base Map: McNeil Engineering concept grading plan SK-02 dated April 24, 2020.

SITE PLAN

GEOLOGIC HAZARDS EVALUATION

AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 4



Unit 1. *Lacustrine sediments related to the transgressive stage of Pleistocene Lake Bonneville* - well-bedded sequence of loose to medium-density, interbedded, sand and silt with lesser clay (SM/CL).

a. Pale brown fine sand with silt (SM).
b. Yellowish-brown to pale-olive lean clay (CL) in basal part, grading upward to fine sand with discontinuous medium sand lenses (SM); beds contorted by soft-sediment deformation, likely a low-energy subaqueous landslide.
c. Pale-brown to yellowish-brown fine sand (SM) with cross-bedded silt and dark medium-sand lenses up to 1" thick.

WESTERN GEOLOGIC

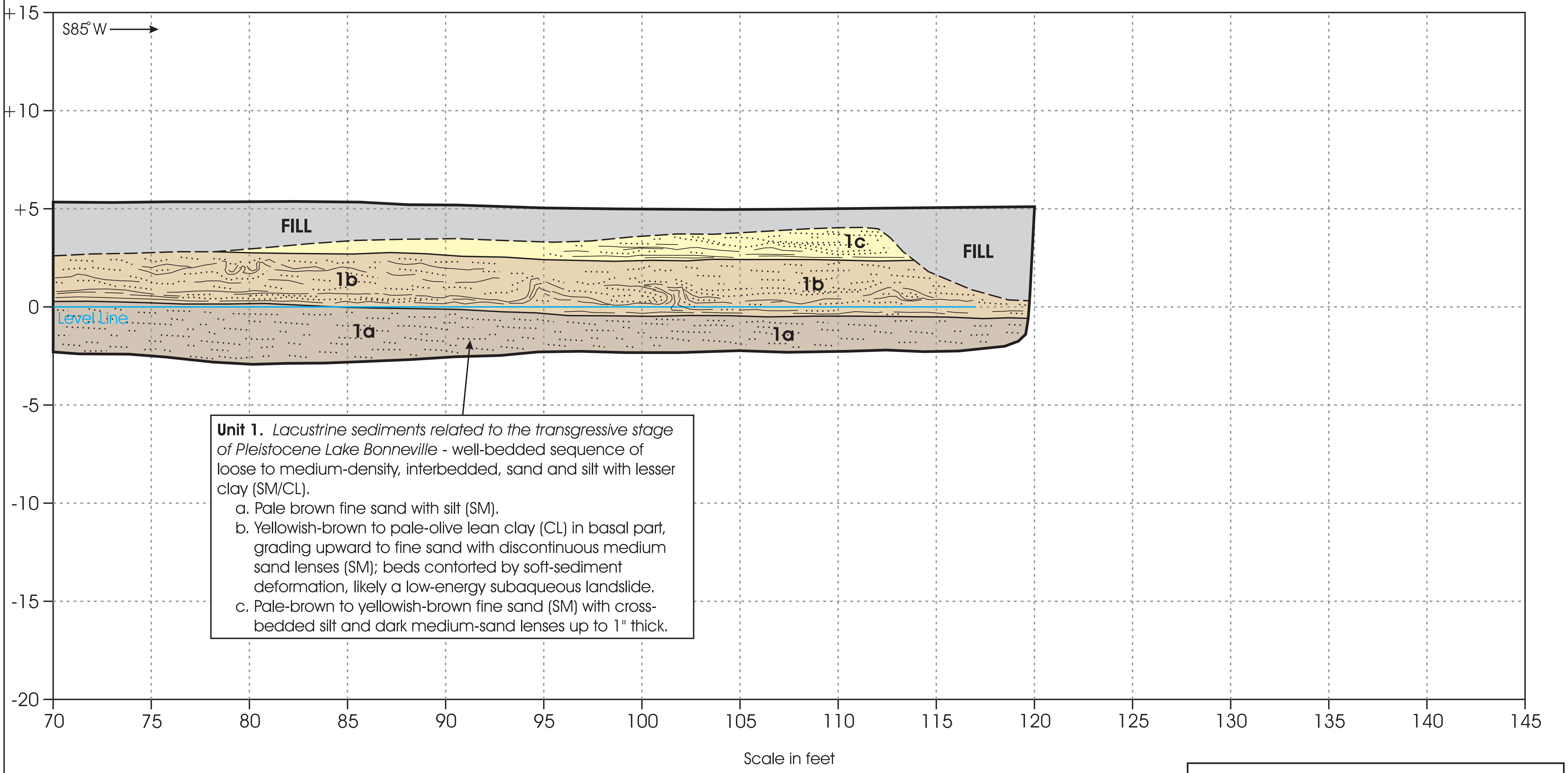
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

Trench logged by Bill Black, P.G. on
April 25, 2009

TRENCH 1 LOG, SHEET 1

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 5A



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- c. Pale-brown to yellowish-brown fine sand (SM) with cross-bedded silt and dark medium-sand lenses up to 1" thick.

WESTERN
GEOLOGIC

SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

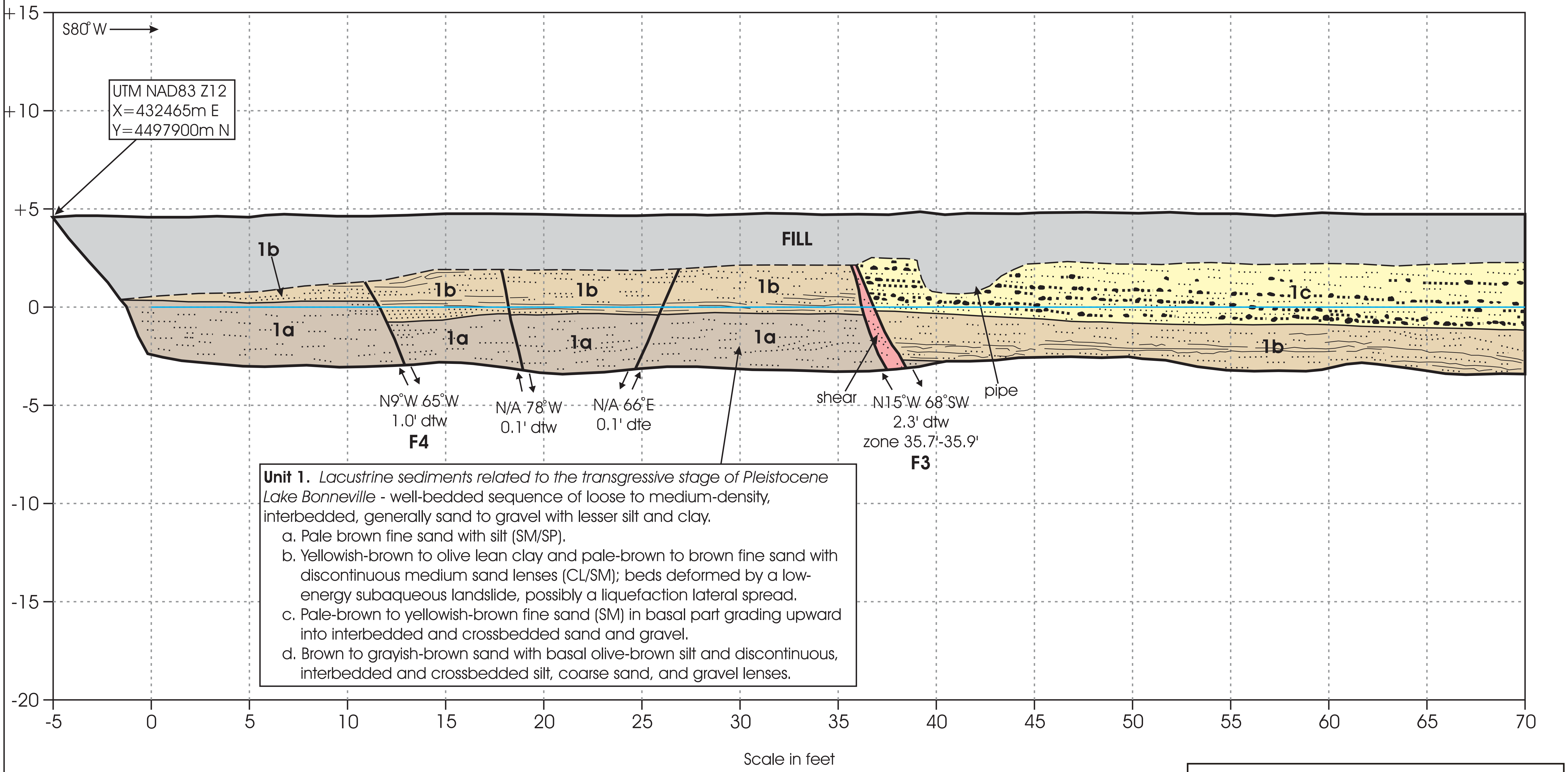
Trench logged by Bill Black, P.G. on
April 25, 2009

TRENCH 1 LOG, SHEET 2

GEOLOGIC HAZARDS EVALUATION

AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 5B



Unit 1. *Lacustrine sediments related to the transgressive stage of Pleistocene Lake Bonneville* - well-bedded sequence of loose to medium-density, interbedded, generally sand to gravel with lesser silt and clay.

- a. Pale brown fine sand with silt (SM/SP).
- b. Yellowish-brown to olive lean clay and pale-brown to brown fine sand with discontinuous medium sand lenses (CL/SM); beds deformed by a low-energy subaqueous landslide, possibly a liquefaction lateral spread.
- c. Pale-brown to yellowish-brown fine sand (SM) in basal part grading upward into interbedded and crossbedded sand and gravel.
- d. Brown to grayish-brown sand with basal olive-brown silt and discontinuous, interbedded and crossbedded silt, coarse sand, and gravel lenses.

WESTERN GEOLOGIC

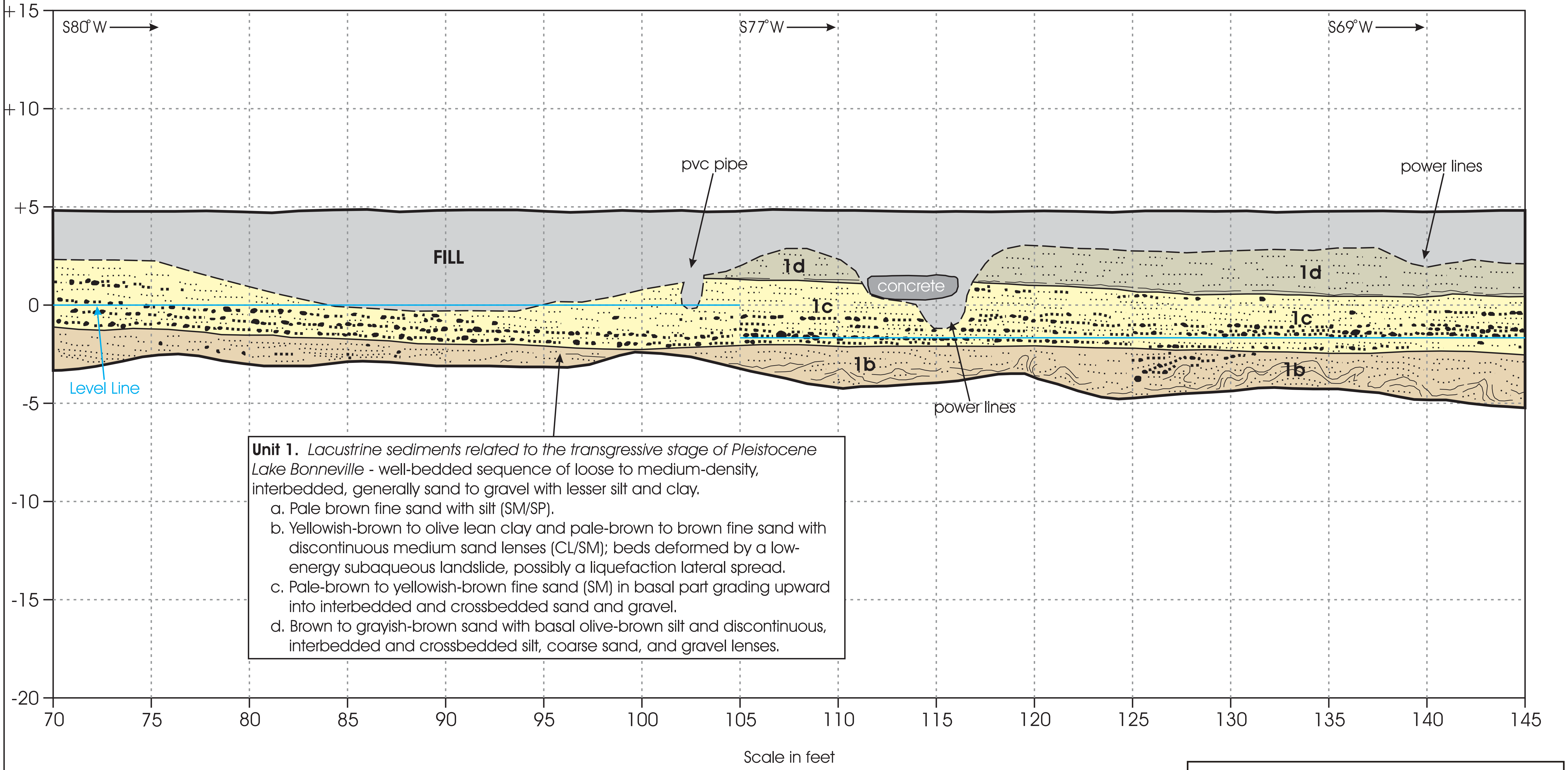
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

Trench logged by Bill Black, P.G. on
April 25-26 and May 9, 2009

TRENCH 2 LOG, SHEET 1

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 6A



Unit 1. *Lacustrine sediments related to the transgressive stage of Pleistocene Lake Bonneville* - well-bedded sequence of loose to medium-density, interbedded, generally sand to gravel with lesser silt and clay.

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b. Yellowish-brown to olive lean clay and pale-brown to brown fine sand with discontinuous medium sand lenses (CL/SM); beds deformed by a low-energy subaqueous landslide, possibly a liquefaction lateral spread.

c. Pale-brown to yellowish-brown fine sand (SM) in basal part grading upward into interbedded and crossbedded sand and gravel.

d. Brown to grayish-brown sand with basal olive-brown silt and discontinuous, interbedded and crossbedded silt, coarse sand, and gravel lenses.

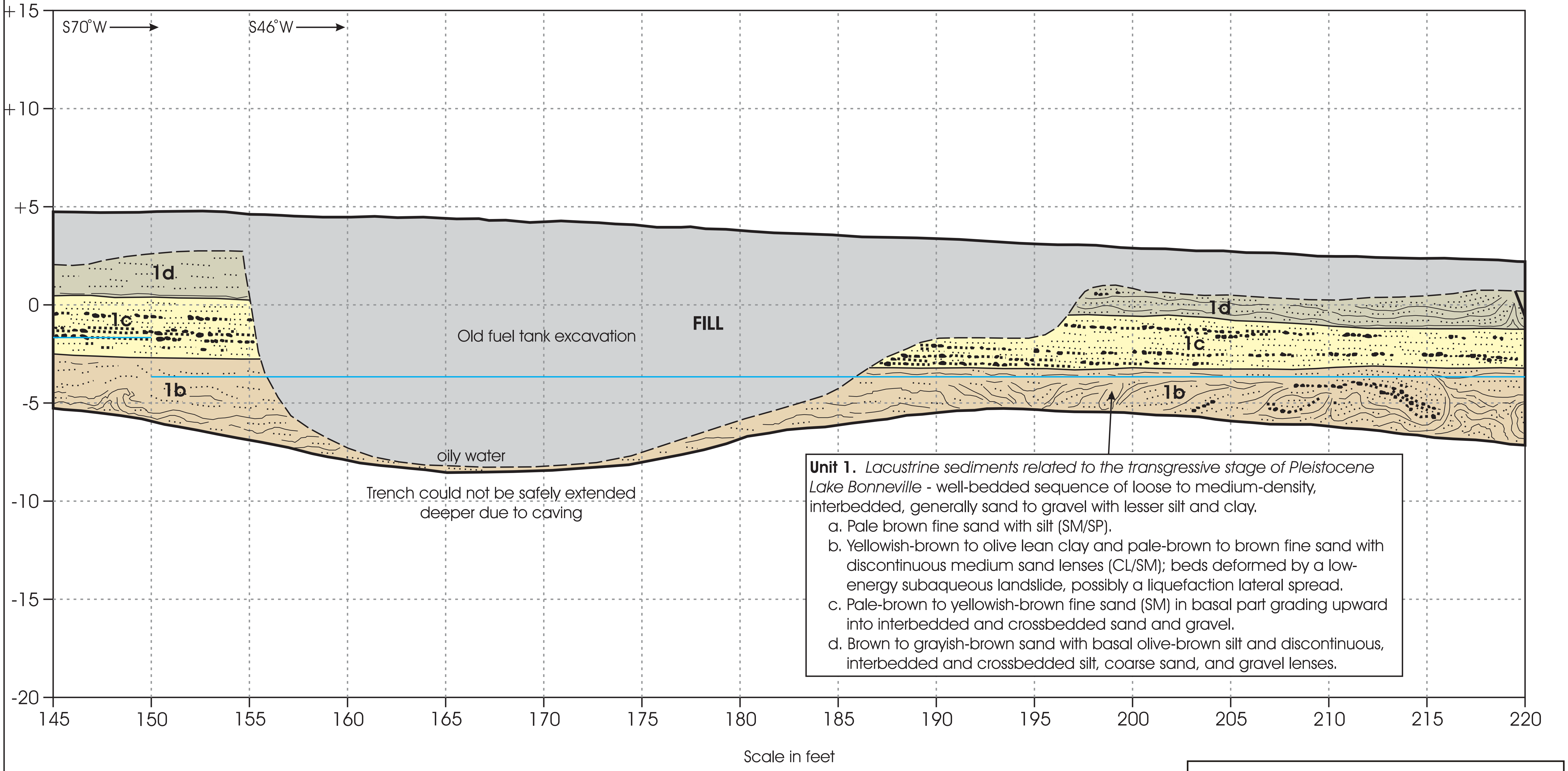
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

Trench logged by Bill Black, P.G. on
April 25-26 and May 9, 2009

TRENCH 2 LOG, SHEET 2

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 6B



Unit 1. *Lacustrine sediments related to the transgressive stage of Pleistocene Lake Bonneville* - well-bedded sequence of loose to medium-density, interbedded, generally sand to gravel with lesser silt and clay.

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- c. Pale-brown to yellowish-brown fine sand (SM) in basal part grading upward into interbedded and crossbedded sand and gravel.
- d. Brown to grayish-brown sand with basal olive-brown silt and discontinuous, interbedded and crossbedded silt, coarse sand, and gravel lenses.

SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

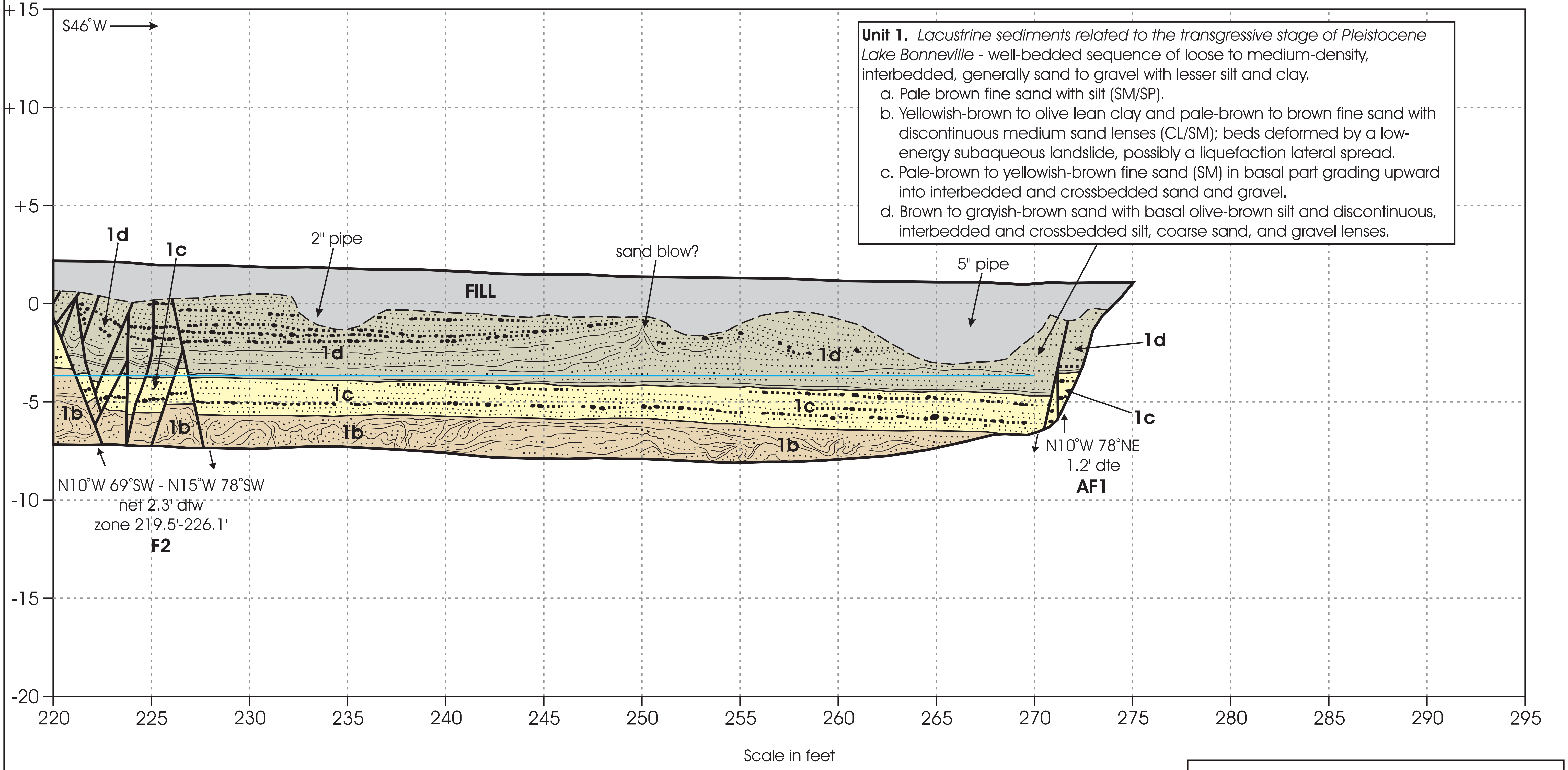
Trench logged by Bill Black, P.G. on
April 25-26 and May 9, 2009

WESTERN
GeoLogic

TRENCH 2 LOG, SHEET 3

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 6C



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- c. Pale-brown to yellowish-brown fine sand (SM) in basal part grading upward into interbedded and crossbedded sand and gravel.
- d. Brown to grayish-brown sand with basal olive-brown silt and discontinuous, interbedded and crossbedded silt, coarse sand, and gravel lenses.

SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

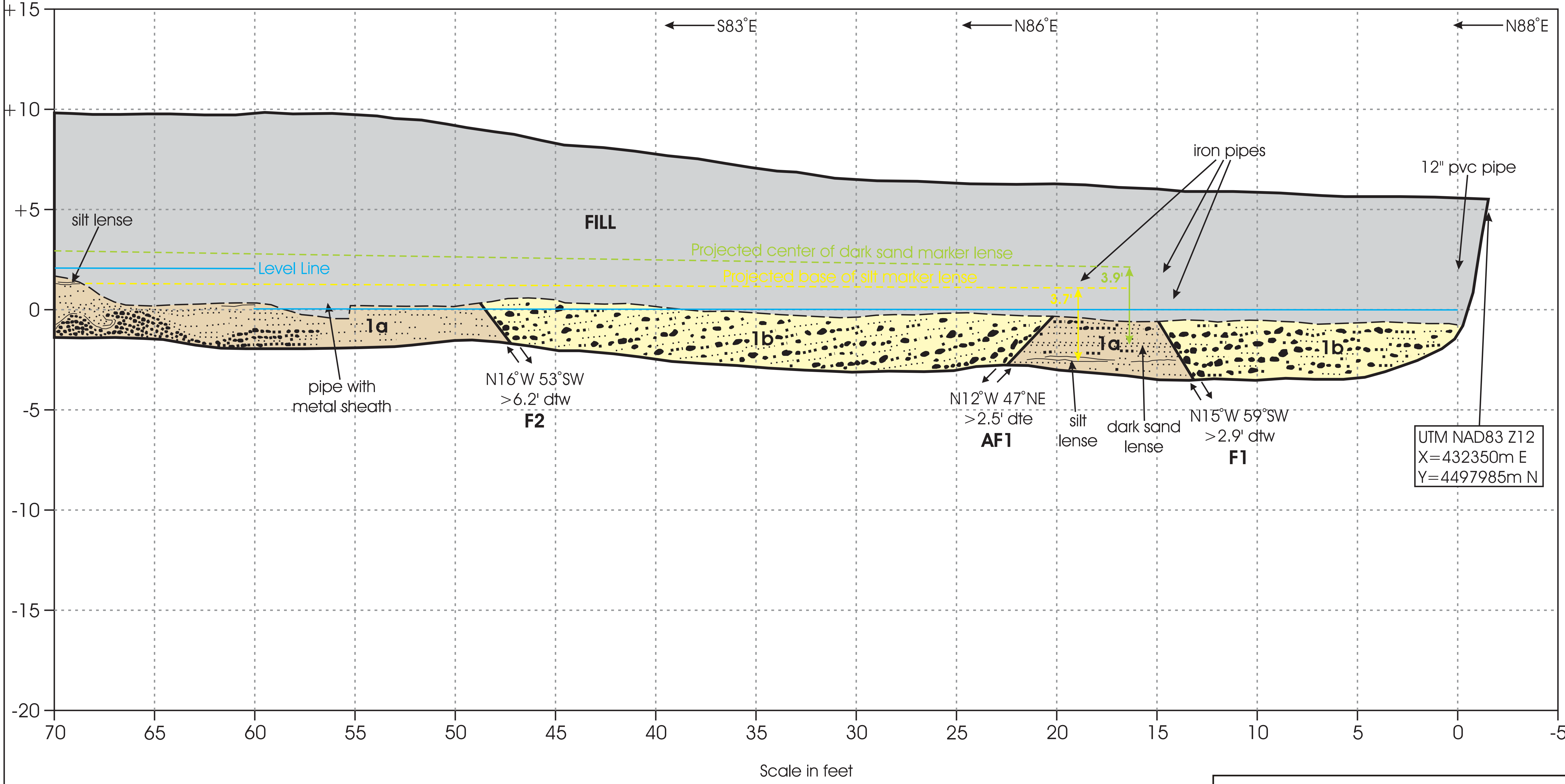
Trench logged by Bill Black, P.G. on
April 25-26 and May 9, 2009

TRENCH 2 LOG, SHEET 4

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 6D

WESTERN GEOLOGIC



SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

Trench logged by Bill Black, P.G. on
May 2, 2009

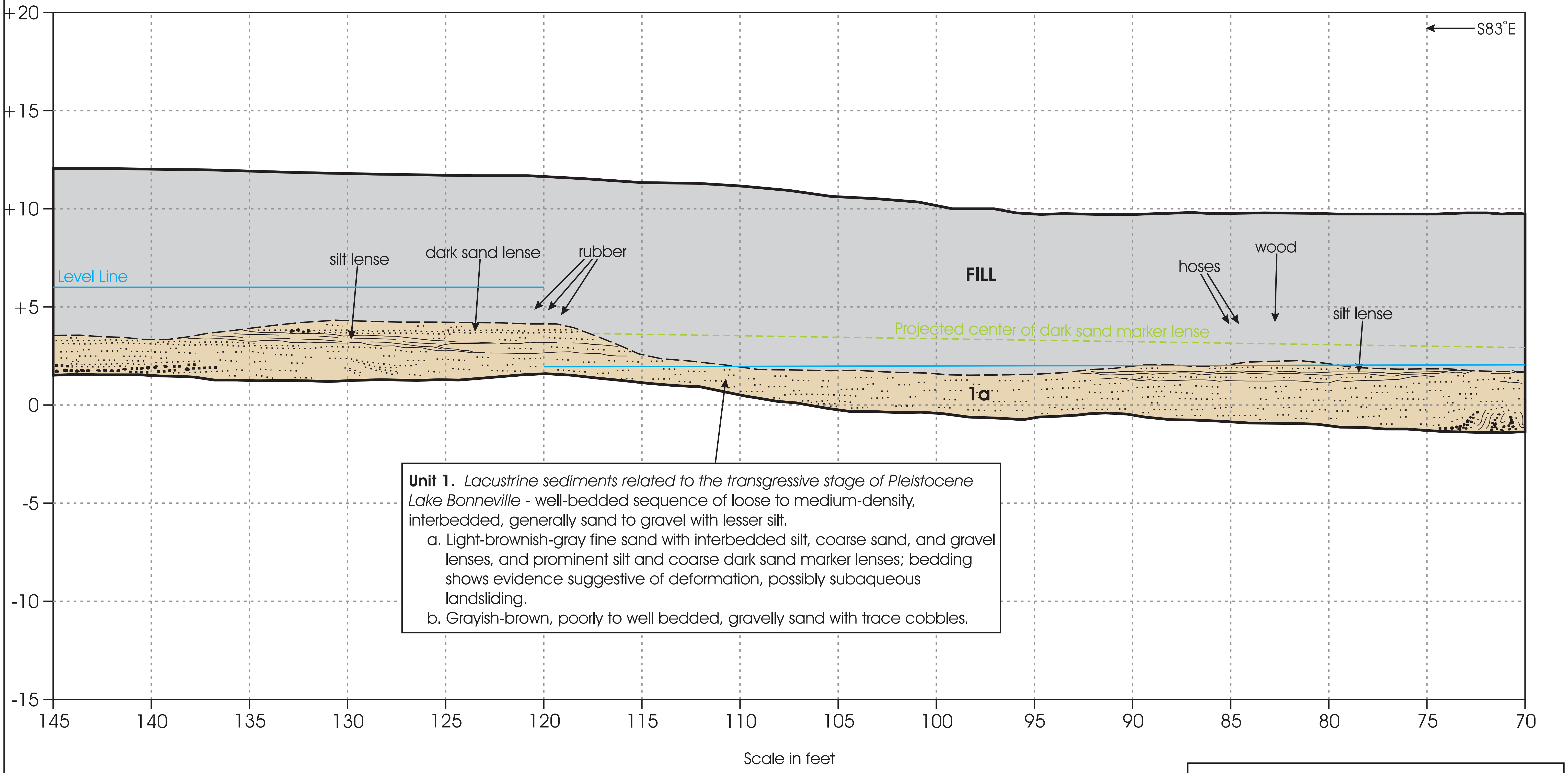
WESTERN GEOLOGIC

TRENCH 3 LOG, SHEET 1

GEOLOGIC HAZARDS EVALUATION

AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 7A



Unit 1. *Lacustrine sediments related to the transgressive stage of Pleistocene Lake Bonneville* - well-bedded sequence of loose to medium-density, interbedded, generally sand to gravel with lesser silt.

a. Light-brownish-gray fine sand with interbedded silt, coarse sand, and gravel lenses, and prominent silt and coarse dark sand marker lenses; bedding shows evidence suggestive of deformation, possibly subaqueous landsliding.

b. Grayish-brown, poorly to well bedded, gravelly sand with trace cobbles.

WESTERN GEOLOGIC

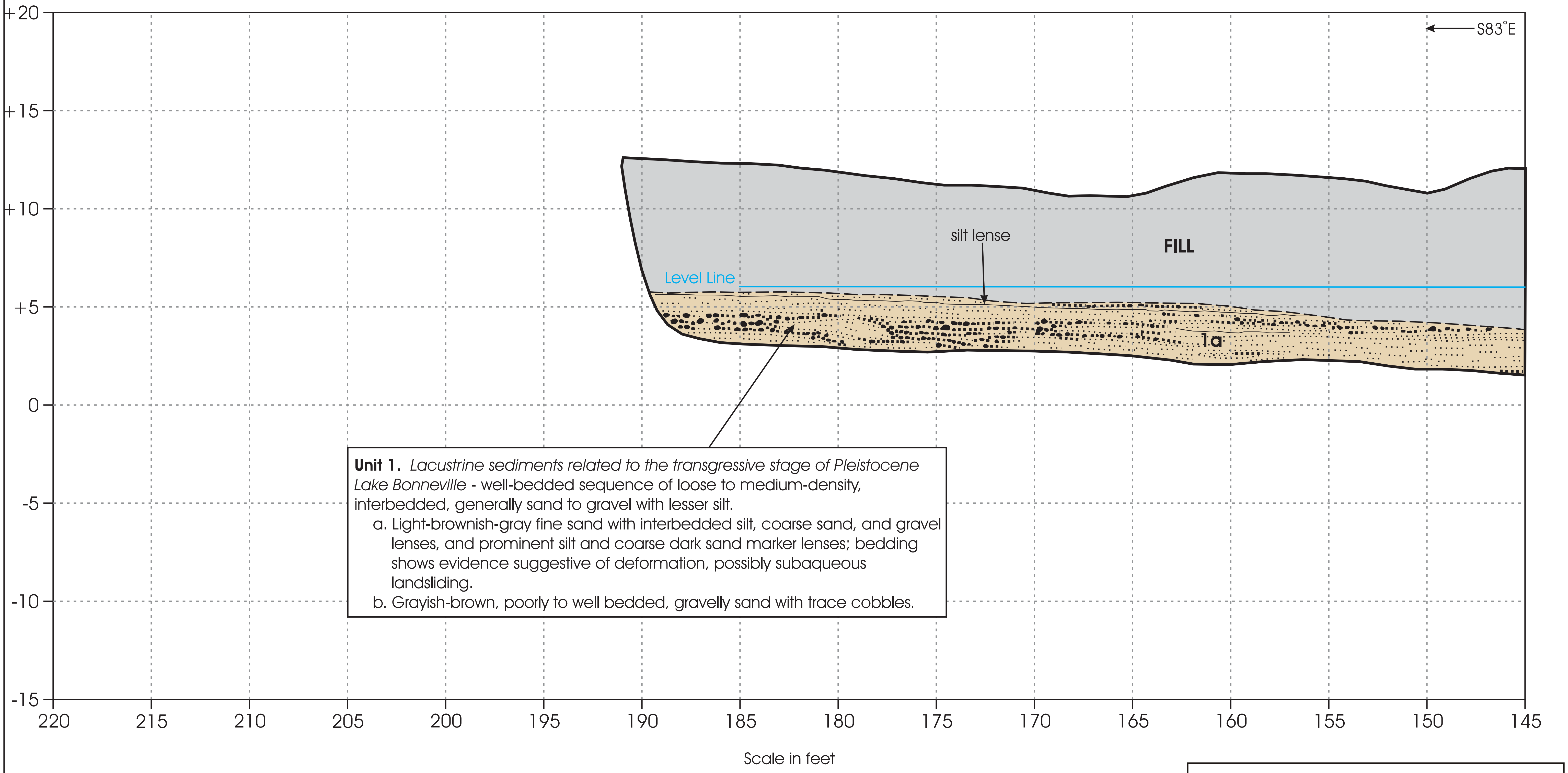
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

Trench logged by Bill Black, P.G. on
May 2, 2009

TRENCH 3 LOG, SHEET 2

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 7B



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b. Grayish-brown, poorly to well bedded, gravelly sand with trace cobbles.

SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

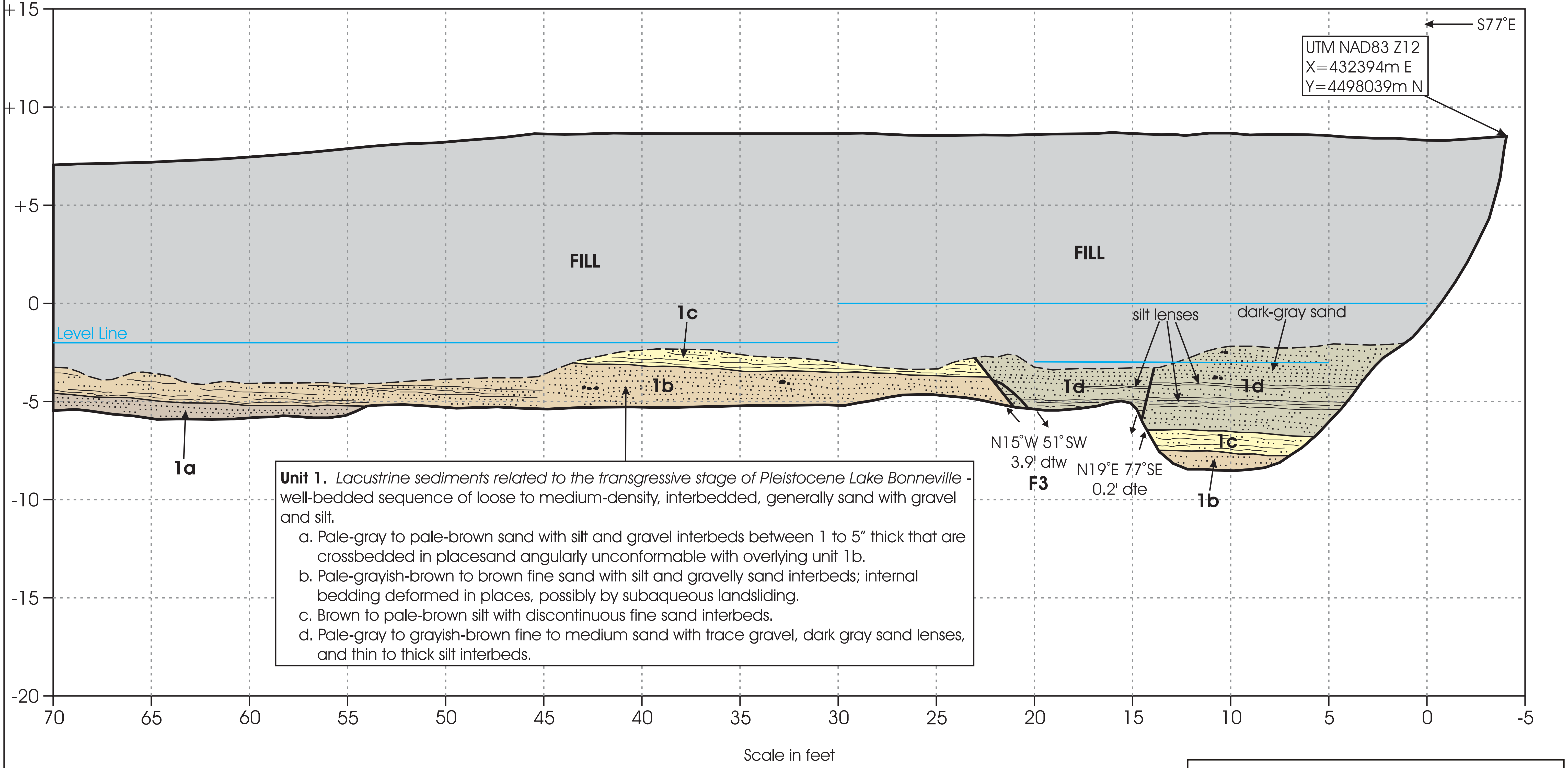
Trench logged by Bill Black, P.G. on
May 2, 2009

WESTERN GEOLOGIC

TRENCH 3 LOG, SHEET 3

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 7C



Unit 1. *Lacustrine sediments related to the transgressive stage of Pleistocene Lake Bonneville* - well-bedded sequence of loose to medium-density, interbedded, generally sand with gravel and silt.

a. Pale-gray to pale-brown sand with silt and gravel interbeds between 1 to 5" thick that are crossbedded in placesand angularly unconformable with overlying unit 1b.
b. Pale-grayish-brown to brown fine sand with silt and gravelly sand interbeds; internal bedding deformed in places, possibly by subaqueous landsliding.
c. Brown to pale-brown silt with discontinuous fine sand interbeds.
d. Pale-gray to grayish-brown fine to medium sand with trace gravel, dark gray sand lenses, and thin to thick silt interbeds.

SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

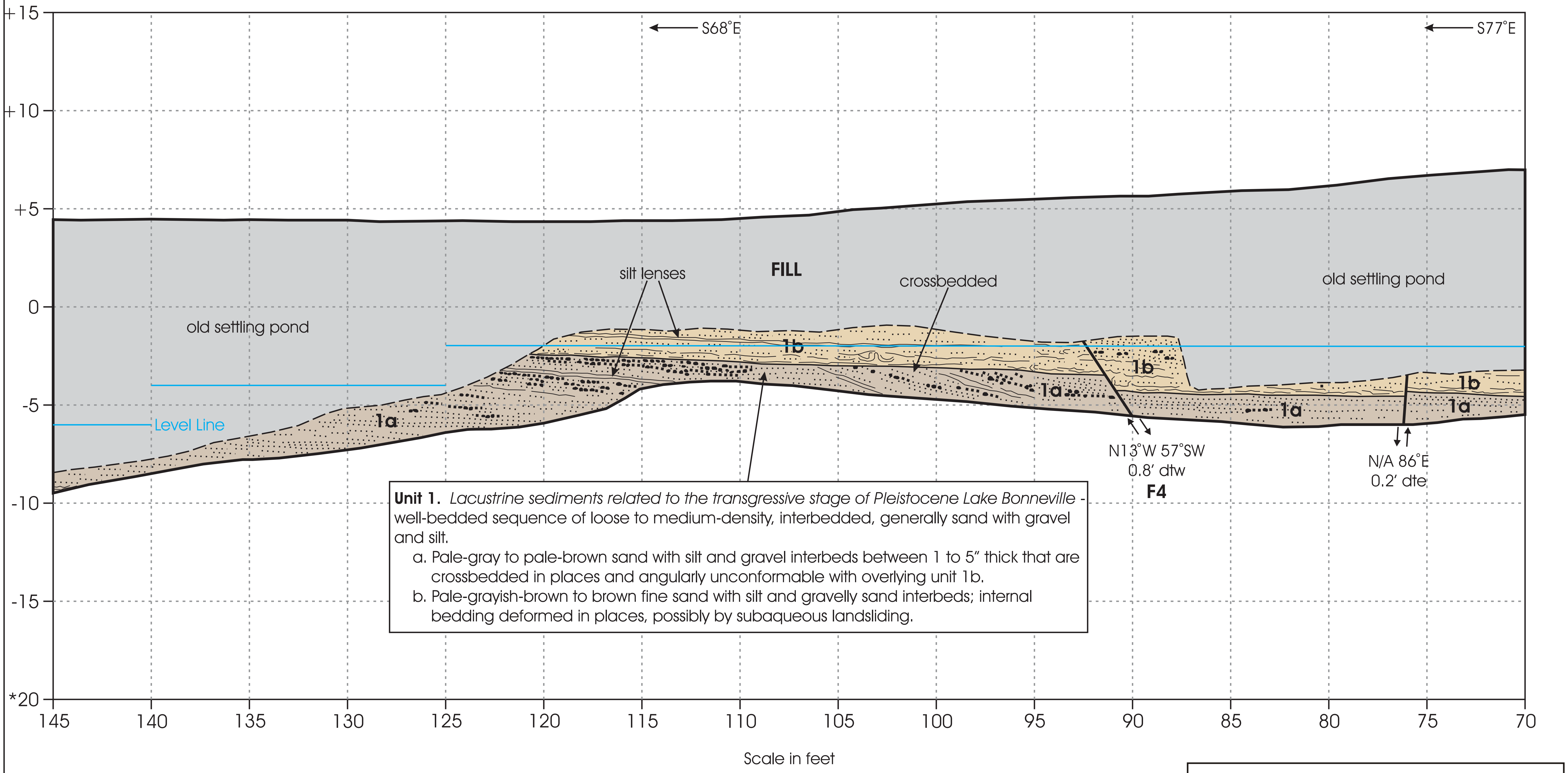
Trench logged by Bill Black, P.G. on
May 3 and 16, 2009

WESTERN GEOLOGIC

TRENCH 4 LOG, SHEET 1

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 8A



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b. Pale-grayish-brown to brown fine sand with silt and gravelly sand interbeds; internal bedding deformed in places, possibly by subaqueous landsliding.

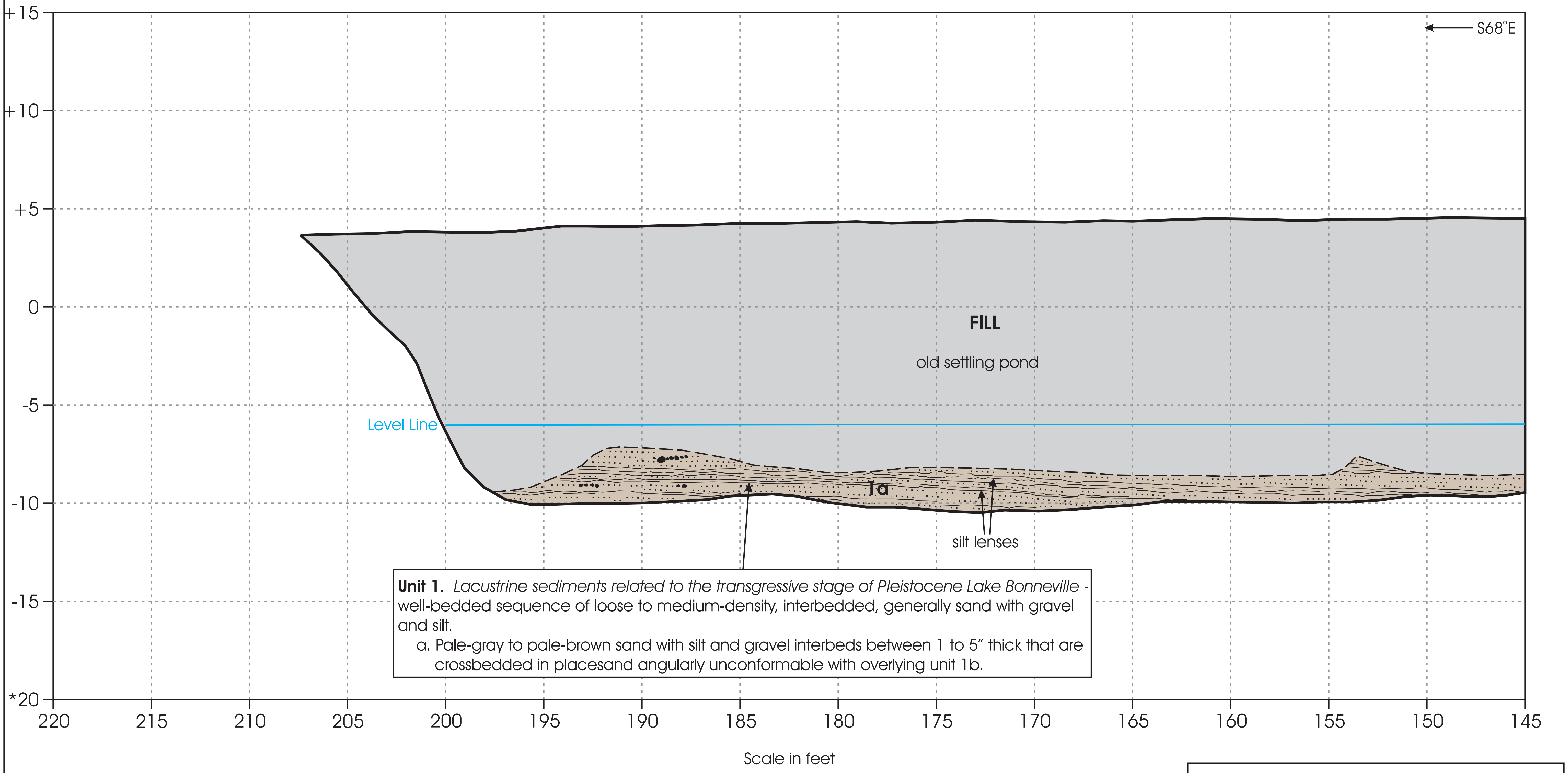
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

Trench logged by Bill Black, P.G. on
May 3 and 16, 2009

TRENCH 4 LOG, SHEET 2

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 8B



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SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

Trench logged by Bill Black, P.G. on
May 3 and 16, 2009
Log reviewed by Craig V Nelson, P.G., R.G., C.E.G

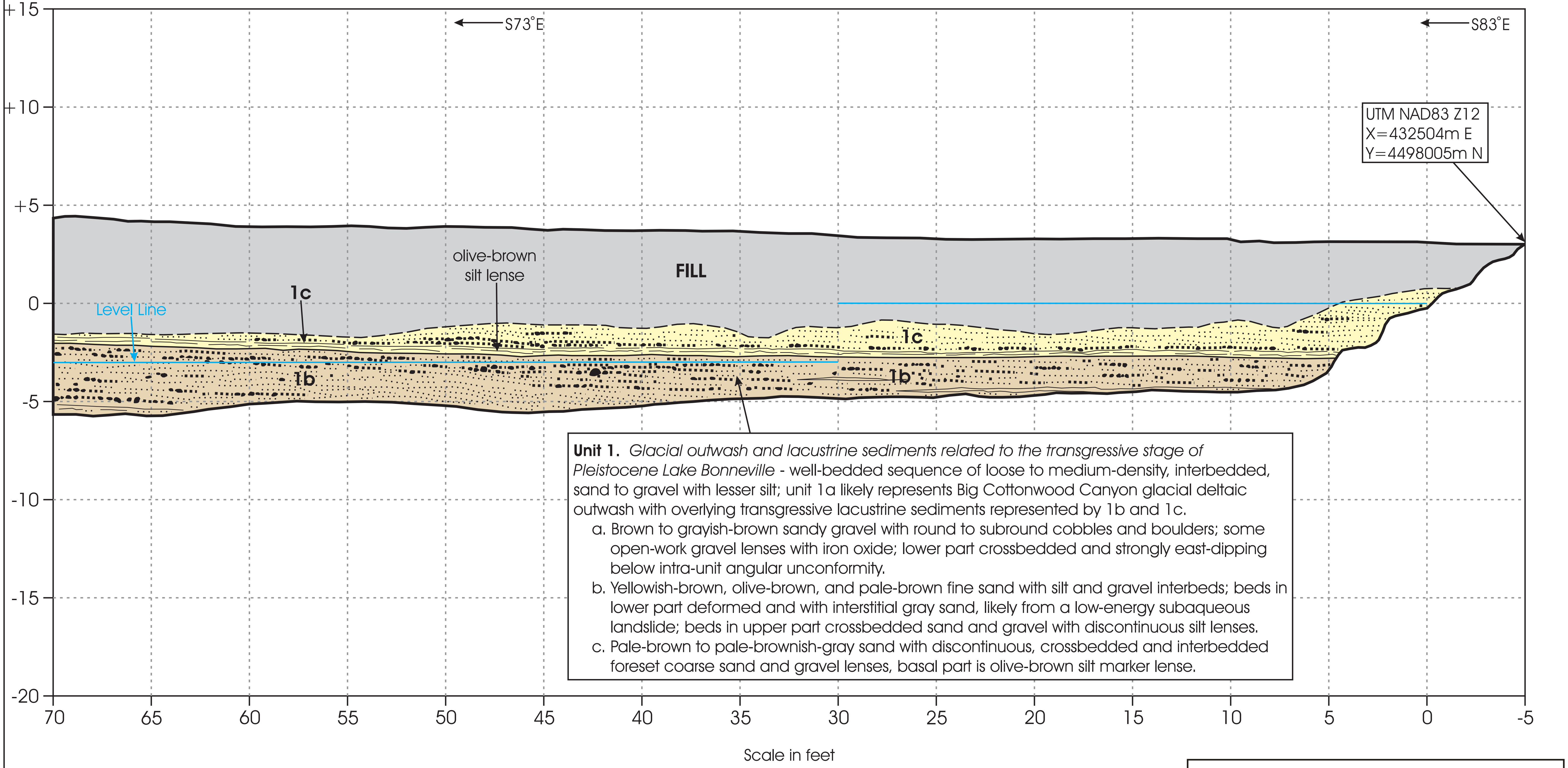
WESTERN GEOLOGIC

TRENCH 4 LOG, SHEET 3

GEOLOGIC HAZARDS EVALUATION

AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 8C



Unit 1. *Glacial outwash and lacustrine sediments related to the transgressive stage of Pleistocene Lake Bonneville* - well-bedded sequence of loose to medium-density, interbedded, sand to gravel with lesser silt; unit 1a likely represents Big Cottonwood Canyon glacial deltaic outwash with overlying transgressive lacustrine sediments represented by 1b and 1c.

a. Brown to grayish-brown sandy gravel with round to subround cobbles and boulders; some open-work gravel lenses with iron oxide; lower part crossbedded and strongly east-dipping below intra-unit angular unconformity.

b. Yellowish-brown, olive-brown, and pale-brown fine sand with silt and gravel interbeds; beds in lower part deformed and with interstitial gray sand, likely from a low-energy subaqueous landslide; beds in upper part crossbedded sand and gravel with discontinuous silt lenses.

c. Pale-brown to pale-brownish-gray sand with discontinuous, crossbedded and interbedded foreset coarse sand and gravel lenses, basal part is olive-brown silt marker lense.

SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

Trench logged by Bill Black, P.G. on
May 16-17 and 24, 2009

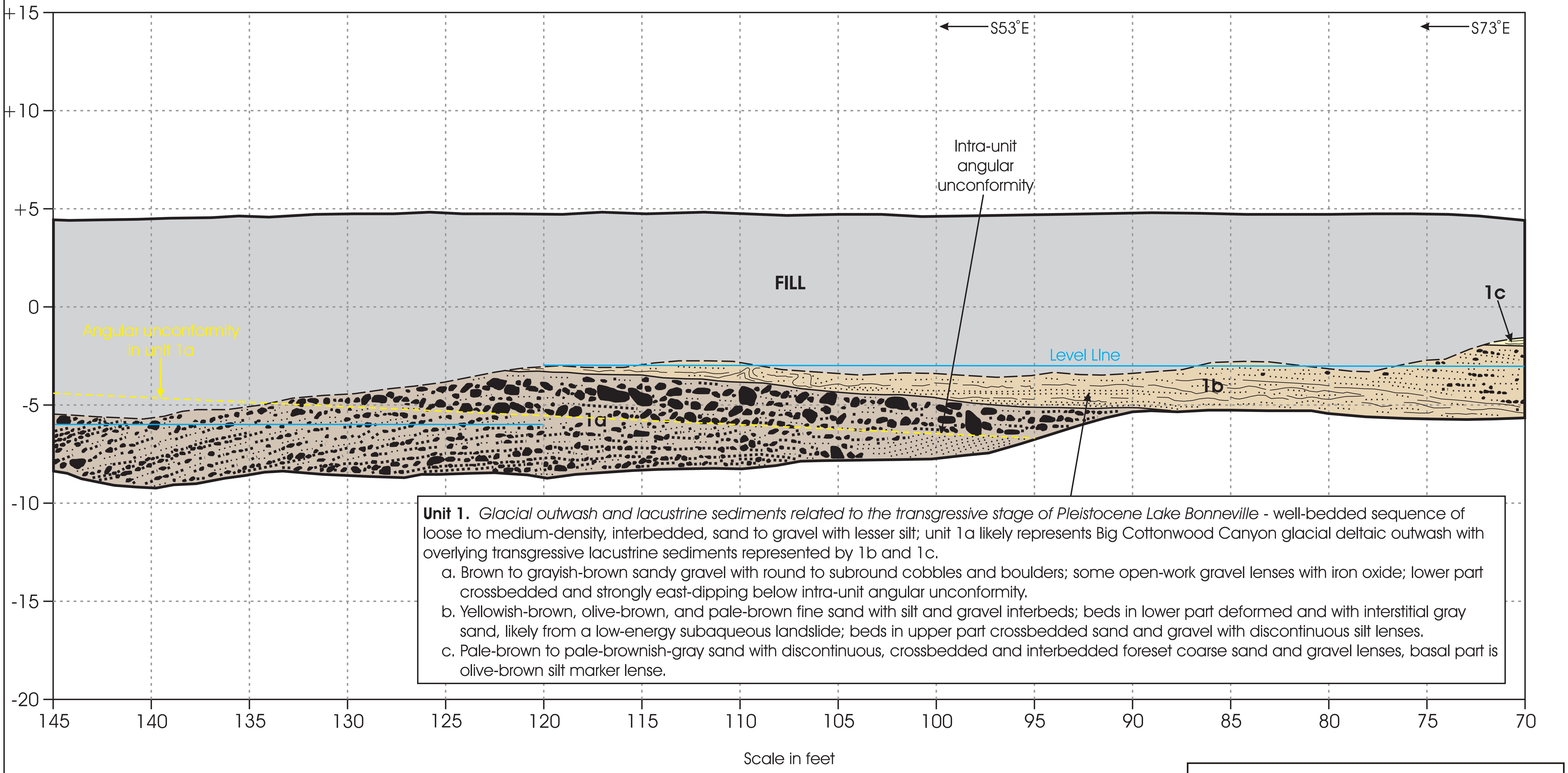
WESTERN
GeoLogic

TRENCH 5 LOG, SHEET 1

GEOLOGIC HAZARDS EVALUATION

AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 9A



Unit 1. *Glacial outwash and lacustrine sediments related to the transgressive stage of Pleistocene Lake Bonneville* - well-bedded sequence of loose to medium-density, interbedded, sand to gravel with lesser silt; unit 1a likely represents Big Cottonwood Canyon glacial deltaic outwash with overlying transgressive lacustrine sediments represented by 1b and 1c.

a. Brown to grayish-brown sandy gravel with round to subround cobbles and boulders; some open-work gravel lenses with iron oxide; lower part crossbedded and strongly east-dipping below intra-unit angular unconformity.

b. Yellowish-brown, olive-brown, and pale-brown fine sand with silt and gravel interbeds; beds in lower part deformed and with interstitial gray sand, likely from a low-energy subaqueous landslide; beds in upper part crossbedded sand and gravel with discontinuous silt lenses.

c. Pale-brown to pale-brownish-gray sand with discontinuous, crossbedded and interbedded foreset coarse sand and gravel lenses, basal part is olive-brown silt marker lense.

WESTERN
GeoLogic

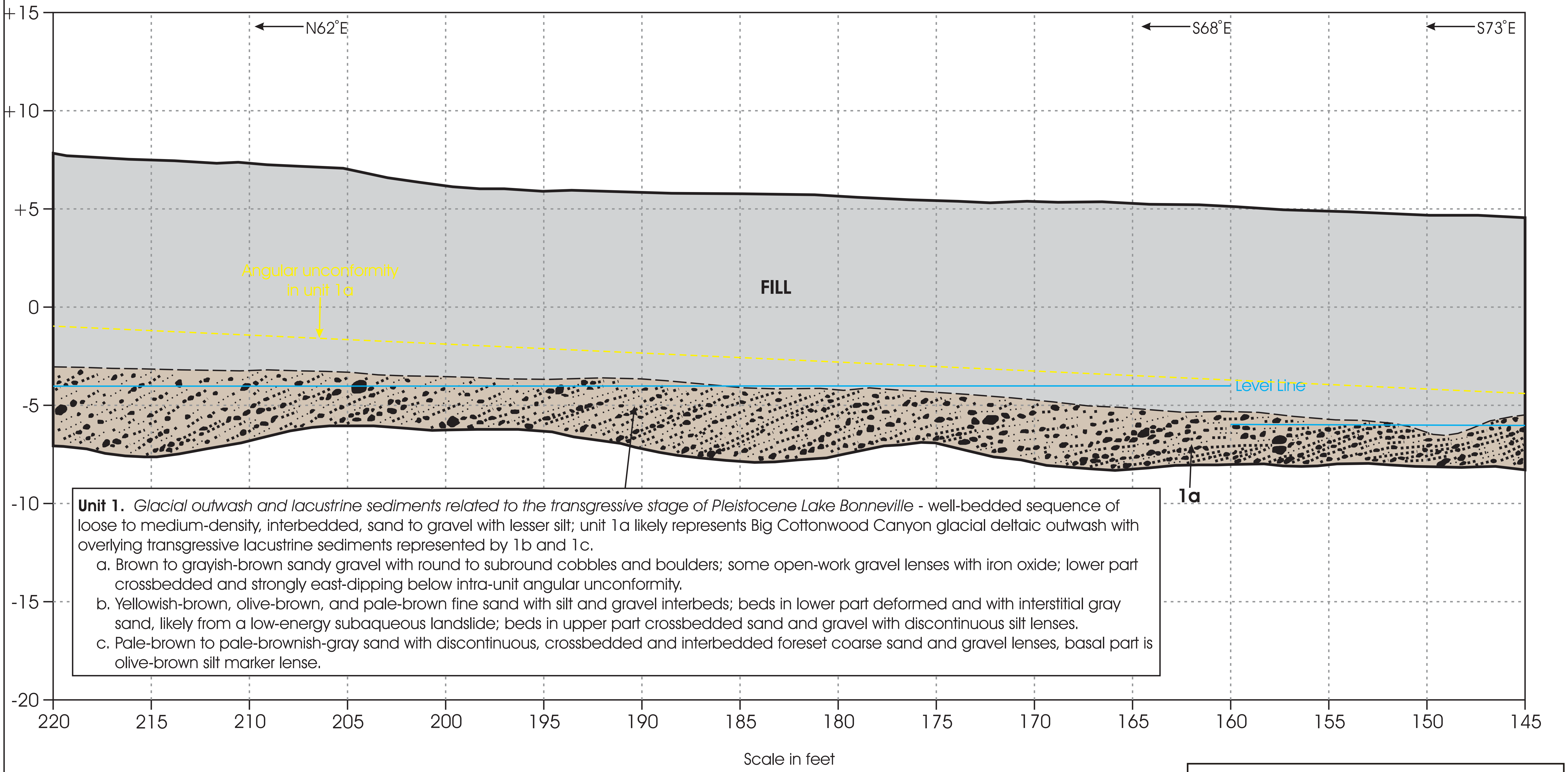
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

Trench logged by Bill Black, P.G. on
May 16-17 and 24, 2009

TRENCH 5 LOG, SHEET 2

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 9B



Unit 1. *Glacial outwash and lacustrine sediments related to the transgressive stage of Pleistocene Lake Bonneville* - well-bedded sequence of loose to medium-density, interbedded, sand to gravel with lesser silt; unit 1a likely represents Big Cottonwood Canyon glacial deltaic outwash with overlying transgressive lacustrine sediments represented by 1b and 1c.

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b. Yellowish-brown, olive-brown, and pale-brown fine sand with silt and gravel interbeds; beds in lower part deformed and with interstitial gray sand, likely from a low-energy subaqueous landslide; beds in upper part crossbedded sand and gravel with discontinuous silt lenses.

c. Pale-brown to pale-brownish-gray sand with discontinuous, crossbedded and interbedded foreset coarse sand and gravel lenses, basal part is olive-brown silt marker lense.

WESTERN GEOLOGIC

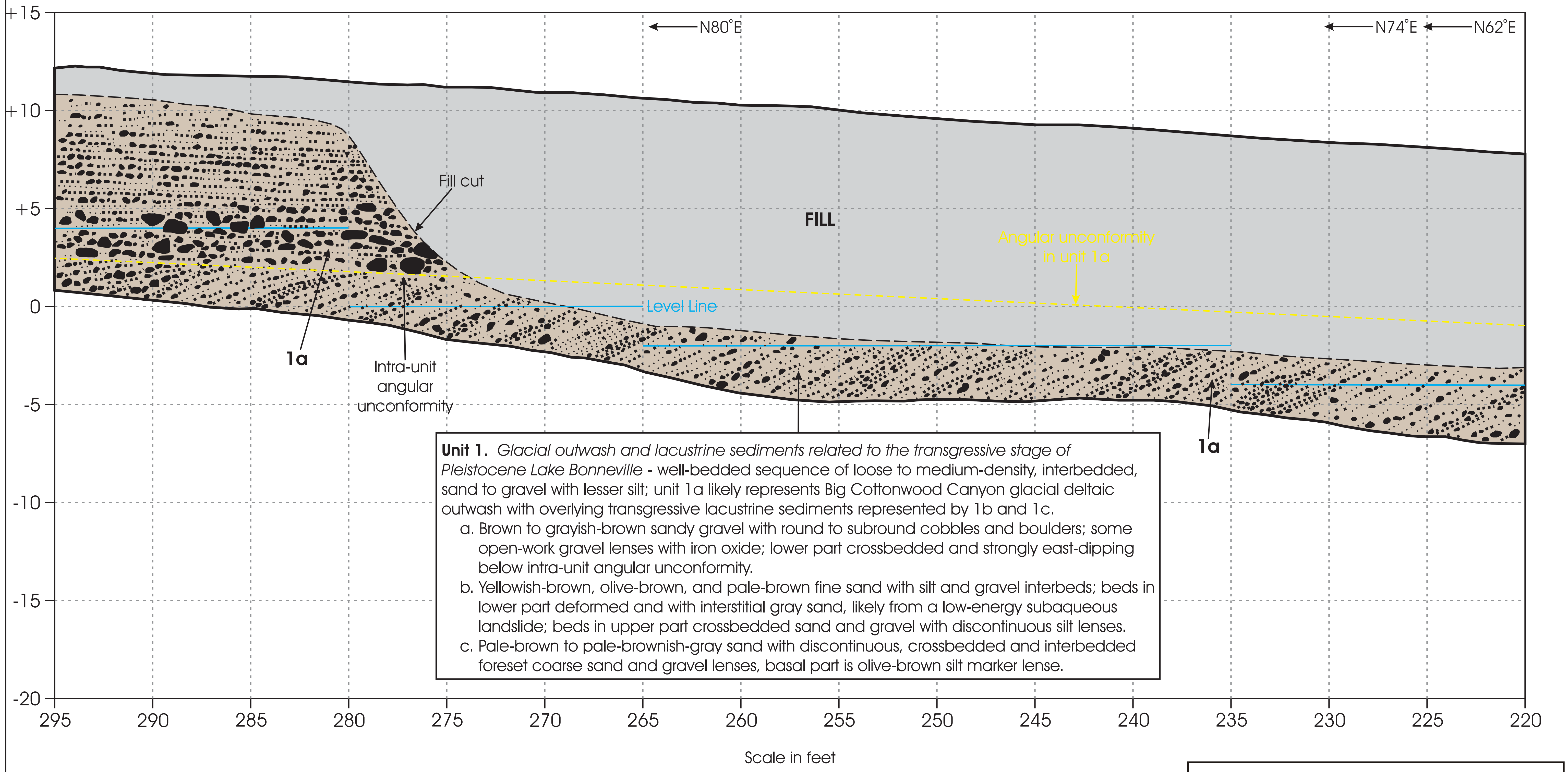
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

Trench logged by Bill Black, P.G. on
May 16-17 and 24, 2009

TRENCH 5 LOG, SHEET 3

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 9C



Unit 1. *Glacial outwash and lacustrine sediments related to the transgressive stage of Pleistocene Lake Bonneville* - well-bedded sequence of loose to medium-density, interbedded, sand to gravel with lesser silt; unit 1a likely represents Big Cottonwood Canyon glacial deltaic outwash with overlying transgressive lacustrine sediments represented by 1b and 1c.

a. Brown to grayish-brown sandy gravel with round to subround cobbles and boulders; some open-work gravel lenses with iron oxide; lower part crossbedded and strongly east-dipping below intra-unit angular unconformity.

b. Yellowish-brown, olive-brown, and pale-brown fine sand with silt and gravel interbeds; beds in lower part deformed and with interstitial gray sand, likely from a low-energy subaqueous landslide; beds in upper part crossbedded sand and gravel with discontinuous silt lenses.

c. Pale-brown to pale-brownish-gray sand with discontinuous, crossbedded and interbedded foreset coarse sand and gravel lenses, basal part is olive-brown silt marker lense.

SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

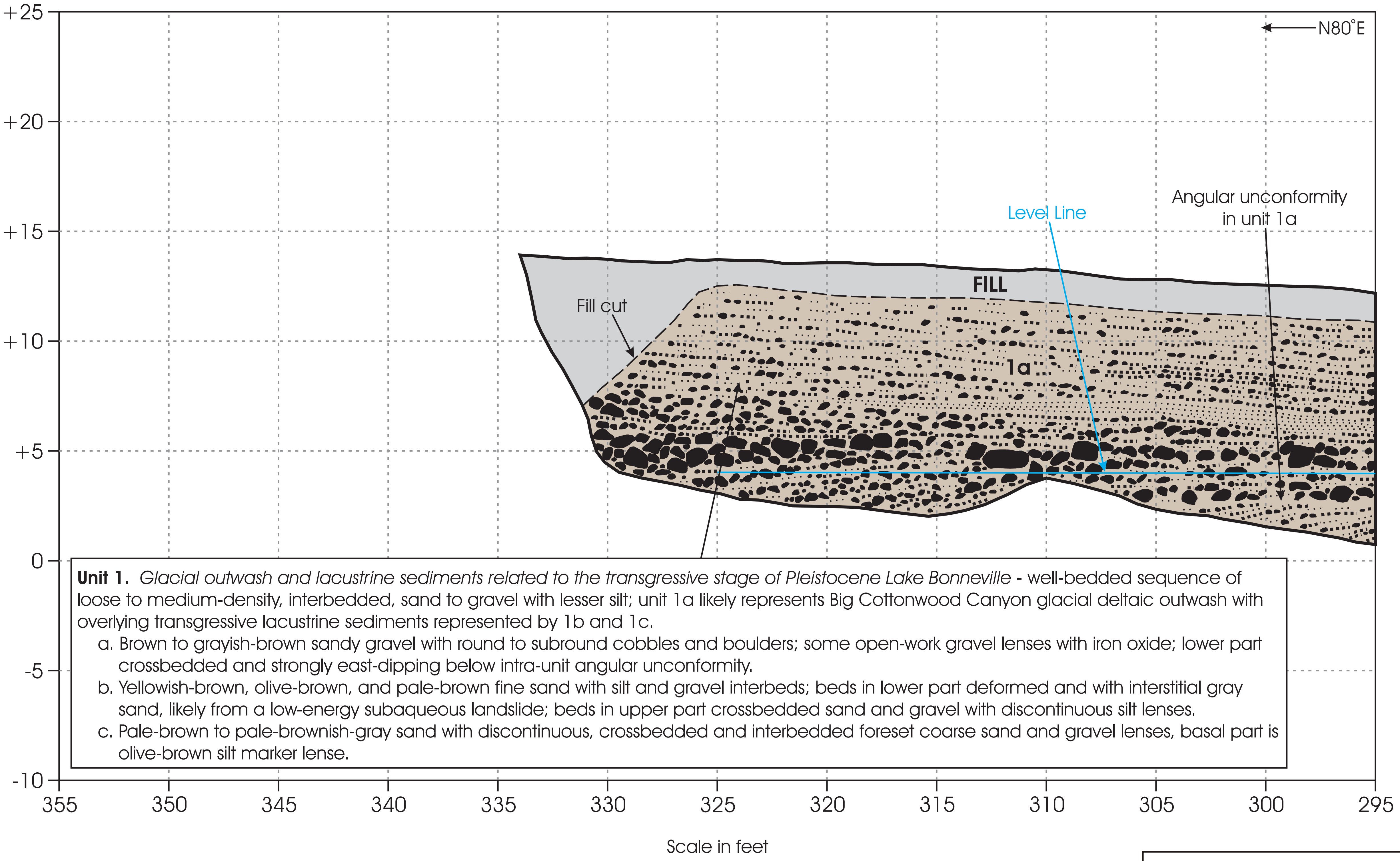
Trench logged by Bill Black, P.G. on
May 16-17 and 24, 2009

WESTERN
GEOLOGIC

TRENCH 5 LOG, SHEET 4

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 9D



Unit 1. *Glacial outwash and lacustrine sediments related to the transgressive stage of Pleistocene Lake Bonneville* - well-bedded sequence of loose to medium-density, interbedded, sand to gravel with lesser silt; unit 1a likely represents Big Cottonwood Canyon glacial deltaic outwash with overlying transgressive lacustrine sediments represented by 1b and 1c.

- a. Brown to grayish-brown sandy gravel with round to subround cobbles and boulders; some open-work gravel lenses with iron oxide; lower part crossbedded and strongly east-dipping below intra-unit angular unconformity.
- b. Yellowish-brown, olive-brown, and pale-brown fine sand with silt and gravel interbeds; beds in lower part deformed and with interstitial gray sand, likely from a low-energy subaqueous landslide; beds in upper part crossbedded sand and gravel with discontinuous silt lenses.
- c. Pale-brown to pale-brownish-gray sand with discontinuous, crossbedded and interbedded foreset coarse sand and gravel lenses, basal part is olive-brown silt marker lense.

SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

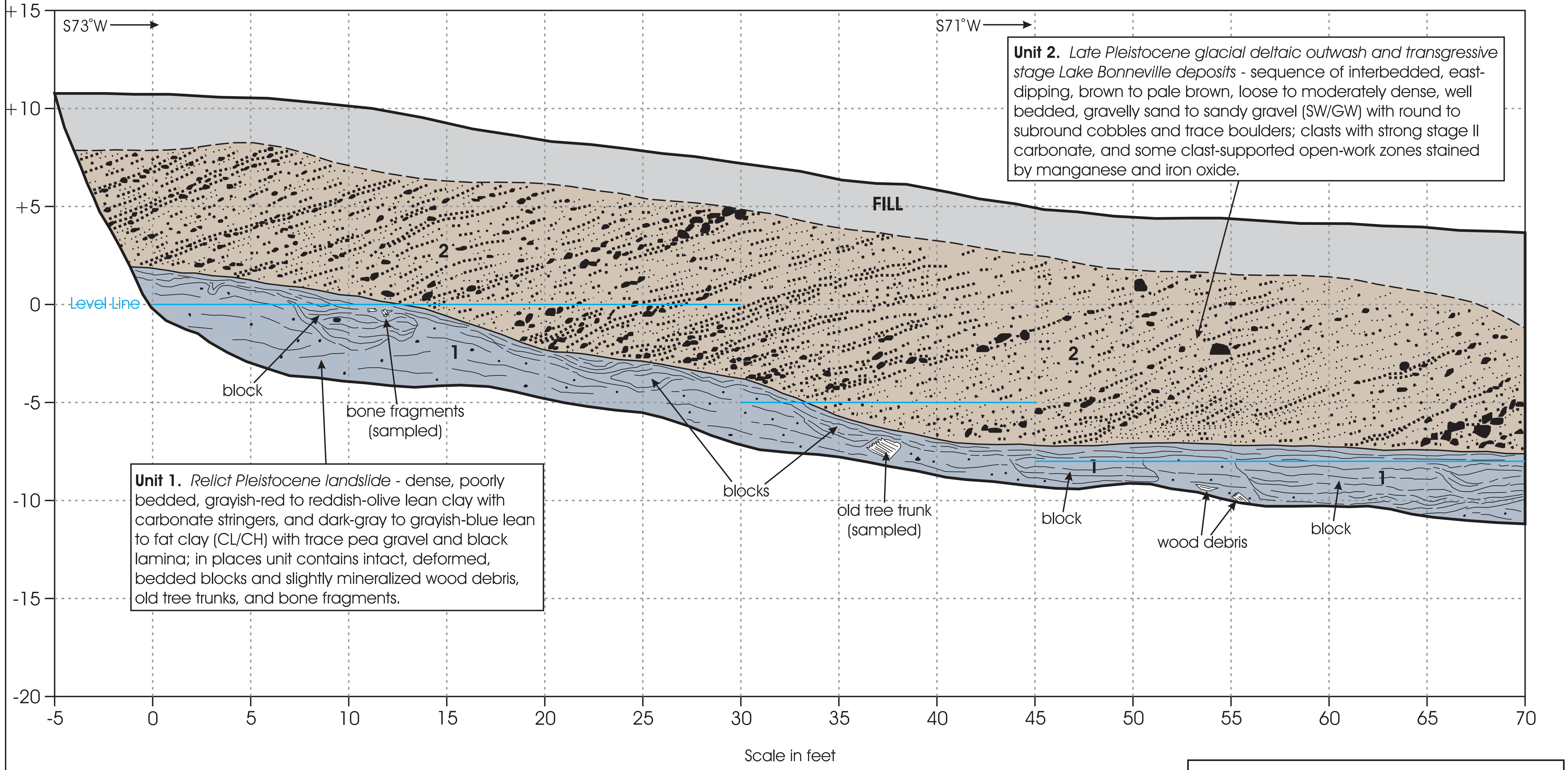
Trench logged by Bill Black, P.G. on
May 16-17 and 24, 2009
Log reviewed by Craig V Nelson, P.G., R.G., C.E.G

WESTERN GEOLOGIC

TRENCH 5 LOG, SHEET 5

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 9E



Unit 2. *Late Pleistocene glacial deltaic outwash and transgressive stage Lake Bonneville deposits* - sequence of interbedded, east-dipping, brown to pale brown, loose to moderately dense, well bedded, gravelly sand to sandy gravel (SW/GW) with round to subround cobbles and trace boulders; clasts with strong stage II carbonate, and some clast-supported open-work zones stained by manganese and iron oxide.

Unit 1. *Relict Pleistocene landslide* - dense, poorly bedded, grayish-red to reddish-olive lean clay with carbonate stringers, and dark-gray to grayish-blue lean to fat clay (CL/CH) with trace pea gravel and black lamina; in places unit contains intact, deformed, bedded blocks and slightly mineralized wood debris, old tree trunks, and bone fragments.

SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

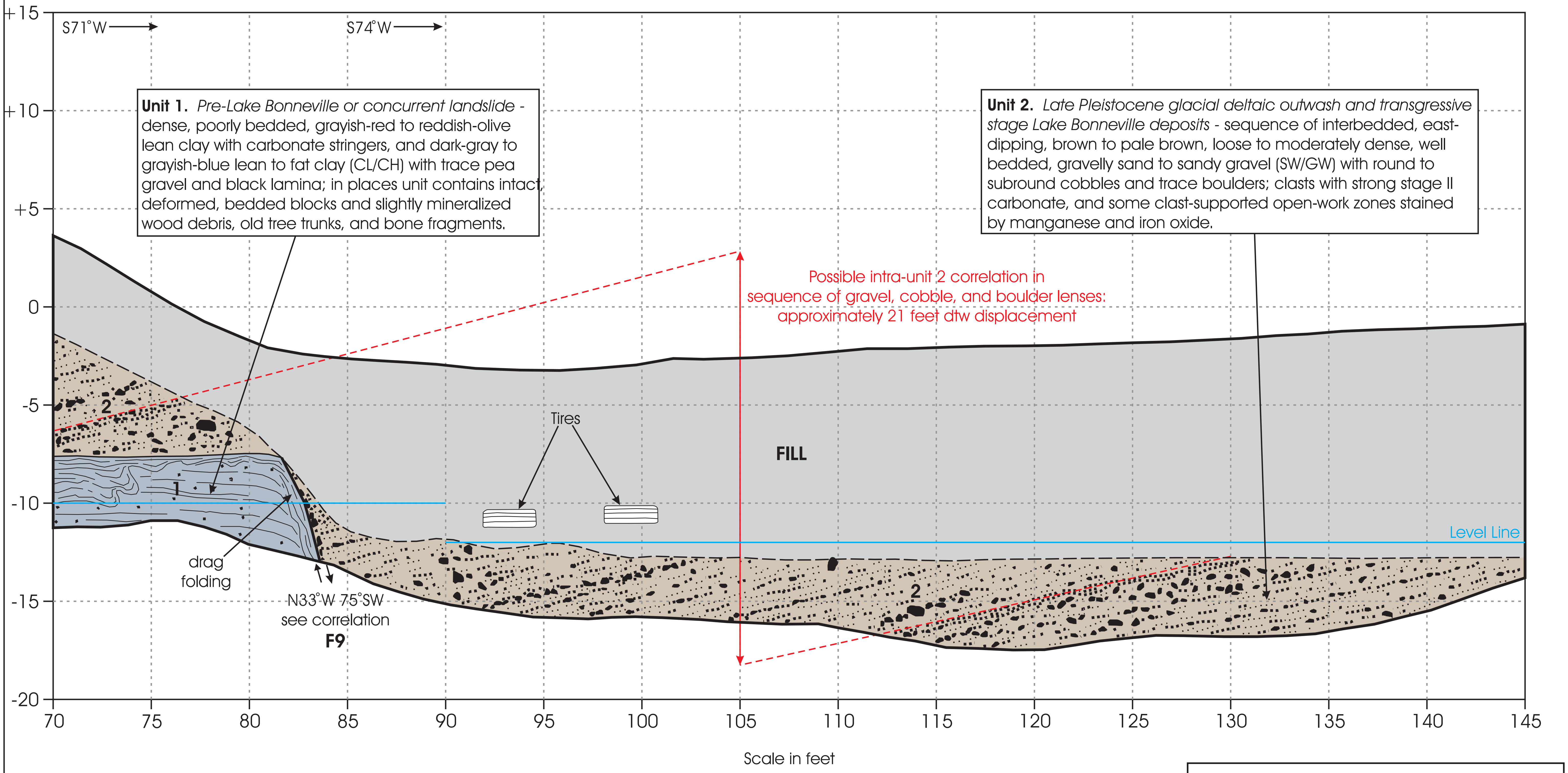
Trench logged by Bill Black, P.G. on
May 23, 2009

WESTERN GEOLOGIC

TRENCH 6 LOG, SHEET 1

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 10A



Unit 1. *Pre-Lake Bonneville or concurrent landslide* - dense, poorly bedded, grayish-red to reddish-olive lean clay with carbonate stringers, and dark-gray to grayish-blue lean to fat clay (CL/CH) with trace pea gravel and black lamina; in places unit contains intact, deformed, bedded blocks and slightly mineralized wood debris, old tree trunks, and bone fragments.

Unit 2. *Late Pleistocene glacial deltaic outwash and transgressive stage Lake Bonneville deposits* - sequence of interbedded, east-dipping, brown to pale brown, loose to moderately dense, well bedded, gravelly sand to sandy gravel (SW/GW) with round to subround cobbles and trace boulders; clasts with strong stage II carbonate, and some clast-supported open-work zones stained by manganese and iron oxide.

Possible intra-unit 2 correlation in sequence of gravel, cobble, and boulder lenses: approximately 21 feet dtw displacement

SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

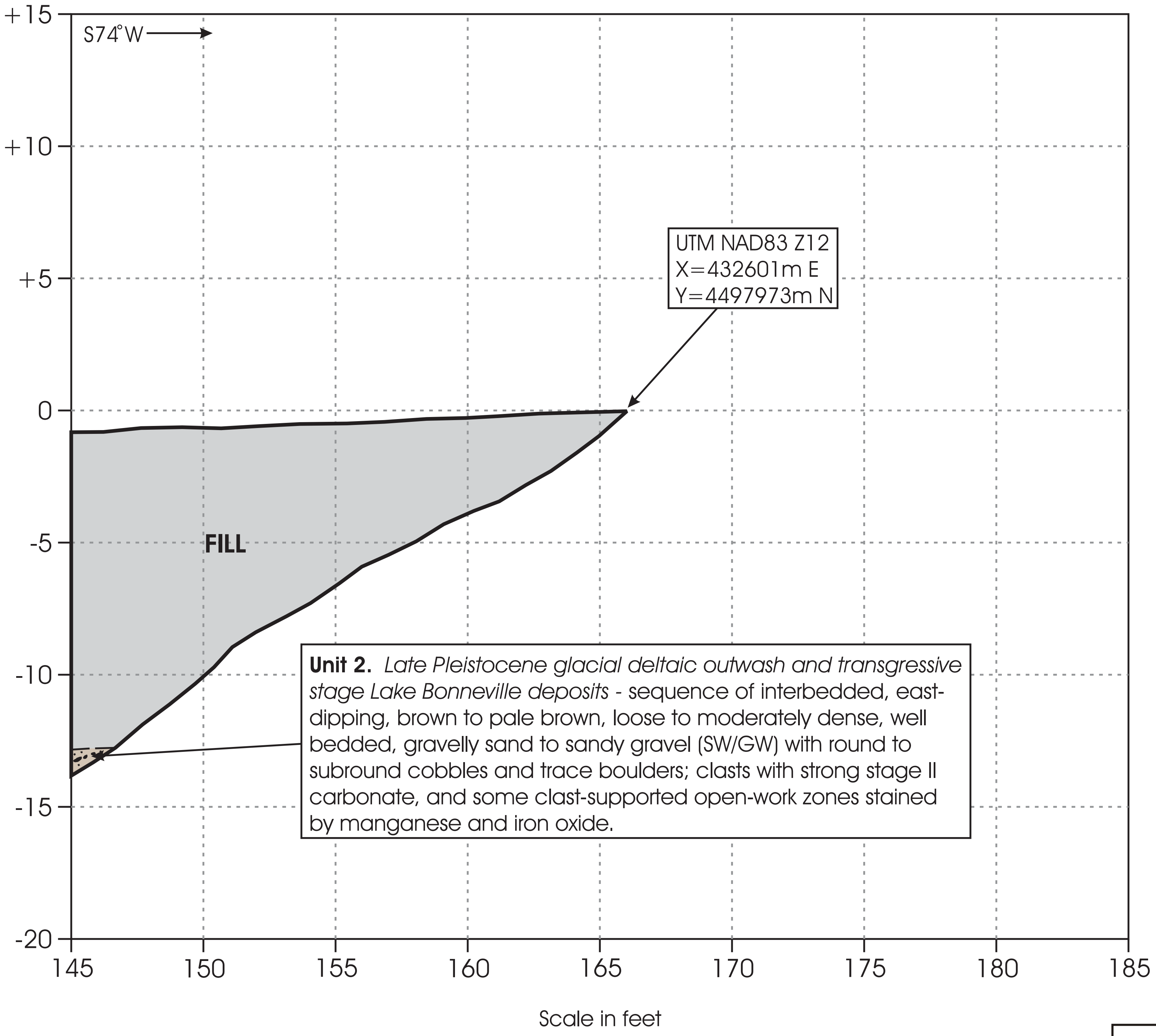
Trench logged by Bill Black, P.G. on
May 23, 2009

WESTERN
GEOLOGIC

TRENCH 6 LOG, SHEET 2

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 10B



Unit 2. *Late Pleistocene glacial deltaic outwash and transgressive stage Lake Bonneville deposits* - sequence of interbedded, east-dipping, brown to pale brown, loose to moderately dense, well bedded, gravelly sand to sandy gravel (SW/GW) with round to subround cobbles and trace boulders; clasts with strong stage II carbonate, and some clast-supported open-work zones stained by manganese and iron oxide.

SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

Trench logged by Bill Black, P.G. on
May 23, 2009

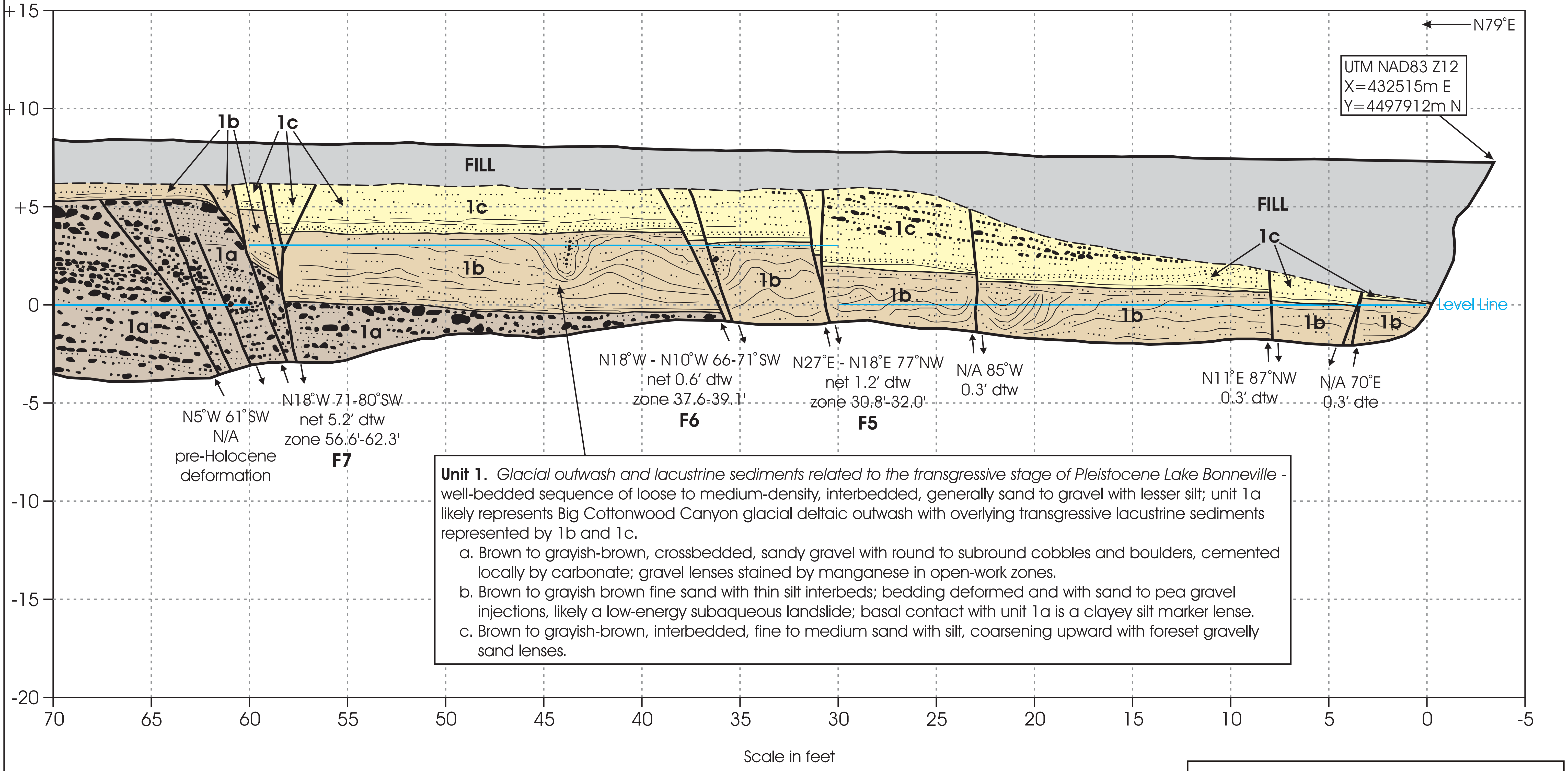
WESTERN GEOLOGIC

TRENCH 6 LOG, SHEET 3

GEOLOGIC HAZARDS EVALUATION

AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 10C



Unit 1. *Glacial outwash and lacustrine sediments related to the transgressive stage of Pleistocene Lake Bonneville* - well-bedded sequence of loose to medium-density, interbedded, generally sand to gravel with lesser silt; unit 1a likely represents Big Cottonwood Canyon glacial deltaic outwash with overlying transgressive lacustrine sediments represented by 1b and 1c.

a. Brown to grayish-brown, crossbedded, sandy gravel with round to subround cobbles and boulders, cemented locally by carbonate; gravel lenses stained by manganese in open-work zones.
b. Brown to grayish brown fine sand with thin silt interbeds; bedding deformed and with sand to pea gravel injections, likely a low-energy subaqueous landslide; basal contact with unit 1a is a clayey silt marker lense.
c. Brown to grayish-brown, interbedded, fine to medium sand with silt, coarsening upward with foreset gravelly sand lenses.

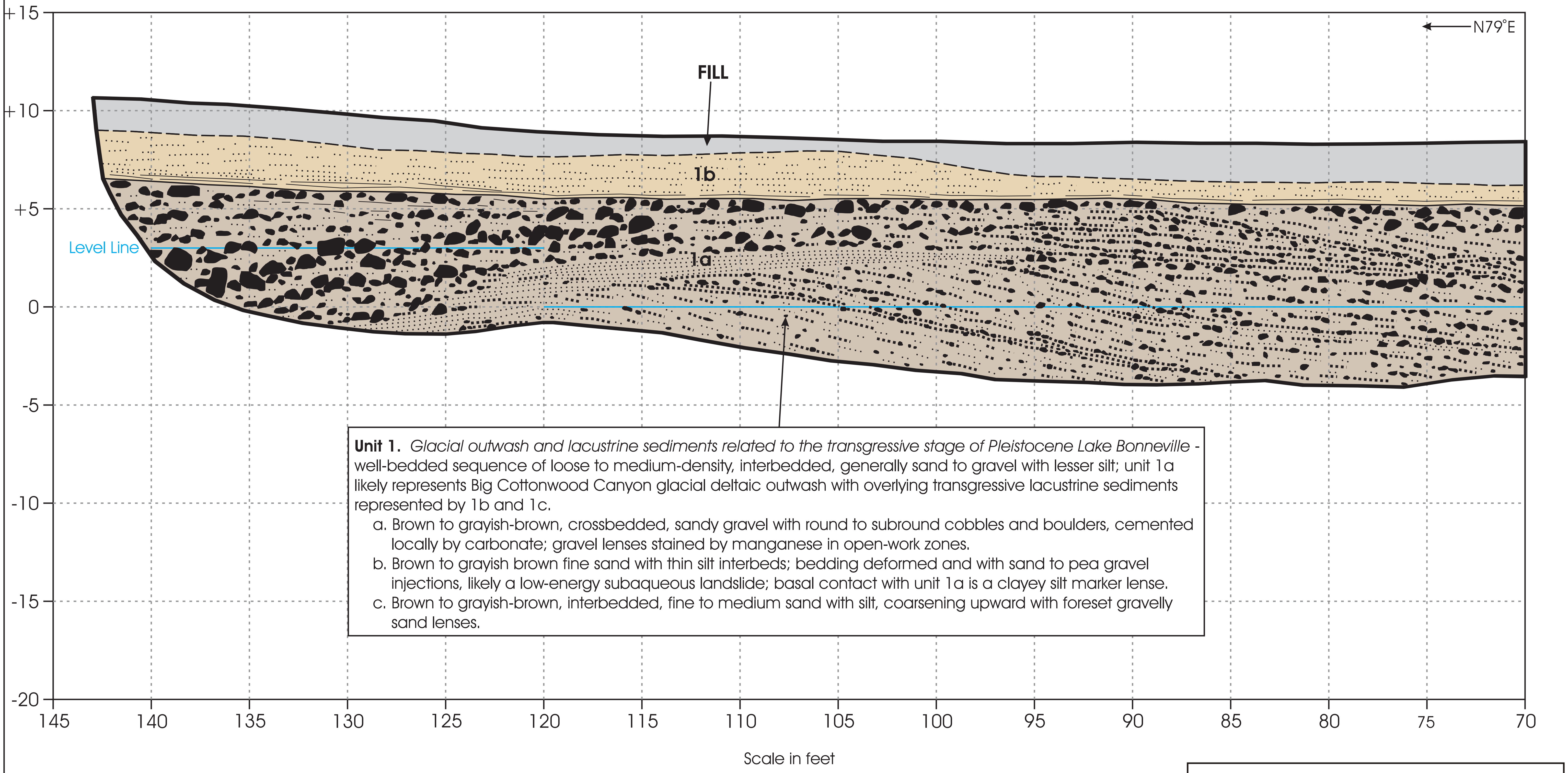
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

Trench logged by Bill Black, P.G. on
May 30, 2009

TRENCH 7 LOG, SHEET 1

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 11A



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a. Brown to grayish-brown, crossbedded, sandy gravel with round to subround cobbles and boulders, cemented locally by carbonate; gravel lenses stained by manganese in open-work zones.

b. Brown to grayish brown fine sand with thin silt interbeds; bedding deformed and with sand to pea gravel injections, likely a low-energy subaqueous landslide; basal contact with unit 1a is a clayey silt marker lense.

c. Brown to grayish-brown, interbedded, fine to medium sand with silt, coarsening upward with foreset gravelly sand lenses.

SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

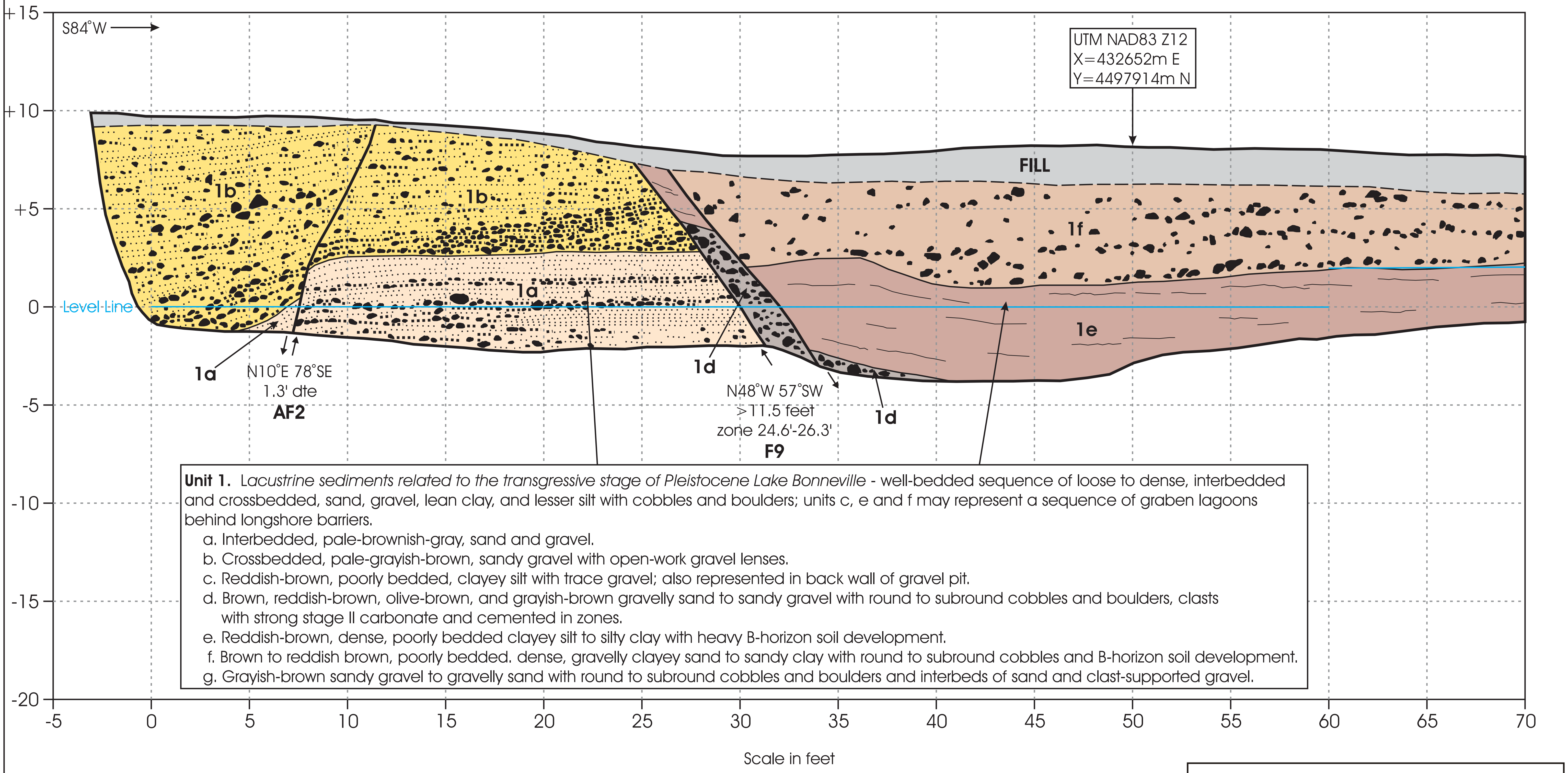
Trench logged by Bill Black, P.G. on
May 30, 2009

WESTERN GEOLOGIC

TRENCH 7 LOG, SHEET 2

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 11B



Unit 1. *Lacustrine sediments related to the transgressive stage of Pleistocene Lake Bonneville* - well-bedded sequence of loose to dense, interbedded and crossbedded, sand, gravel, lean clay, and lesser silt with cobbles and boulders; units c, e and f may represent a sequence of graben lagoons behind longshore barriers.

a. Interbedded, pale-brownish-gray, sand and gravel.
b. Crossbedded, pale-grayish-brown, sandy gravel with open-work gravel lenses.
c. Reddish-brown, poorly bedded, clayey silt with trace gravel; also represented in back wall of gravel pit.
d. Brown, reddish-brown, olive-brown, and grayish-brown gravelly sand to sandy gravel with round to subround cobbles and boulders, clasts with strong stage II carbonate and cemented in zones.
e. Reddish-brown, dense, poorly bedded clayey silt to silty clay with heavy B-horizon soil development.
f. Brown to reddish brown, poorly bedded. dense, gravelly clayey sand to sandy clay with round to subround cobbles and B-horizon soil development.
g. Grayish-brown sandy gravel to gravelly sand with round to subround cobbles and boulders and interbeds of sand and clast-supported gravel.

WESTERN
GEOLOGIC

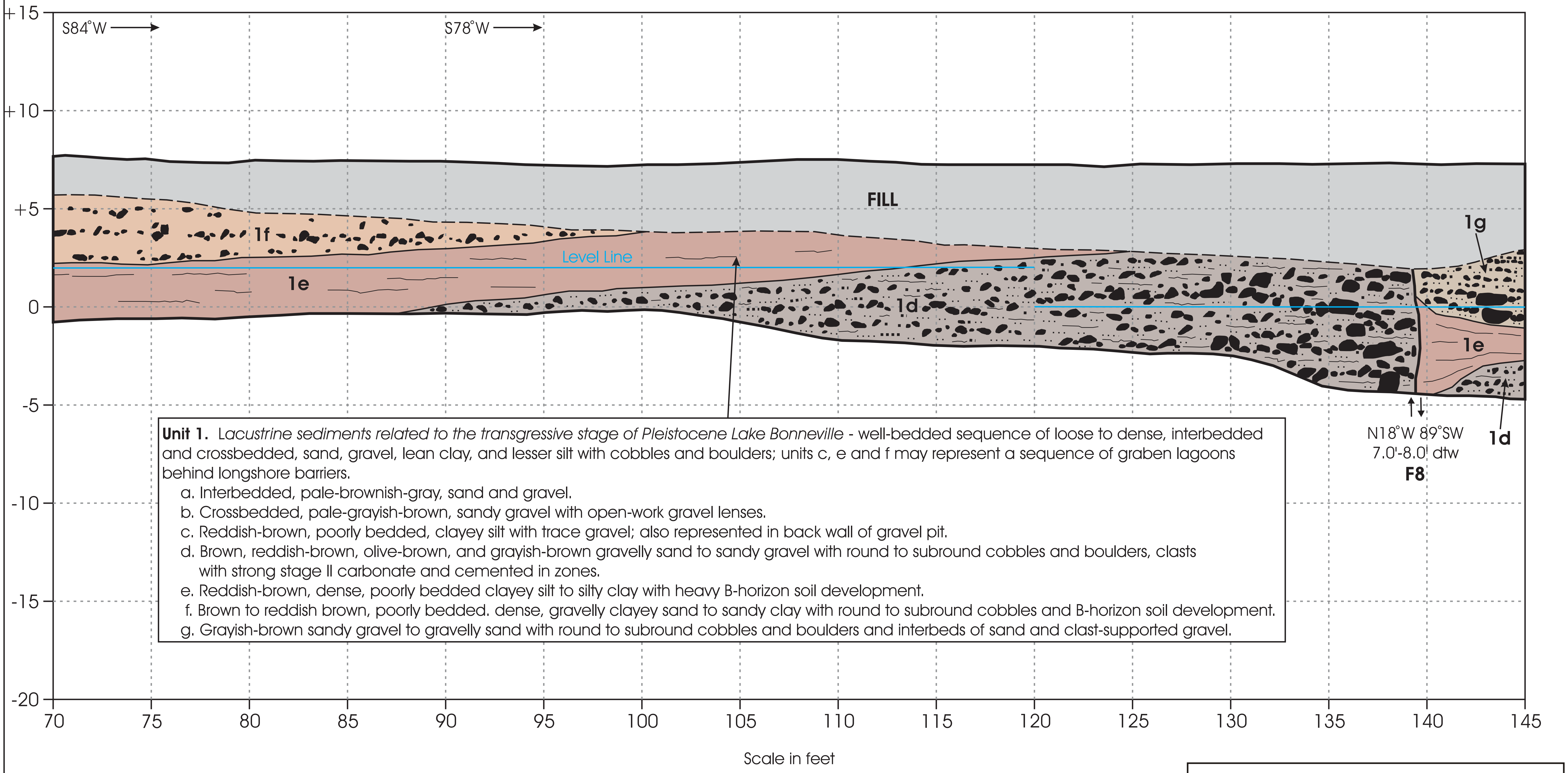
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

Trench logged by Bill Black, P.G. on
May 30-31, 2009

TRENCH 8 LOG, SHEET 1

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 12A



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a. Interbedded, pale-brownish-gray, sand and gravel.
b. Crossbedded, pale-grayish-brown, sandy gravel with open-work gravel lenses.
c. Reddish-brown, poorly bedded, clayey silt with trace gravel; also represented in back wall of gravel pit.
d. Brown, reddish-brown, olive-brown, and grayish-brown gravelly sand to sandy gravel with round to subround cobbles and boulders, clasts with strong stage II carbonate and cemented in zones.
e. Reddish-brown, dense, poorly bedded clayey silt to silty clay with heavy B-horizon soil development.
f. Brown to reddish brown, poorly bedded. dense, gravelly clayey sand to sandy clay with round to subround cobbles and B-horizon soil development.
g. Grayish-brown sandy gravel to gravelly sand with round to subround cobbles and boulders and interbeds of sand and clast-supported gravel.

SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

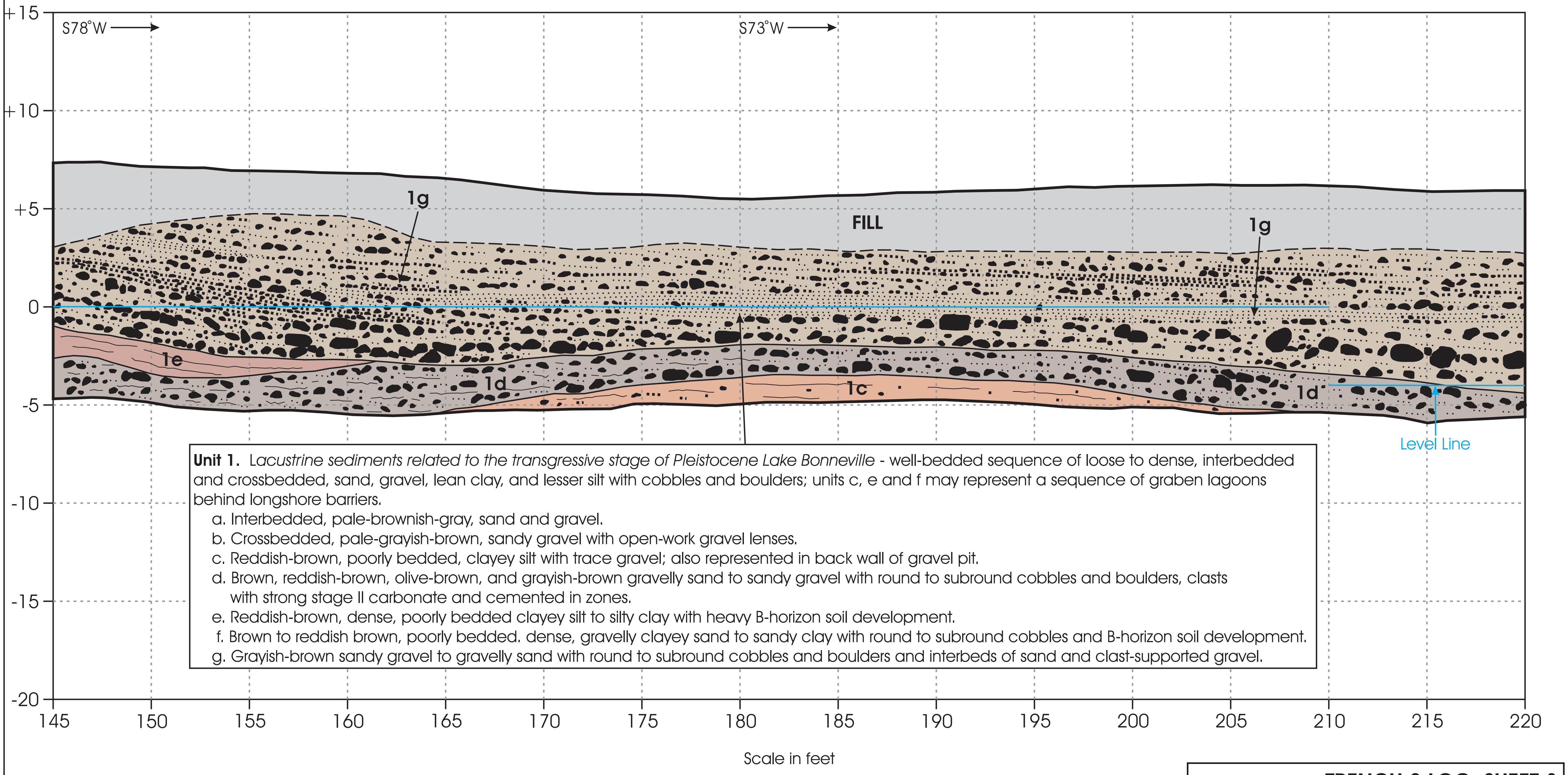
Trench logged by Bill Black, P.G. on
May 30-31, 2009

WESTERN GEOLOGIC

TRENCH 8 LOG, SHEET 2

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 12B



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d. Brown, reddish-brown, olive-brown, and grayish-brown gravelly sand to sandy gravel with round to subround cobbles and boulders, clasts with strong stage II carbonate and cemented in zones.
e. Reddish-brown, dense, poorly bedded clayey silt to silty clay with heavy B-horizon soil development.
f. Brown to reddish brown, poorly bedded. dense, gravelly clayey sand to sandy clay with round to subround cobbles and B-horizon soil development.
g. Grayish-brown sandy gravel to gravelly sand with round to subround cobbles and boulders and interbeds of sand and clast-supported gravel.

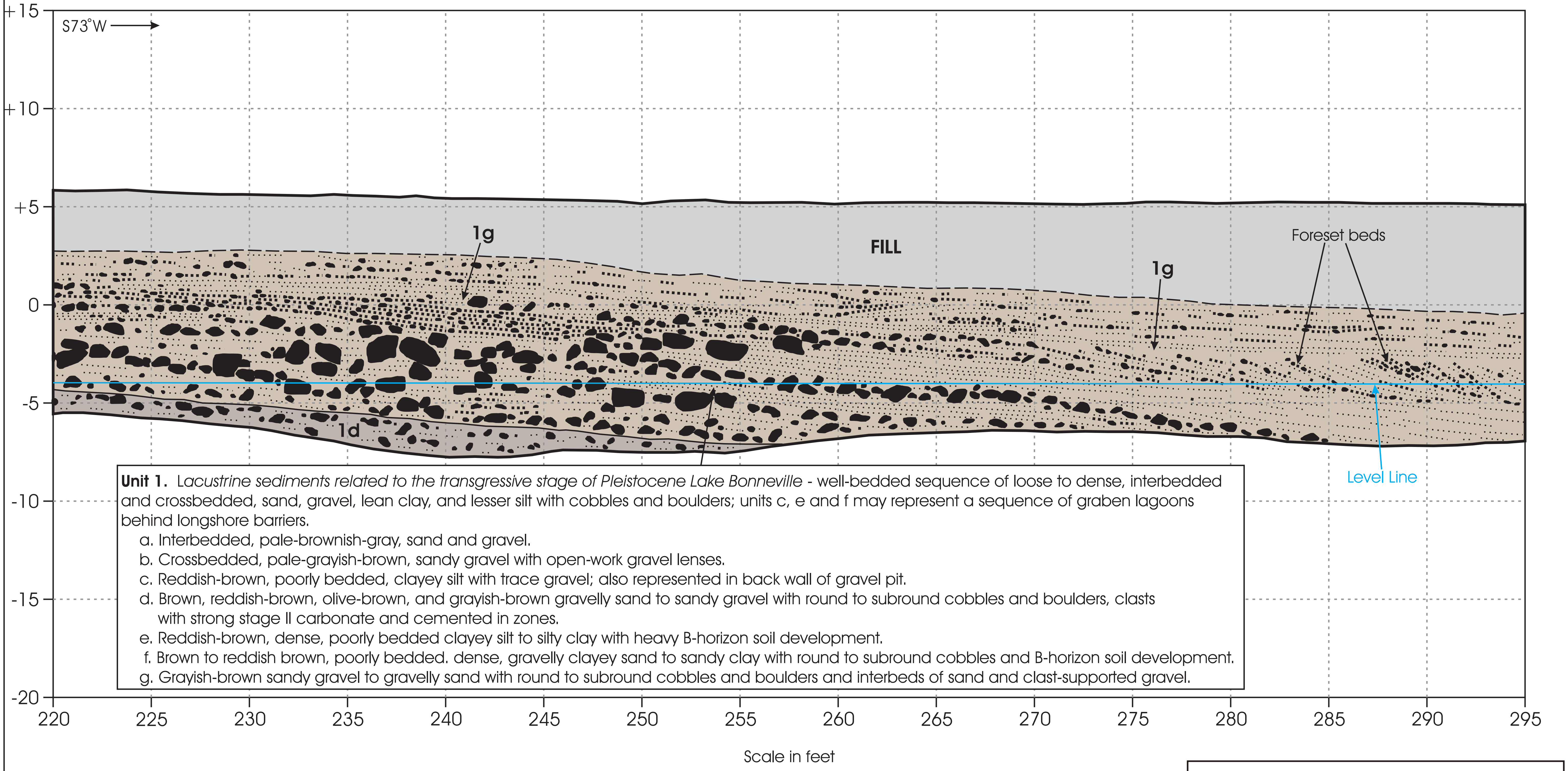
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

Trench logged by Bill Black, P.G. on
May 30-31, 2009

TRENCH 8 LOG, SHEET 3

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 12C



Unit 1. *Lacustrine sediments related to the transgressive stage of Pleistocene Lake Bonneville* - well-bedded sequence of loose to dense, interbedded and crossbedded, sand, gravel, lean clay, and lesser silt with cobbles and boulders; units c, e and f may represent a sequence of graben lagoons behind longshore barriers.

- a. Interbedded, pale-brownish-gray, sand and gravel.
- b. Crossbedded, pale-grayish-brown, sandy gravel with open-work gravel lenses.
- c. Reddish-brown, poorly bedded, clayey silt with trace gravel; also represented in back wall of gravel pit.
- d. Brown, reddish-brown, olive-brown, and grayish-brown gravelly sand to sandy gravel with round to subround cobbles and boulders, clasts with strong stage II carbonate and cemented in zones.
- e. Reddish-brown, dense, poorly bedded clayey silt to silty clay with heavy B-horizon soil development.
- f. Brown to reddish brown, poorly bedded. dense, gravelly clayey sand to sandy clay with round to subround cobbles and B-horizon soil development.
- g. Grayish-brown sandy gravel to gravelly sand with round to subround cobbles and boulders and interbeds of sand and clast-supported gravel.

WESTERN GEOLOGIC

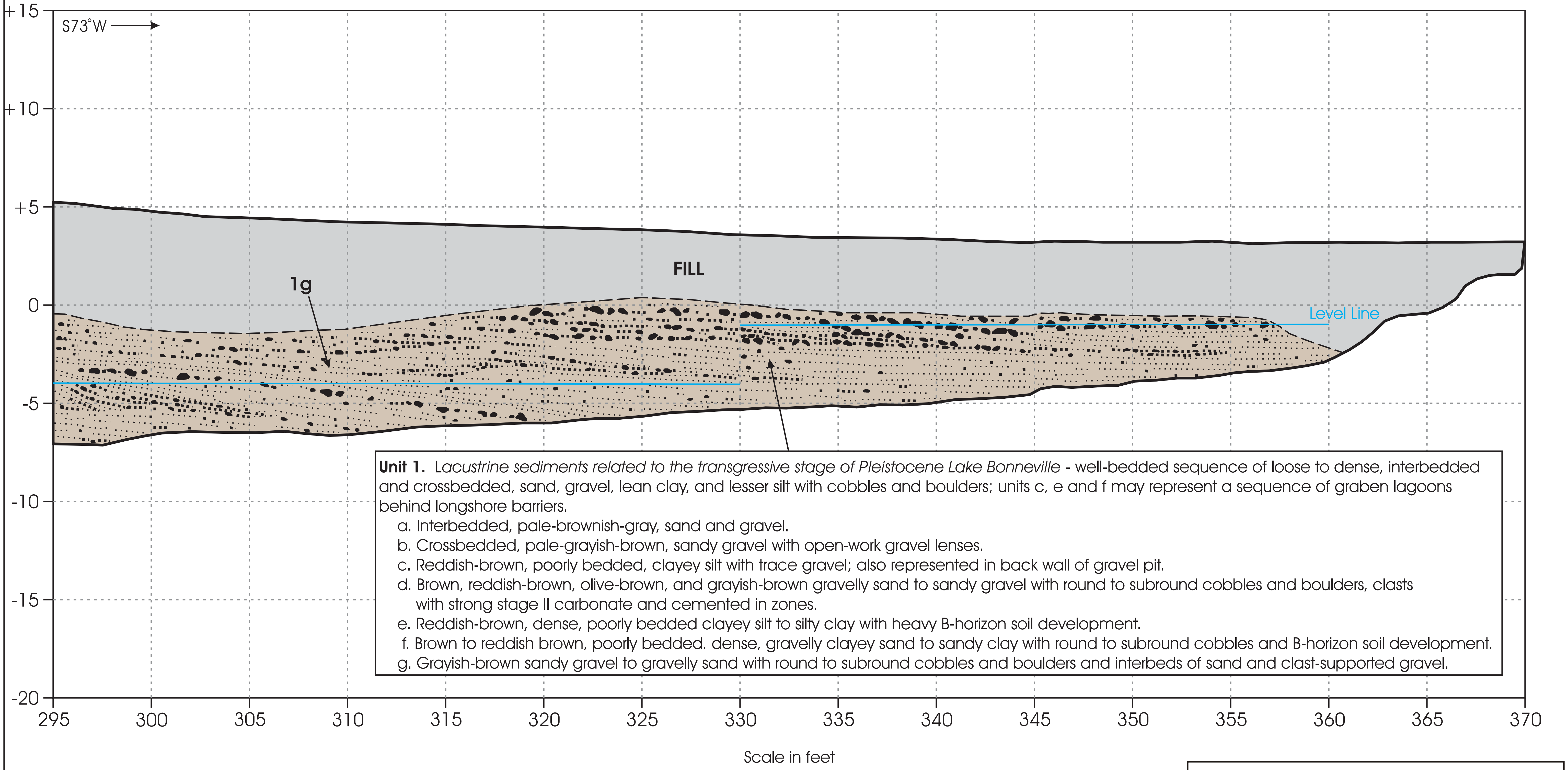
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

Trench logged by Bill Black, P.G. on
May 30-31, 2009

TRENCH 8 LOG, SHEET 4

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 12D



Unit 1. *Lacustrine sediments related to the transgressive stage of Pleistocene Lake Bonneville* - well-bedded sequence of loose to dense, interbedded and crossbedded, sand, gravel, lean clay, and lesser silt with cobbles and boulders; units c, e and f may represent a sequence of graben lagoons behind longshore barriers.

a. Interbedded, pale-brownish-gray, sand and gravel.
b. Crossbedded, pale-grayish-brown, sandy gravel with open-work gravel lenses.
c. Reddish-brown, poorly bedded, clayey silt with trace gravel; also represented in back wall of gravel pit.
d. Brown, reddish-brown, olive-brown, and grayish-brown gravelly sand to sandy gravel with round to subround cobbles and boulders, clasts with strong stage II carbonate and cemented in zones.
e. Reddish-brown, dense, poorly bedded clayey silt to silty clay with heavy B-horizon soil development.
f. Brown to reddish brown, poorly bedded. dense, gravelly clayey sand to sandy clay with round to subround cobbles and B-horizon soil development.
g. Grayish-brown sandy gravel to gravelly sand with round to subround cobbles and boulders and interbeds of sand and clast-supported gravel.

WESTERN
GEOLOGIC

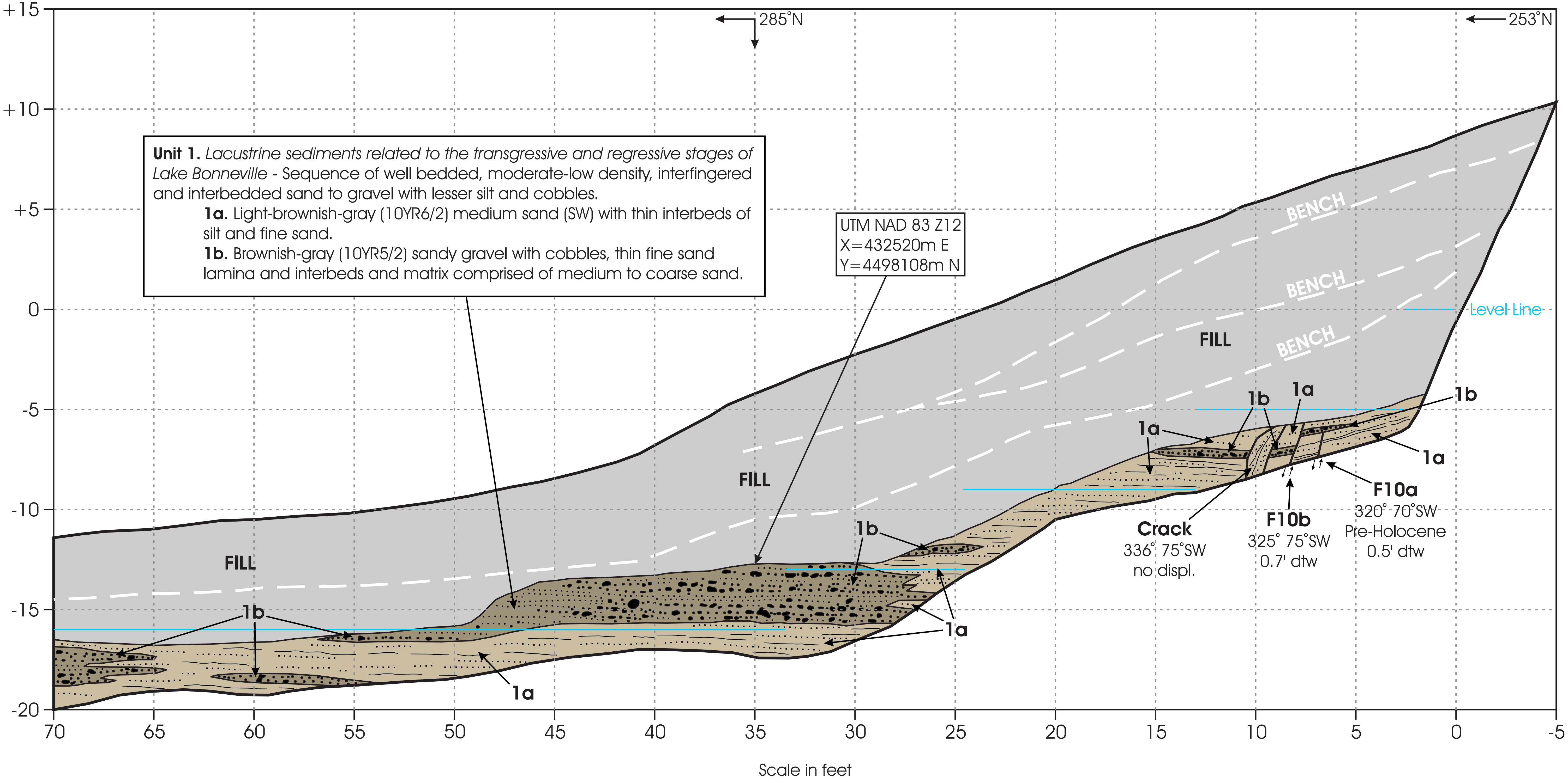
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Trench Wall Logged

Trench logged by Bill Black, P.G. on
May 30-31, 2009

TRENCH 8 LOG, SHEET 5

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 12E



Unit 1. *Lacustrine sediments related to the transgressive and regressive stages of Lake Bonneville* - Sequence of well bedded, moderate-low density, interfingered and interbedded sand to gravel with lesser silt and cobbles.

1a. Light-brownish-gray (10YR6/2) medium sand (SW) with thin interbeds of silt and fine sand.

1b. Brownish-gray (10YR5/2) sandy gravel with cobbles, thin fine sand lamina and interbeds and matrix comprised of medium to coarse sand.

UTM NAD 83 Z12
X=432520m E
Y=4498108m N

SCALE: 1 inch = 5 feet
(no vertical exaggeration)
North Trench Wall Logged

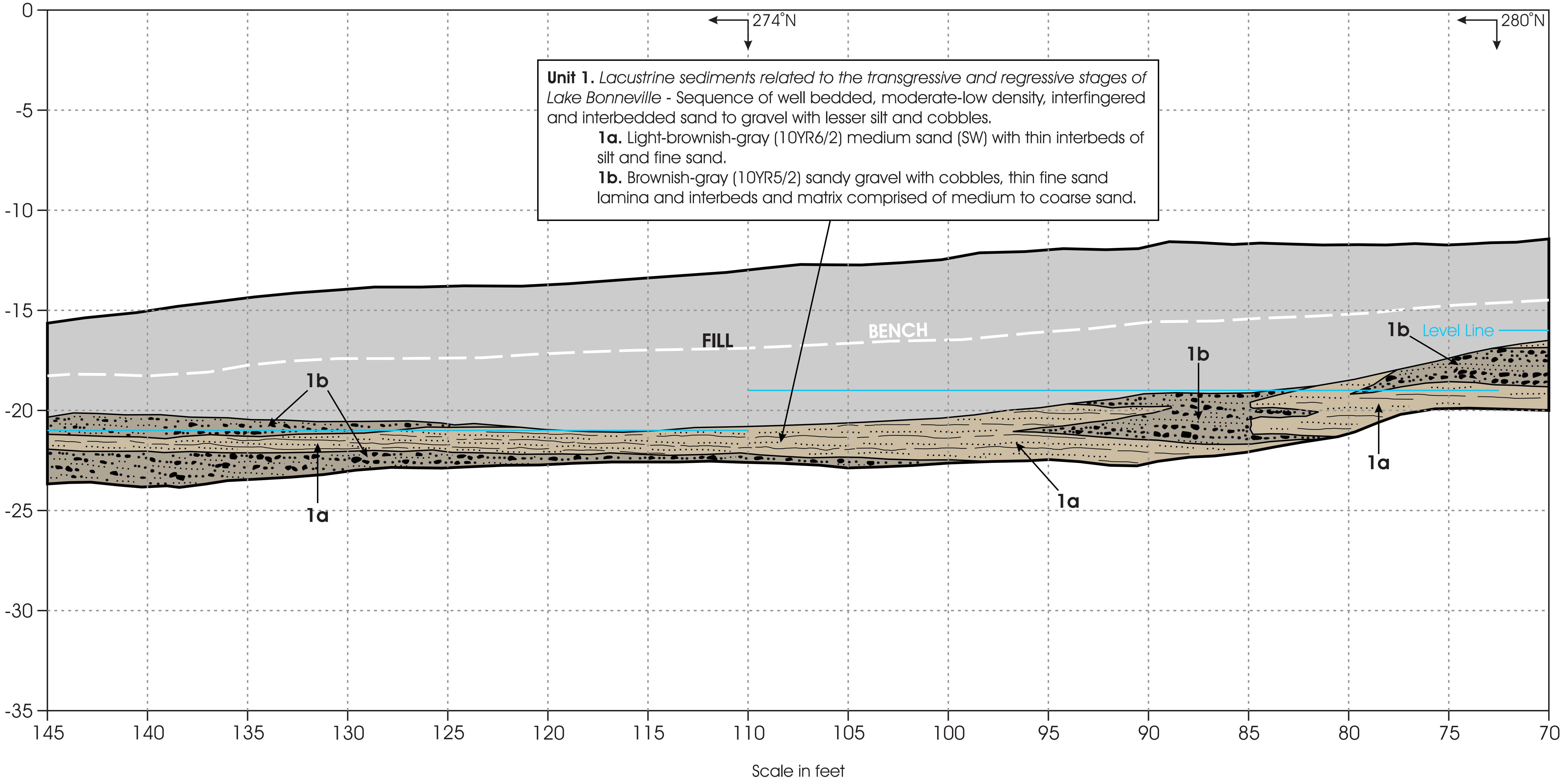
Trench logged by Bill Black, P.G. on
March 25-26, 2020

TRENCH 9 LOG, SHEET 1

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 13A

WESTERN GEOLOGIC



Unit 1. *Lacustrine sediments related to the transgressive and regressive stages of Lake Bonneville* - Sequence of well bedded, moderate-low density, interfingered and interbedded sand to gravel with lesser silt and cobbles.

- **1a.** Light-brownish-gray (10YR6/2) medium sand (SW) with thin interbeds of silt and fine sand.
- **1b.** Brownish-gray (10YR5/2) sandy gravel with cobbles, thin fine sand lamina and interbeds and matrix comprised of medium to coarse sand.

SCALE: 1 inch = 5 feet
(no vertical exaggeration)
North Trench Wall Logged

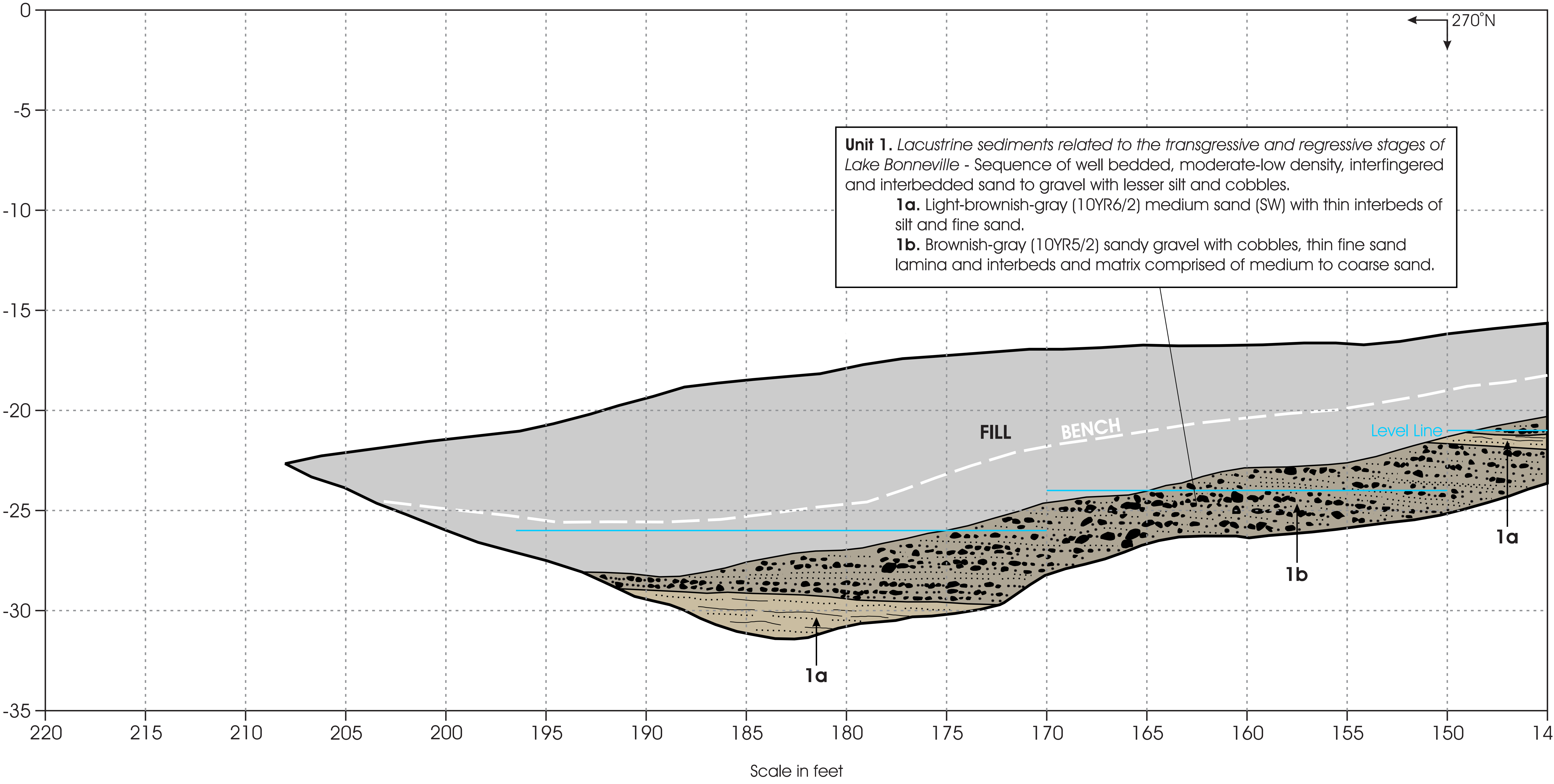
Trench logged by Bill Black, P.G. on
March 25-26, 2020

WESTERN
GEOLOGIC

TRENCH 9 LOG, SHEET 2

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 13B



Unit 1. *Lacustrine sediments related to the transgressive and regressive stages of Lake Bonneville* - Sequence of well bedded, moderate-low density, interfingered and interbedded sand to gravel with lesser silt and cobbles.

1a. Light-brownish-gray (10YR6/2) medium sand (SW) with thin interbeds of silt and fine sand.

1b. Brownish-gray (10YR5/2) sandy gravel with cobbles, thin fine sand lamina and interbeds and matrix comprised of medium to coarse sand.

WESTERN GEOLOGIC

SCALE: 1 inch = 5 feet
(no vertical exaggeration)
North Trench Wall Logged

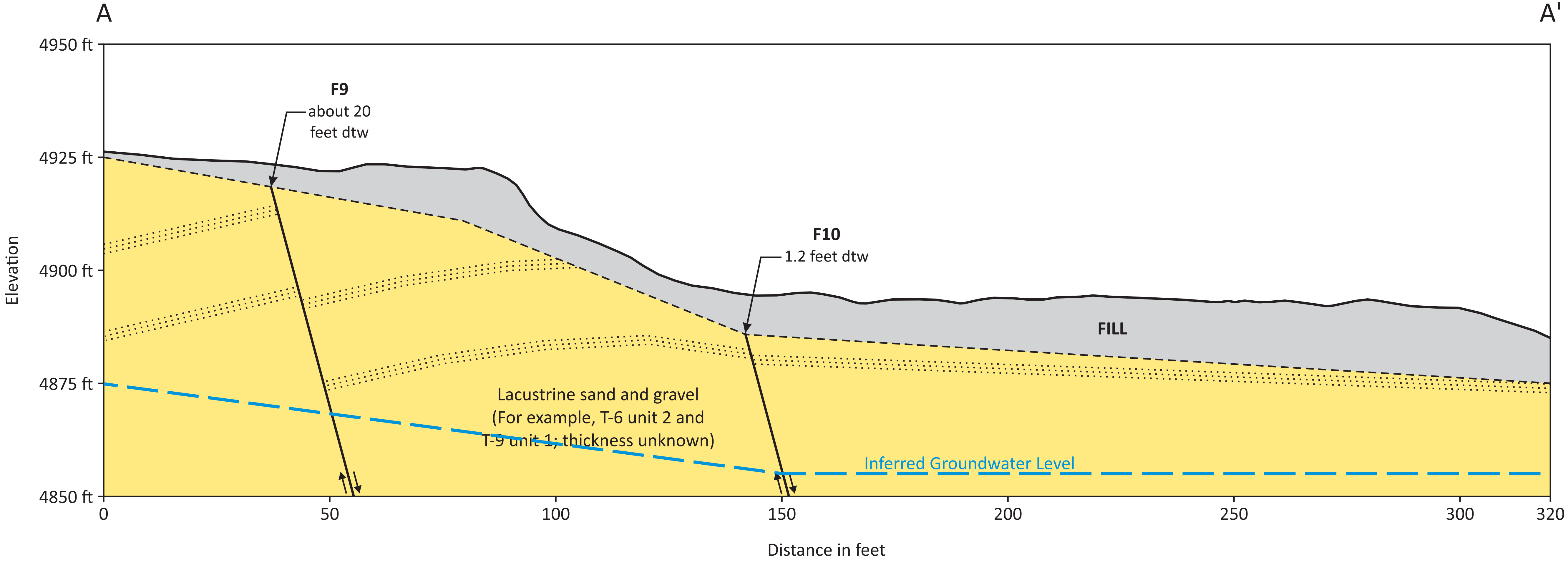
Trench logged by Bill Black, P.G. on
March 25-26, 2020

TRENCH 9 LOG, SHEET 3

GEOLOGIC HAZARDS EVALUATION

AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 13C



SCALE: 1 inch = 25 feet
(no vertical exaggeration)
East to West trending 228°N
Unit and textural contacts
are approximate and inferred

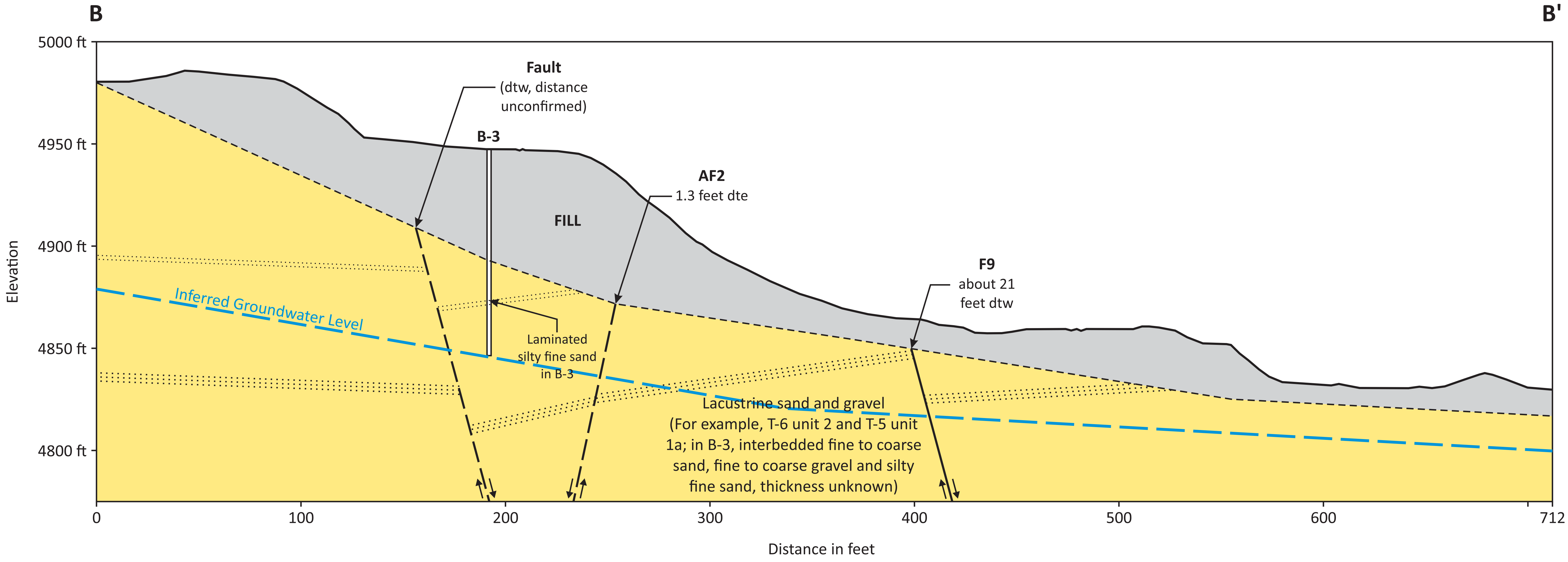
WESTERN GEOLOGIC

CROSS SECTION A-A'

GEOLOGIC HAZARDS EVALUATION

AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 14



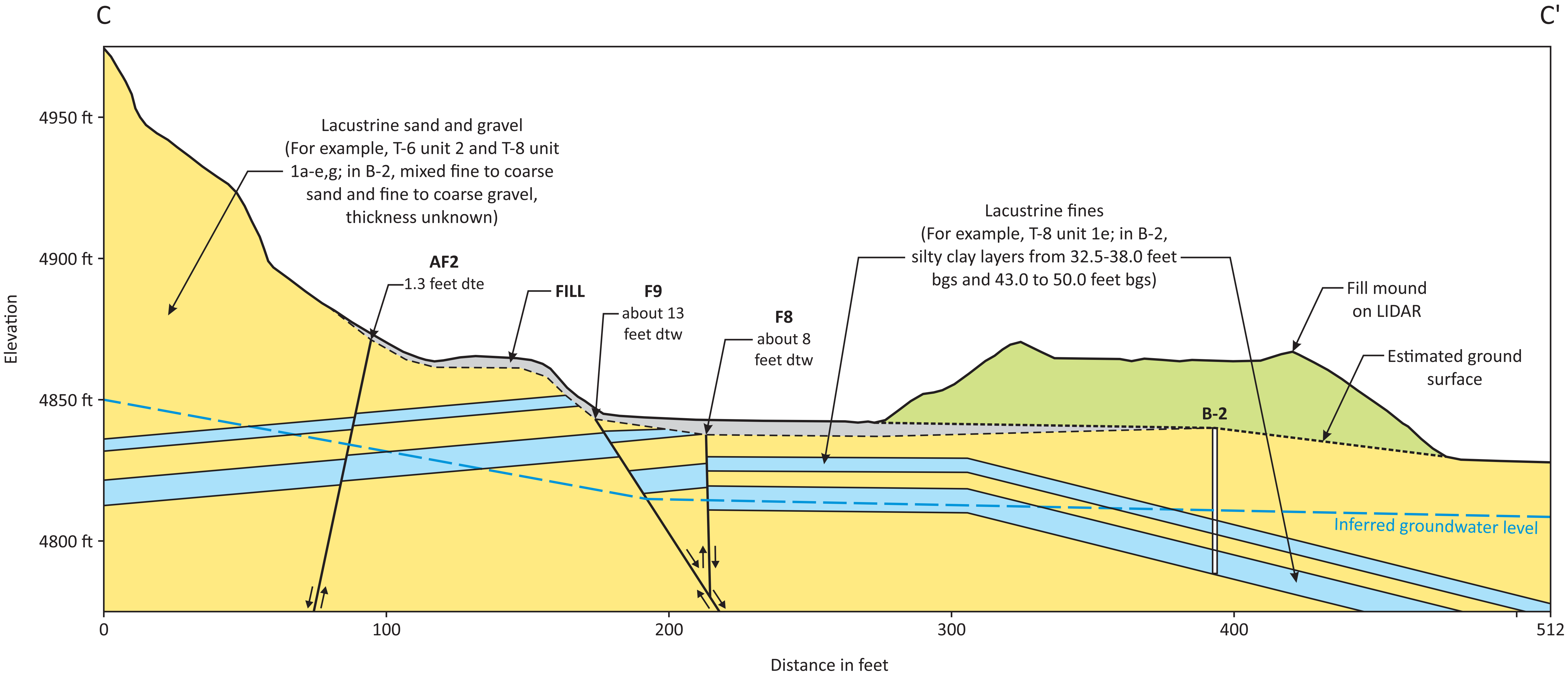
SCALE: 1 inch = 50 feet
(no vertical exaggeration)
East to West trending 226°N
Unit and textural contacts
are approximate and inferred

WESTERN GEOLOGIC

CROSS SECTION B-B'

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

FIGURE 15



SCALE: 1 inch = 40 feet
(no vertical exaggeration)
East to West trending 257°N
Unit and textural contacts
are approximate and inferred

CROSS SECTION C-C'

GEOLOGIC HAZARDS EVALUATION
AJ Rock LLC Property
6695 South Wasatch Boulevard
Cottonwood Heights, Utah

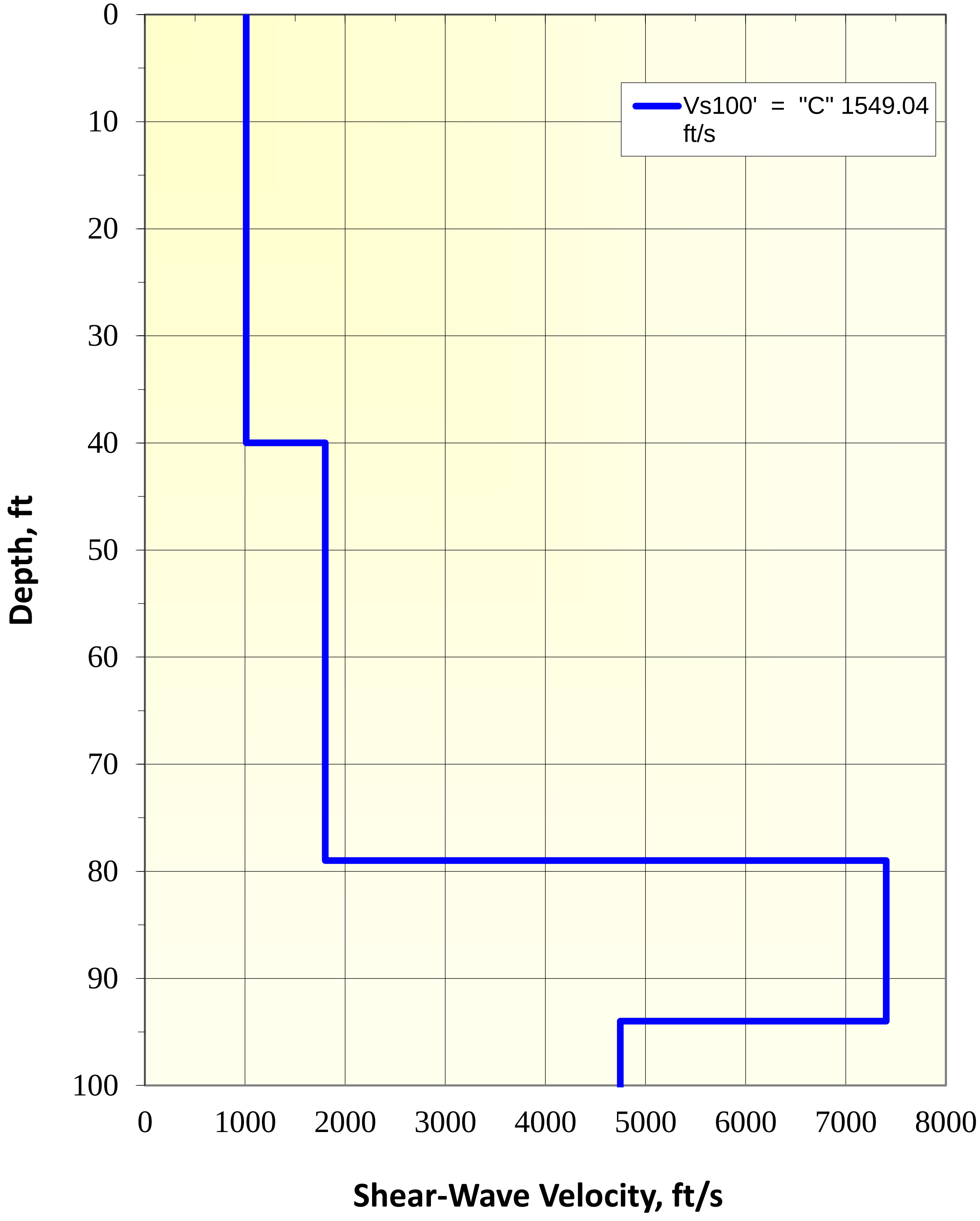
FIGURE 16

APPENDIX B

ReMi Survey Results

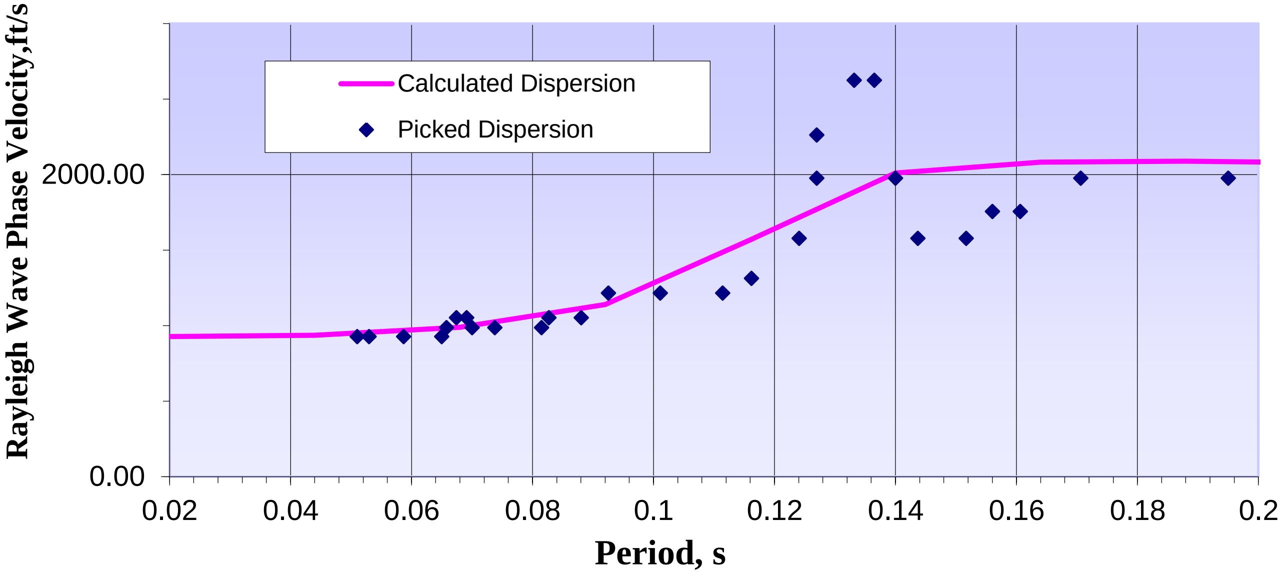
Shear Wave Velocity Profile

ReMi Line-01

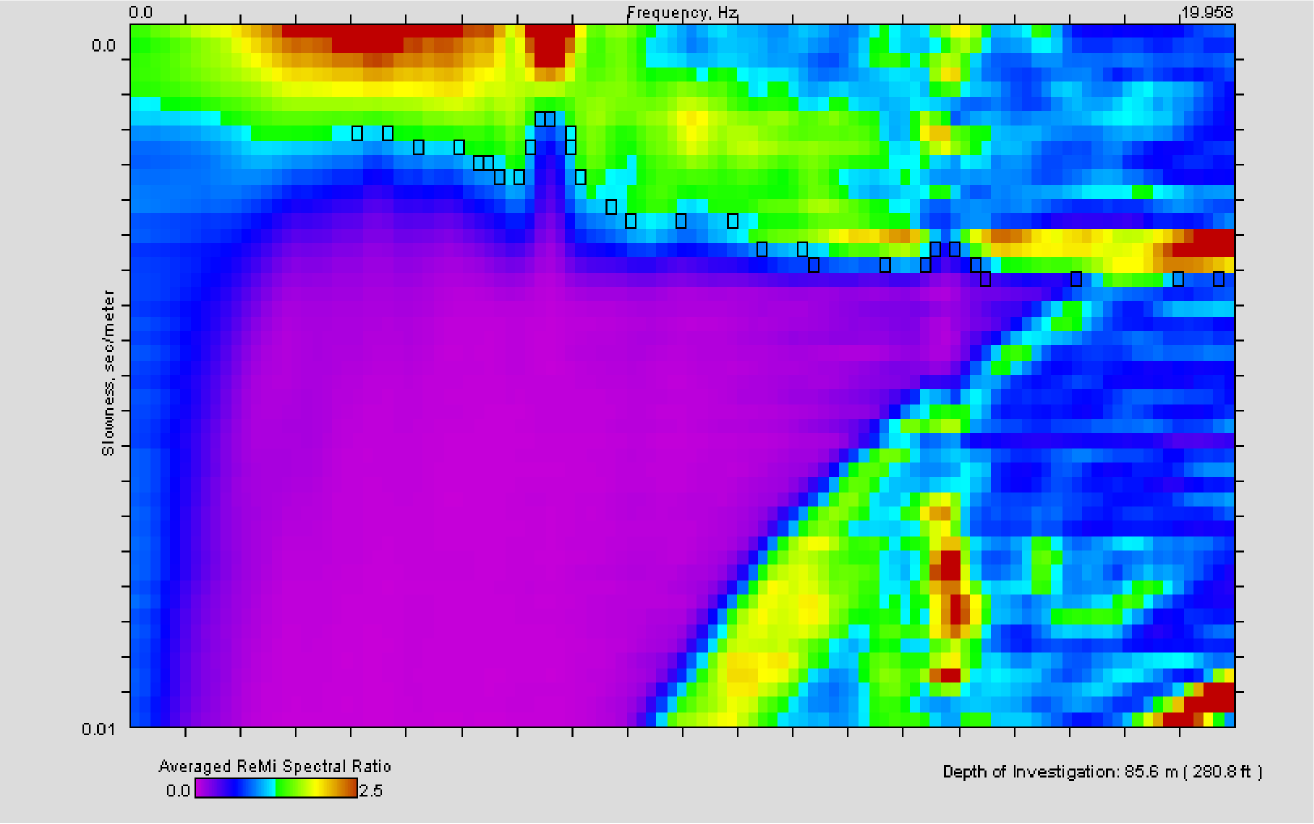


Dispersion Curve and Slowness Spectrum

Dispersion Curve Showing Picks and Fit



p-f Image with Dispersion Modeling Picks



APPENDIX C

Direct Shear Test Results

2B

Chain of Custody

IGES
Intermountain GeoEnvironmental Services Inc.
2702 South 1030 West, Suite 10, South Salt Lake, UT 84119
Phone: 801.270.9400 Fax: 801.270.9401

▨ **Required**

Company Name: Gordon Geotech Engineering
Project Number: 528-005-20
Address: 4426 Century Drive
Contact Name: Jordan Culp
Phone Number: 801-327-9600 Fax:
Email: Jordan@gordongeotech.com
Project Name: Gravel Pit Development
Location:
Client:

Special Instructions:
☑ Standard
☐ Rush Service. Number of Days: ______ Approved: ______

* Sample Type:		** Container Type	
S - Soil	C - Concrete	B - Bag	T - Tube
G - Geosynthetic	O - Other	Bk - Bucket	O - Other
R - Rock		J - Jar	

Sample Identification	Depth	Sample Type*	Container Type**	Analysis										Instructions/Comments
				Direct Shear										
B-2	35'	S	Ring	☒										2, 4, 8 KSF
B-2	40'	S	↓	☒										2, 4, 8 KSF
B-3	75'	S	↓	☒										2.5, 5, 10 KSF

Relinquished By: Jordan Culp
Date: 3-31-20
Time 13:43
☐ Hazardous Material
☐ Contaminated

Received By:
IGES Project Number: MA2106-015

Date: 3/31/2020
Time 1343
☐ Existing Client
☐ New Client

Results Sent By:
Date:
Time

Z:\FORMS\[CHAIN_OF_CUSTODY.xls]CHAIN

Page_____ of _____

Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Gordon Geotechnical Engineering
No: M02106-015 (528-005-20)
Location: Gravel Pit Development
Date: 4/9/2020
By: EH

Boring No.: B-2
Sample: 8
Depth: 35'
Sample Description: Grey clay
Sample type: Undisturbed-trimmed from ring

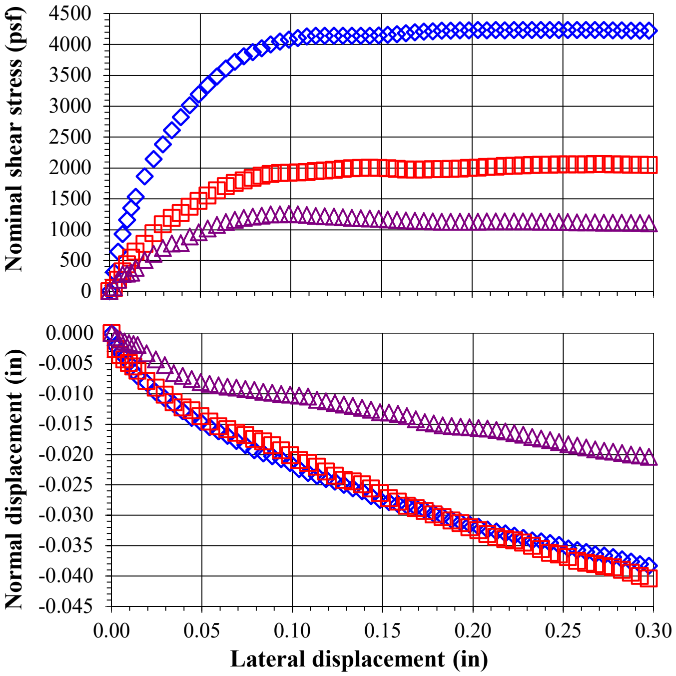
Test type: Inundated
Lateral displacement (in.): 0.3
Shear rate (in./min): 0.0005
Specific gravity, Gs: 2.70 Assumed

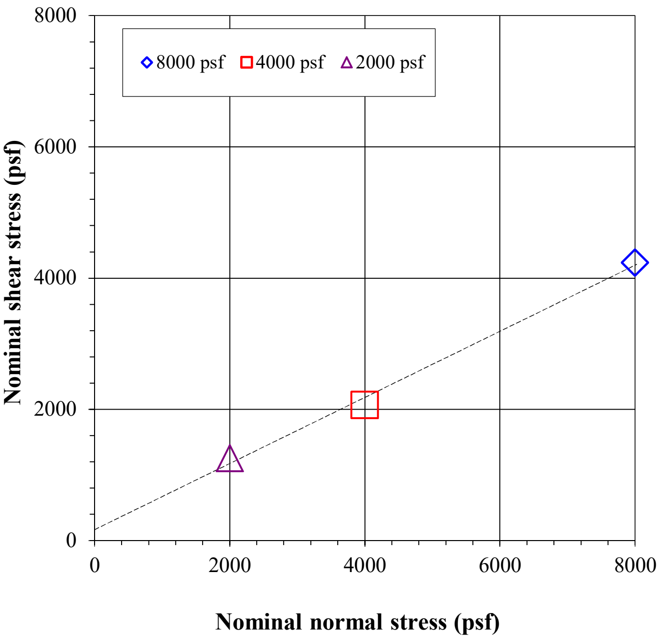
	Sample 1		Sample 2		Sample 3	
Nominal normal stress (psf)	8000		4000		2000	
Peak shear stress (psf)	4237		2061		1254	
Lateral displacement at peak (in)	0.238		0.273		0.099	
Load Duration (min)	2512		2512		2512	
	Initial	Pre-shear	Initial	Pre-shear	Initial	Pre-shear
Sample height (in)	0.994	0.918	0.997	0.950	0.994	0.951
Sample diameter (in)	2.416	2.416	2.413	2.413	2.417	2.417
Wt. rings + wet soil (g)	189.14	182.84	191.38	187.07	189.06	185.56
Wt. rings (g)	43.02	43.02	44.79	44.79	43.36	43.36
Wet soil + tare (g)	288.86		288.86		288.86	
Dry soil + tare (g)	250.46		250.46		250.46	
Tare (g)	121.68		121.68		121.68	
Water content (%)	29.8	24.2	29.8	26.0	29.8	26.7
Dry unit weight (pcf)	94.1	101.9	94.3	99.0	93.7	97.9
Void ratio, e, for assumed Gs	0.79	0.65	0.79	0.70	0.80	0.72
Saturation (%)*	101.7	100.0	102.4	100.0	100.9	100.0

ϕ' (deg)	27
c' (psf)	166

Average of 3 samples	Initial	Pre-shear
Water content (%)	29.8	25.6
Dry unit weight (pcf)	94.1	99.6

*Pre-shear saturation set to 100% for phase calculations





Comments:

Test specimens swelled at 100 psf load step.

Entered by:____________

Reviewed:____________

Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Gordon Geotechnical Engineering
No: M02106-015 (528-005-20)
Location: Gravel Pit Development

Boring No.: B-2
Sample: 8
Depth: 35'

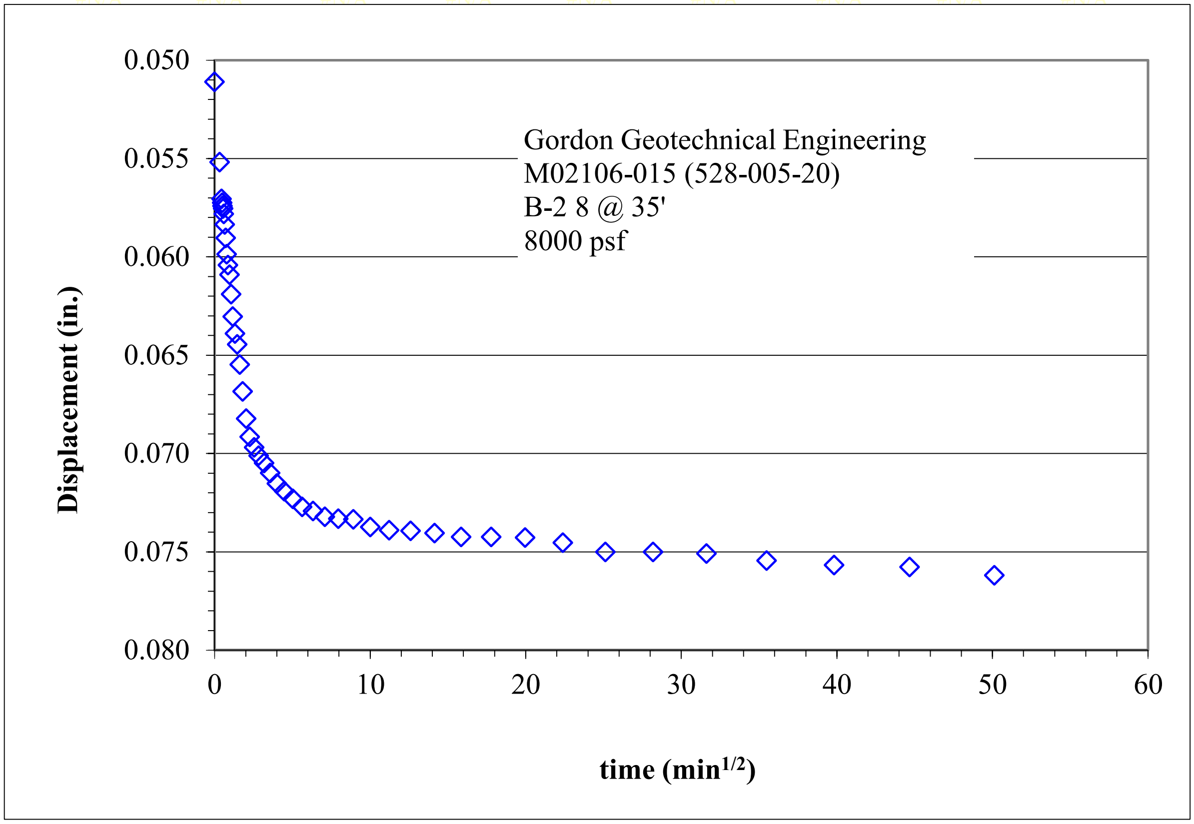
Nominal normal stress = 8000 psf			Nominal normal stress = 4000 psf			Nominal normal stress = 2000 psf		
Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)
0.000	0	0.000	0.000	0	0.000	0.000	0	0.000
0.002	312	-0.002	0.002	53	-0.003	0.002	65	-0.001
0.005	646	-0.003	0.005	187	-0.004	0.005	188	-0.001
0.007	928	-0.004	0.007	325	-0.004	0.007	277	-0.001
0.010	1161	-0.005	0.010	439	-0.005	0.010	286	-0.002
0.012	1351	-0.006	0.012	548	-0.006	0.012	304	-0.002
0.014	1530	-0.007	0.014	646	-0.006	0.014	379	-0.002
0.019	1863	-0.008	0.019	768	-0.008	0.019	507	-0.003
0.024	2142	-0.010	0.024	939	-0.009	0.024	609	-0.004
0.029	2380	-0.011	0.029	1085	-0.010	0.029	701	-0.005
0.034	2609	-0.012	0.034	1191	-0.011	0.034	773	-0.007
0.039	2824	-0.013	0.039	1269	-0.012	0.039	780	-0.007
0.044	3017	-0.014	0.044	1369	-0.013	0.044	889	-0.008
0.049	3191	-0.015	0.049	1466	-0.014	0.049	958	-0.008
0.054	3343	-0.015	0.054	1554	-0.015	0.054	1019	-0.008
0.059	3484	-0.016	0.059	1634	-0.015	0.059	1075	-0.009
0.064	3611	-0.017	0.064	1704	-0.016	0.064	1121	-0.009
0.069	3721	-0.018	0.069	1756	-0.016	0.069	1160	-0.009
0.074	3809	-0.018	0.074	1798	-0.017	0.074	1189	-0.009
0.079	3876	-0.019	0.079	1838	-0.017	0.079	1213	-0.009
0.084	3942	-0.020	0.084	1875	-0.018	0.084	1228	-0.010
0.089	4002	-0.020	0.089	1901	-0.018	0.089	1246	-0.010
0.094	4047	-0.021	0.094	1914	-0.019	0.094	1249	-0.010
0.099	4080	-0.021	0.099	1924	-0.020	0.099	1254	-0.010
0.104	4106	-0.022	0.104	1926	-0.021	0.104	1249	-0.010
0.109	4129	-0.023	0.109	1933	-0.021	0.109	1234	-0.010
0.114	4144	-0.023	0.114	1947	-0.022	0.114	1223	-0.011
0.119	4144	-0.024	0.119	1956	-0.023	0.119	1215	-0.011
0.124	4142	-0.024	0.124	1970	-0.024	0.124	1208	-0.012
0.129	4141	-0.025	0.129	1982	-0.024	0.129	1202	-0.012
0.134	4142	-0.026	0.134	1993	-0.024	0.134	1190	-0.012
0.139	4141	-0.026	0.139	1998	-0.025	0.139	1184	-0.012
0.144	4140	-0.027	0.144	2003	-0.026	0.144	1179	-0.013
0.148	4145	-0.027	0.148	1998	-0.026	0.148	1172	-0.013
0.153	4156	-0.028	0.153	1995	-0.027	0.153	1163	-0.013
0.158	4169	-0.028	0.158	1981	-0.028	0.158	1156	-0.013
0.163	4185	-0.028	0.163	1972	-0.028	0.163	1149	-0.014
0.168	4198	-0.029	0.168	1971	-0.029	0.168	1145	-0.014
0.173	4210	-0.029	0.173	1973	-0.029	0.173	1138	-0.015
0.178	4217	-0.030	0.178	1977	-0.030	0.178	1134	-0.015
0.183	4223	-0.030	0.183	1976	-0.030	0.183	1128	-0.015
0.188	4226	-0.031	0.188	1982	-0.031	0.188	1130	-0.015
0.193	4230	-0.031	0.193	1989	-0.031	0.193	1128	-0.015
0.198	4229	-0.032	0.198	1996	-0.032	0.198	1129	-0.016
0.203	4235	-0.032	0.203	2007	-0.032	0.203	1128	-0.016
0.208	4232	-0.032	0.208	2015	-0.033	0.208	1132	-0.016
0.213	4235	-0.033	0.213	2022	-0.034	0.213	1133	-0.016
0.218	4231	-0.033	0.218	2029	-0.034	0.218	1133	-0.016
0.223	4234	-0.033	0.223	2034	-0.034	0.223	1132	-0.016
0.228	4236	-0.034	0.228	2039	-0.035	0.228	1130	-0.017
0.233	4235	-0.034	0.233	2045	-0.035	0.233	1130	-0.017
0.238	4237	-0.034	0.238	2049	-0.036	0.238	1128	-0.017
0.243	4236	-0.035	0.243	2051	-0.036	0.243	1126	-0.018
0.248	4236	-0.035	0.248	2054	-0.036	0.248	1125	-0.018
0.253	4235	-0.035	0.253	2056	-0.037	0.253	1123	-0.018
0.258	4236	-0.036	0.258	2056	-0.038	0.258	1119	-0.019
0.263	4234	-0.036	0.263	2056	-0.038	0.263	1117	-0.019
0.268	4234	-0.036	0.268	2060	-0.038	0.268	1115	-0.019
0.273	4234	-0.037	0.273	2061	-0.038	0.273	1113	-0.019
0.278	4229	-0.037	0.278	2057	-0.039	0.278	1113	-0.020
0.282	4227	-0.037	0.282	2057	-0.039	0.282	1110	-0.020
0.287	4227	-0.038	0.287	2052	-0.039	0.287	1109	-0.020
0.292	4226	-0.038	0.292	2052	-0.040	0.292	1104	-0.020
0.297	4225	-0.038	0.297	2047	-0.040	0.297	1102	-0.020
0.300	4225	-0.038	0.300	2046	-0.041	0.300	1102	-0.020

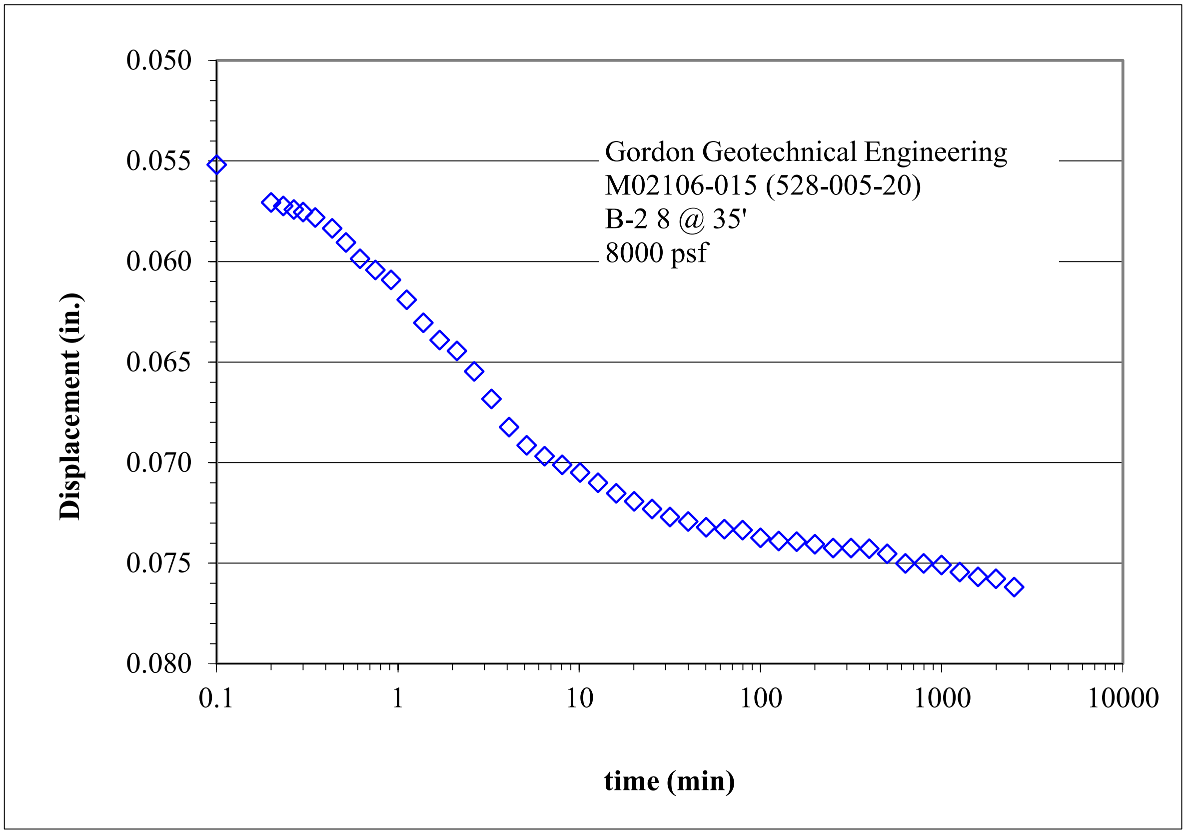
Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Gordon Geotechnical Engineering
No: M02106-015 (528-005-20)
Location: Gravel Pit Development

Boring No.: B-2
Sample: 8
Depth: 35'





Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: **Gordon Geotechnical Engineering**
No: **M02106-015 (528-005-20)**
Location: Gravel Pit Development
Date: 4/9/2020
By: EH

Boring No.: **B-2**
Sample: **9**
Depth: **40'**
Sample Description: Brown sand with clay
Sample type: Undisturbed-trimmed from ring

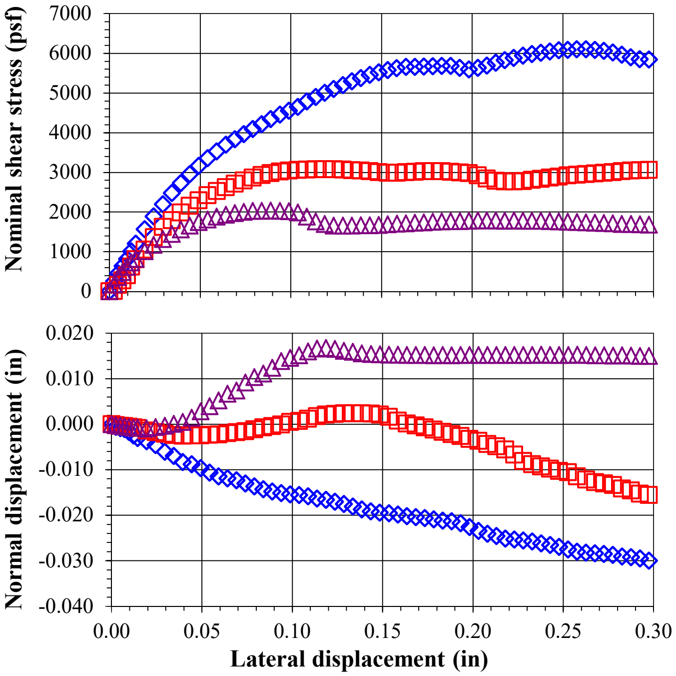
Test type: Inundated
Lateral displacement (in.): 0.3
Shear rate (in./min): 0.0010
Specific gravity, Gs: 2.70 Assumed

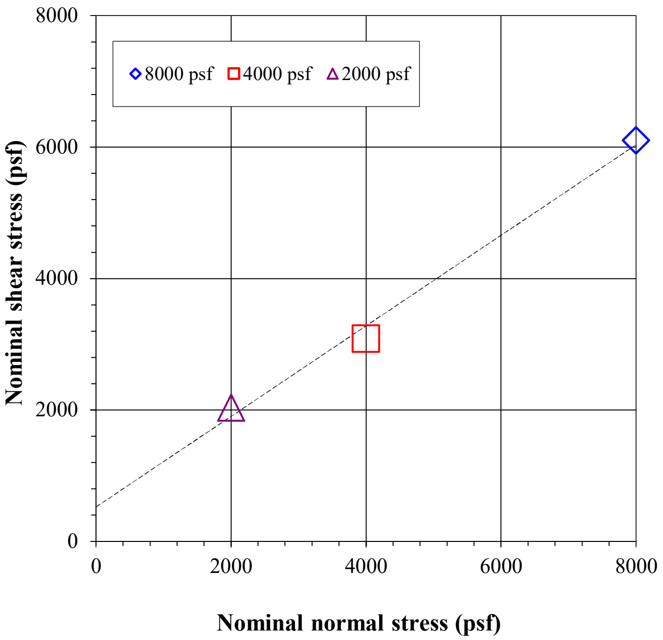
	Sample 1		Sample 2		Sample 3	
Nominal normal stress (psf)	8000		4000		2000	
Peak shear stress (psf)	6103		3080		2035	
Lateral displacement at peak (in)	0.258		0.114		0.089	
Load Duration (min)	1000		1000		1000	
	Initial	Pre-shear	Initial	Pre-shear	Initial	Pre-shear
Sample height (in)	0.992	0.944	0.994	0.966	0.997	0.974
Sample diameter (in)	2.422	2.422	2.414	2.414	2.417	2.417
Wt. rings + wet soil (g)	189.34	189.35	196.76	195.67	197.52	196.65
Wt. rings (g)	41.52	41.52	44.18	44.18	43.77	43.77
Wet soil + tare (g)	316.53		316.53		316.53	
Dry soil + tare (g)	282.93		282.93		282.93	
Tare (g)	127.31		127.31		127.31	
Water content (%)	21.6	21.6	21.6	20.7	21.6	20.9
Dry unit weight (pcf)	101.3	106.4	105.1	108.0	105.3	107.7
Void ratio, e, for assumed Gs	0.66	0.58	0.60	0.56	0.60	0.56
Saturation (%)*	87.9	100.0	96.5	100.0	97.1	100.0

ϕ' (deg)	35
c' (psf)	524

Average of 3 samples	Initial	Pre-shear
Water content (%)	21.6	21.1
Dry unit weight (pcf)	103.9	107.4

*Pre-shear saturation set to 100% for phase calculations





Entered by:___________
Reviewed:___________

Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Gordon Geotechnical Engineering
No: M02106-015 (528-005-20)
Location: Gravel Pit Development

Boring No.: B-2
Sample: 9
Depth: 40'

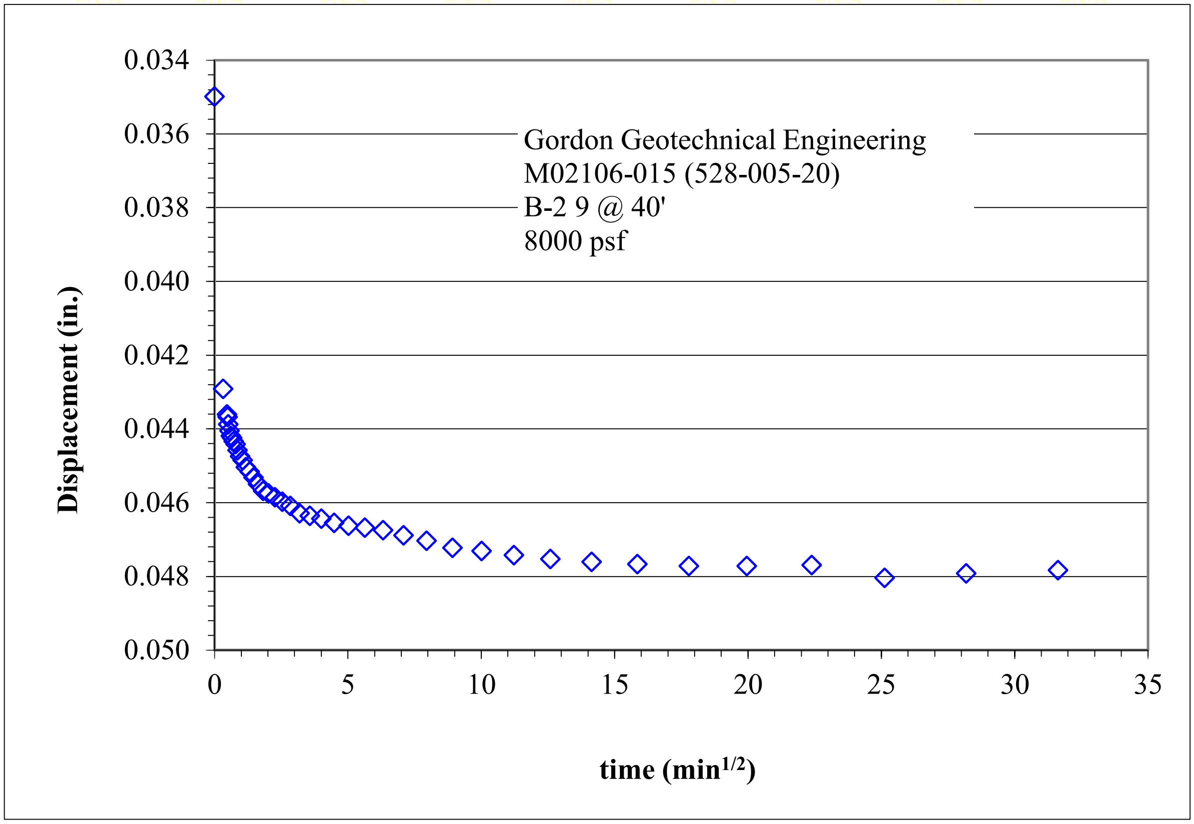
Nominal normal stress = 8000 psf			Nominal normal stress = 4000 psf			Nominal normal stress = 2000 psf		
Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)
0.000	0	0.000	0.000	0	0.000	0.000	0	0.000
0.002	253	-0.001	0.002	7	0.000	0.002	174	0.000
0.005	455	-0.001	0.005	164	0.000	0.005	356	0.000
0.007	639	-0.001	0.007	274	0.000	0.007	489	0.000
0.010	818	-0.002	0.010	408	-0.001	0.010	616	-0.001
0.012	1011	-0.002	0.012	620	-0.001	0.012	726	-0.001
0.014	1207	-0.003	0.014	806	-0.001	0.014	820	-0.001
0.019	1565	-0.004	0.019	1048	-0.001	0.019	1008	-0.001
0.024	1887	-0.005	0.024	1369	-0.002	0.024	1173	-0.001
0.029	2195	-0.006	0.029	1623	-0.002	0.029	1320	0.000
0.034	2477	-0.007	0.034	1793	-0.002	0.034	1449	0.000
0.039	2732	-0.008	0.039	1984	-0.003	0.039	1559	0.001
0.044	2960	-0.009	0.044	2154	-0.002	0.044	1657	0.001
0.049	3166	-0.010	0.049	2293	-0.002	0.049	1742	0.003
0.054	3359	-0.011	0.054	2423	-0.002	0.054	1814	0.004
0.059	3532	-0.011	0.059	2531	-0.002	0.059	1879	0.005
0.064	3698	-0.012	0.064	2625	-0.002	0.064	1931	0.007
0.069	3838	-0.012	0.069	2712	-0.002	0.069	1974	0.007
0.074	3967	-0.013	0.074	2798	-0.002	0.074	2001	0.009
0.079	4097	-0.014	0.079	2868	-0.001	0.079	2028	0.010
0.084	4219	-0.014	0.084	2932	-0.001	0.084	2027	0.011
0.089	4341	-0.015	0.089	2983	0.000	0.089	2035	0.012
0.094	4457	-0.015	0.094	3018	0.000	0.094	2030	0.014
0.099	4557	-0.015	0.099	3041	0.000	0.099	2018	0.015
0.104	4654	-0.016	0.104	3063	0.001	0.104	1986	0.015
0.109	4786	-0.016	0.109	3068	0.001	0.109	1883	0.016
0.114	4900	-0.016	0.114	3080	0.002	0.114	1765	0.017
0.119	5004	-0.017	0.119	3078	0.002	0.119	1698	0.017
0.124	5104	-0.017	0.124	3075	0.002	0.124	1659	0.017
0.129	5201	-0.017	0.129	3070	0.002	0.129	1655	0.016
0.134	5293	-0.018	0.134	3056	0.002	0.134	1655	0.016
0.139	5378	-0.019	0.139	3043	0.002	0.139	1662	0.016
0.144	5457	-0.019	0.144	3028	0.002	0.144	1668	0.015
0.148	5524	-0.019	0.148	3004	0.002	0.148	1677	0.015
0.153	5581	-0.020	0.153	2986	0.002	0.153	1688	0.015
0.158	5626	-0.020	0.158	2987	0.001	0.158	1703	0.015
0.163	5662	-0.020	0.163	3002	0.000	0.163	1715	0.015
0.168	5657	-0.020	0.168	3012	0.000	0.168	1730	0.015
0.173	5666	-0.021	0.173	3028	-0.001	0.173	1740	0.015
0.178	5675	-0.021	0.178	3032	-0.001	0.178	1753	0.015
0.183	5688	-0.021	0.183	3029	-0.002	0.183	1761	0.015
0.188	5684	-0.021	0.188	3016	-0.002	0.188	1767	0.015
0.193	5639	-0.022	0.193	2998	-0.003	0.193	1773	0.015
0.198	5593	-0.023	0.198	2978	-0.003	0.198	1779	0.015
0.203	5633	-0.023	0.203	2924	-0.004	0.203	1782	0.015
0.208	5689	-0.024	0.208	2846	-0.004	0.208	1783	0.015
0.213	5762	-0.025	0.213	2789	-0.005	0.213	1784	0.015
0.218	5827	-0.025	0.218	2767	-0.006	0.218	1782	0.015
0.223	5887	-0.025	0.223	2770	-0.007	0.223	1783	0.015
0.228	5941	-0.026	0.228	2782	-0.008	0.228	1780	0.015
0.233	5981	-0.026	0.233	2812	-0.009	0.233	1777	0.015
0.238	6019	-0.026	0.238	2839	-0.009	0.238	1775	0.015
0.243	6050	-0.027	0.243	2865	-0.010	0.243	1769	0.015
0.248	6076	-0.027	0.248	2892	-0.010	0.248	1763	0.015
0.253	6087	-0.028	0.253	2920	-0.011	0.253	1756	0.015
0.258	6103	-0.028	0.258	2938	-0.011	0.258	1747	0.015
0.263	6100	-0.028	0.263	2957	-0.012	0.263	1737	0.015
0.268	6091	-0.028	0.268	2971	-0.013	0.268	1730	0.015
0.273	6064	-0.029	0.273	2990	-0.013	0.273	1720	0.015
0.277	6026	-0.029	0.278	3008	-0.013	0.278	1710	0.015
0.282	5970	-0.029	0.282	3027	-0.014	0.283	1701	0.015
0.287	5905	-0.029	0.287	3041	-0.015	0.287	1697	0.015
0.292	5856	-0.029	0.292	3049	-0.015	0.292	1687	0.015
0.297	5849	-0.030	0.297	3063	-0.016	0.297	1679	0.015
0.300	5852	-0.030	0.300	3066	-0.016	0.300	1676	0.015

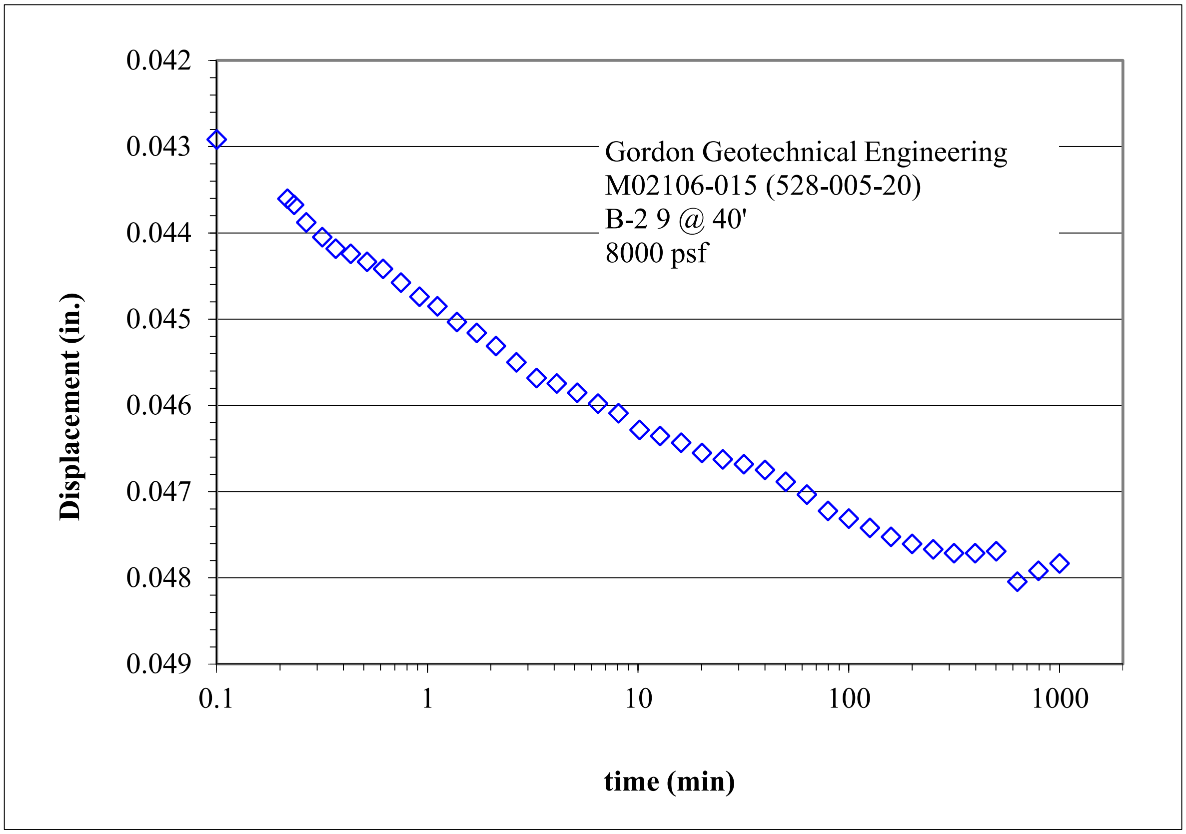
Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Gordon Geotechnical Engineering
No: M02106-015 (528-005-20)
Location: Gravel Pit Development

Boring No.: B-2
Sample: 9
Depth: 40'





Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

IGES

Project: Gordon Geotechnical Engineering
No: M02106-015 (528-005-20)
Location: Gravel Pit Development
Date: 4/9/2020
By: EH

Boring No.: B-3
Sample:
Depth: 75'
Sample Description: Brown clayey sand
Sample type: Undisturbed-trimmed from ring

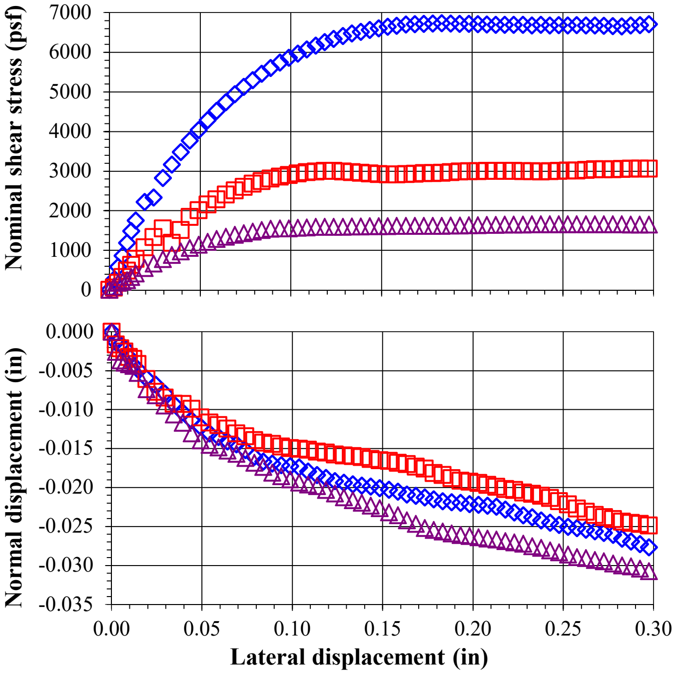
Test type: Inundated
Lateral displacement (in.): 0.3
Shear rate (in./min): 0.0009
Specific gravity, Gs: 2.70 Assumed

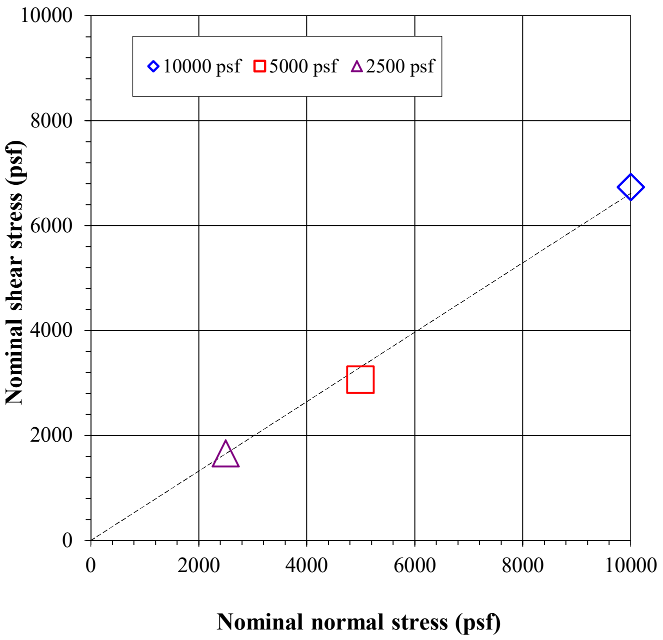
	Sample 1		Sample 2		Sample 3	
Nominal normal stress (psf)	10000		5000		2500	
Peak shear stress (psf)	6735		3062		1659	
Lateral displacement at peak (in)	0.183		0.292		0.268	
Load Duration (min)	1995		1995		1995	
	Initial	Pre-shear	Initial	Pre-shear	Initial	Pre-shear
Sample height (in)	0.995	0.935	0.996	0.938	0.986	0.938
Sample diameter (in)	2.413	2.413	2.414	2.414	2.417	2.417
Wt. rings + wet soil (g)	173.81	187.68	176.37	188.98	179.82	190.10
Wt. rings (g)	44.10	44.10	43.05	43.05	40.73	40.73
Wet soil + tare (g)	264.29		264.29		264.29	
Dry soil + tare (g)	250.55		250.55		250.55	
Tare (g)	127.02		127.02		127.02	
Water content (%)	11.1	23.0	11.1	21.6	11.1	19.3
Dry unit weight (pcf)	97.7	103.9	100.3	106.4	105.4	110.7
Void ratio, e, for assumed Gs	0.72	0.62	0.68	0.58	0.60	0.52
Saturation (%)*	41.4	100.0	44.1	100.0	50.1	100.0

ϕ' (deg)	33
c' (psf)	0

Average of 3 samples	Initial	Pre-shear
Water content (%)	11.1	21.3
Dry unit weight (pcf)	101.1	107.0

*Pre-shear saturation set to 100% for phase calculations





Comments:
Test specimens #1 and #2 contain vertical clay seam.

Entered by:___________
Reviewed:___________

Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Gordon Geotechnical Engineering
No: M02106-015 (528-005-20)
Location: Gravel Pit Development

Boring No.: B-3
Sample:
Depth: 75'

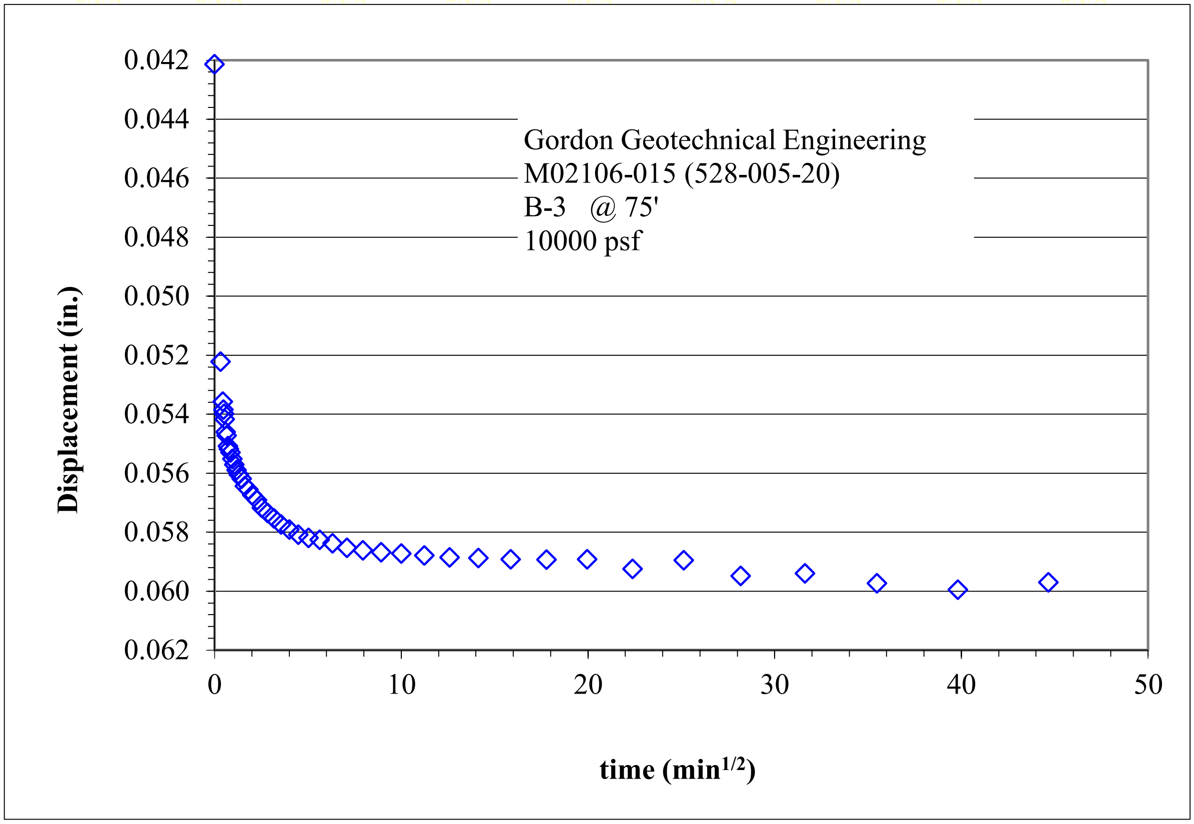
Nominal normal stress = 10000 psf			Nominal normal stress = 5000 psf			Nominal normal stress = 2500 psf		
Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)
0.000	0	0.000	0.000	0	0.000	0.000	0	0.000
0.002	257	-0.001	0.002	58	-0.002	0.002	84	-0.003
0.005	586	-0.002	0.005	180	-0.002	0.005	179	-0.004
0.007	863	-0.002	0.007	326	-0.003	0.007	207	-0.004
0.010	1188	-0.003	0.010	484	-0.003	0.010	246	-0.004
0.012	1484	-0.004	0.012	631	-0.004	0.012	321	-0.004
0.014	1755	-0.005	0.014	786	-0.004	0.014	410	-0.005
0.019	2223	-0.006	0.019	1076	-0.006	0.019	552	-0.007
0.024	2338	-0.007	0.024	1331	-0.008	0.024	667	-0.008
0.029	2824	-0.008	0.029	1551	-0.008	0.029	779	-0.009
0.034	3169	-0.009	0.034	1186	-0.009	0.034	882	-0.011
0.039	3482	-0.010	0.039	1513	-0.009	0.039	974	-0.012
0.044	3769	-0.011	0.044	1843	-0.010	0.044	1062	-0.013
0.049	4037	-0.012	0.049	2011	-0.011	0.049	1141	-0.014
0.054	4289	-0.013	0.054	2159	-0.012	0.054	1215	-0.015
0.059	4521	-0.014	0.059	2288	-0.012	0.059	1284	-0.015
0.064	4741	-0.014	0.064	2407	-0.012	0.064	1343	-0.015
0.069	4941	-0.014	0.069	2510	-0.013	0.069	1390	-0.016
0.074	5123	-0.015	0.074	2601	-0.013	0.074	1433	-0.016
0.079	5296	-0.016	0.079	2680	-0.014	0.079	1474	-0.017
0.084	5460	-0.017	0.084	2750	-0.014	0.084	1508	-0.017
0.089	5601	-0.017	0.089	2808	-0.014	0.089	1540	-0.018
0.094	5737	-0.017	0.094	2858	-0.015	0.094	1553	-0.019
0.099	5859	-0.017	0.099	2898	-0.015	0.099	1554	-0.019
0.104	5970	-0.017	0.104	2937	-0.015	0.104	1564	-0.019
0.109	6076	-0.018	0.109	2969	-0.015	0.109	1580	-0.020
0.114	6172	-0.018	0.114	2992	-0.015	0.114	1589	-0.020
0.119	6257	-0.019	0.119	3001	-0.016	0.119	1589	-0.020
0.124	6340	-0.019	0.124	3001	-0.016	0.124	1589	-0.021
0.129	6410	-0.019	0.129	2992	-0.016	0.129	1595	-0.021
0.134	6472	-0.020	0.134	2975	-0.016	0.134	1598	-0.022
0.139	6514	-0.020	0.139	2960	-0.016	0.139	1602	-0.022
0.144	6565	-0.020	0.144	2944	-0.016	0.144	1609	-0.022
0.148	6601	-0.020	0.148	2925	-0.017	0.148	1609	-0.023
0.153	6642	-0.020	0.153	2919	-0.017	0.153	1612	-0.023
0.158	6667	-0.021	0.158	2920	-0.017	0.158	1612	-0.024
0.163	6699	-0.021	0.163	2922	-0.017	0.163	1614	-0.024
0.168	6710	-0.021	0.168	2929	-0.017	0.168	1616	-0.025
0.173	6719	-0.021	0.173	2944	-0.018	0.173	1615	-0.025
0.178	6723	-0.022	0.178	2952	-0.018	0.178	1615	-0.025
0.183	6735	-0.022	0.183	2958	-0.018	0.183	1619	-0.026
0.188	6724	-0.022	0.188	2974	-0.019	0.188	1622	-0.026
0.193	6719	-0.022	0.193	2987	-0.019	0.193	1624	-0.026
0.198	6715	-0.022	0.198	2999	-0.019	0.198	1634	-0.026
0.203	6711	-0.022	0.203	2999	-0.019	0.203	1634	-0.026
0.208	6698	-0.022	0.208	3008	-0.020	0.208	1641	-0.027
0.213	6692	-0.022	0.213	3007	-0.020	0.213	1642	-0.027
0.218	6693	-0.023	0.218	3009	-0.020	0.218	1647	-0.027
0.223	6690	-0.023	0.223	3004	-0.020	0.223	1650	-0.027
0.228	6692	-0.024	0.228	2997	-0.021	0.228	1653	-0.027
0.233	6679	-0.024	0.233	2993	-0.021	0.233	1654	-0.028
0.238	6689	-0.024	0.238	2992	-0.021	0.238	1657	-0.028
0.243	6678	-0.025	0.243	2994	-0.021	0.243	1656	-0.028
0.248	6682	-0.025	0.248	3002	-0.022	0.248	1657	-0.028
0.253	6679	-0.025	0.253	3013	-0.022	0.253	1655	-0.029
0.258	6682	-0.025	0.258	3017	-0.023	0.258	1658	-0.029
0.263	6676	-0.025	0.263	3033	-0.023	0.263	1657	-0.029
0.268	6678	-0.026	0.268	3042	-0.024	0.268	1659	-0.029
0.273	6672	-0.026	0.273	3048	-0.024	0.273	1655	-0.029
0.278	6659	-0.026	0.278	3047	-0.024	0.278	1657	-0.030
0.282	6655	-0.027	0.282	3052	-0.024	0.282	1654	-0.030
0.287	6675	-0.027	0.287	3058	-0.025	0.287	1649	-0.030
0.292	6687	-0.027	0.292	3062	-0.025	0.292	1648	-0.030
0.297	6712	-0.028	0.297	3059	-0.025	0.297	1648	-0.031
0.300	6729	-0.028	0.300	3062	-0.025	0.300	1649	-0.031

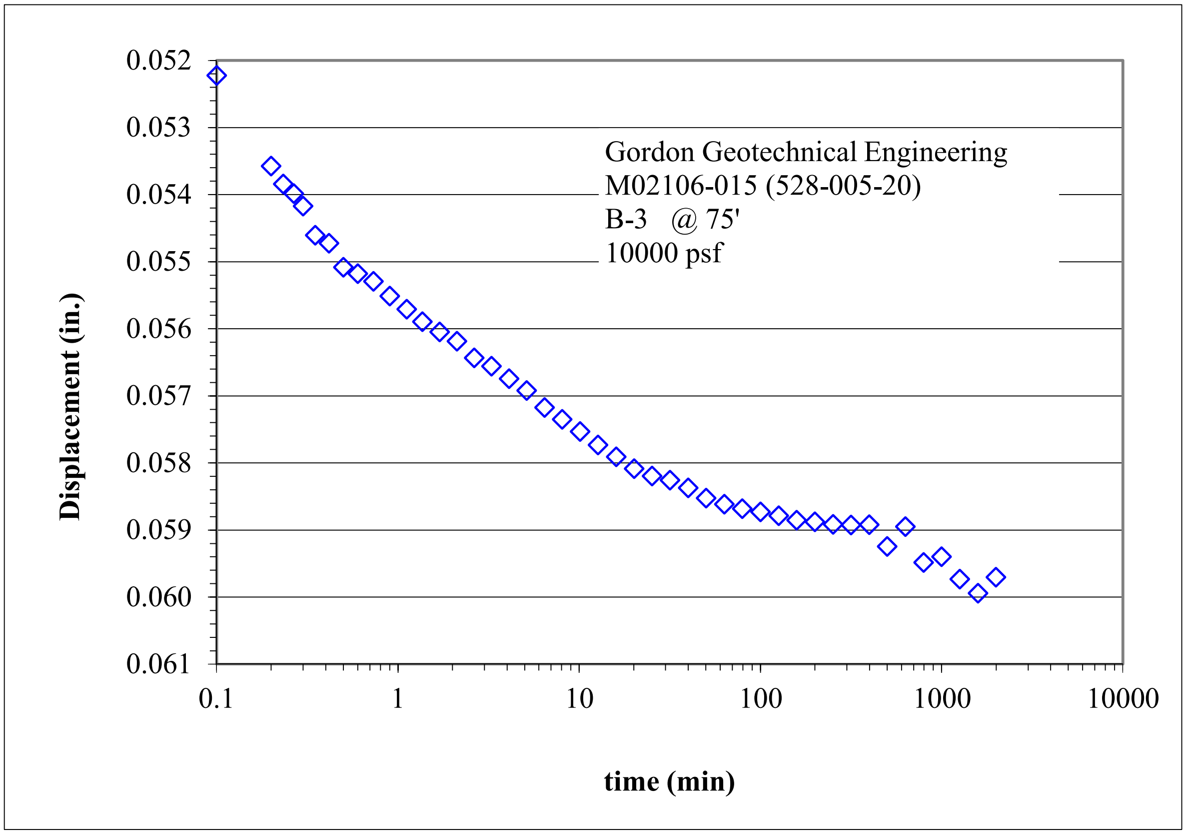
Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Gordon Geotechnical Engineering
No: M02106-015 (528-005-20)
Location: Gravel Pit Development

Boring No.: B-3
Sample:
Depth: 75'





Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

IGES

Project: **Gordon Geotechnical Engineering**
No: **M02106-006 (528-002-18)**
Location: View 62
Date: 4/2/2018
By: JDF

Boring No.:
Sample: **Sample C**
Depth:
Sample Description: Brown sand
Sample type: Laboratory compacted
Dry unit weight 120.5 pcf
at 6.5 (%) w
Compaction specifications: 95% of $\gamma_d$max

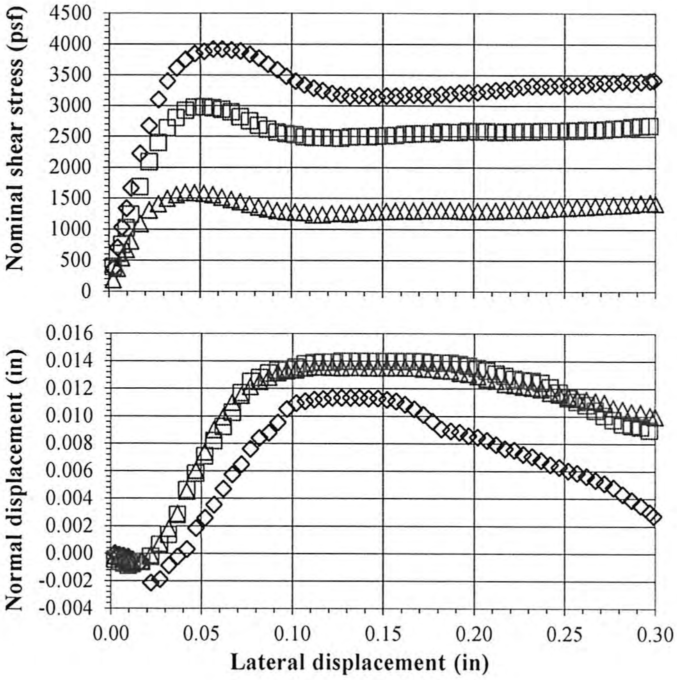
Test type: Inundated
Lateral displacement (in.): 0.3
Shear rate (in./min): 0.0086
Specific gravity, Gs: 2.65 Assumed

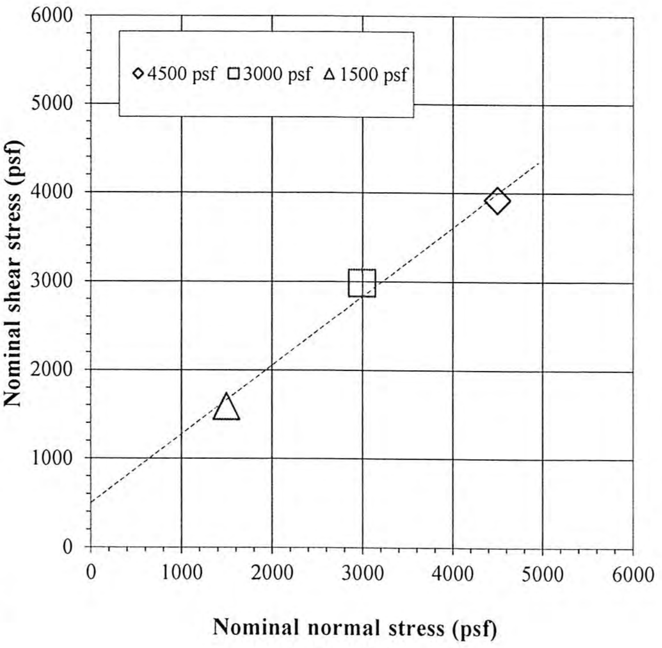
	Sample 1		Sample 2		Sample 3	
Nominal normal stress (psf)	4500		3000		1500	
Peak shear stress (psf)	3919		2980		1589	
Lateral displacement at peak (in)	0.057		0.052		0.042	
Load Duration (min)	79		84		59	
	Initial	Pre-shear	Initial	Pre-shear	Initial	Pre-shear
Sample height (in)	0.995	0.976	1.002	0.982	0.995	0.980
Sample diameter (in)	2.409	2.409	2.414	2.414	2.423	2.423
Wt. rings + wet soil (g)	197.49	207.07	199.36	209.02	195.44	205.97
Wt. rings (g)	44.63	44.63	44.79	44.79	41.95	41.95
Wet soil + tare (g)	313.38		313.38		313.38	
Dry soil + tare (g)	302.13		302.13		302.13	
Tare (g)	122.19		122.19		122.19	
Water content (%)	6.3	12.9	6.3	12.9	6.3	13.5
Dry unit weight (pcf)	120.8	123.2	120.8	123.2	119.9	121.7
Void ratio, e, for assumed Gs	0.37	0.34	0.37	0.34	0.38	0.36
Saturation (%)*	44.9	100.0	44.9	100.0	43.7	100.0

ϕ' (deg)	38
c' (psf)	499

Average of 3 samples	Initial	Pre-shear
Water content (%)	6.3	13.1
Dry unit weight (pcf)	120.5	122.7

*Pre-shear saturation set to 100% for phase calculations





Entered by:
Reviewed:

Direct Shear Test for Soils Under Drained Conditions

IGES

(ASTM D3080)

Project: **Gordon Geotechnical Engineering** **Boring No.:**
No: **M02106-006 (528-002-18)** **Sample:** **Sample C**
Location: View 62 **Depth:**

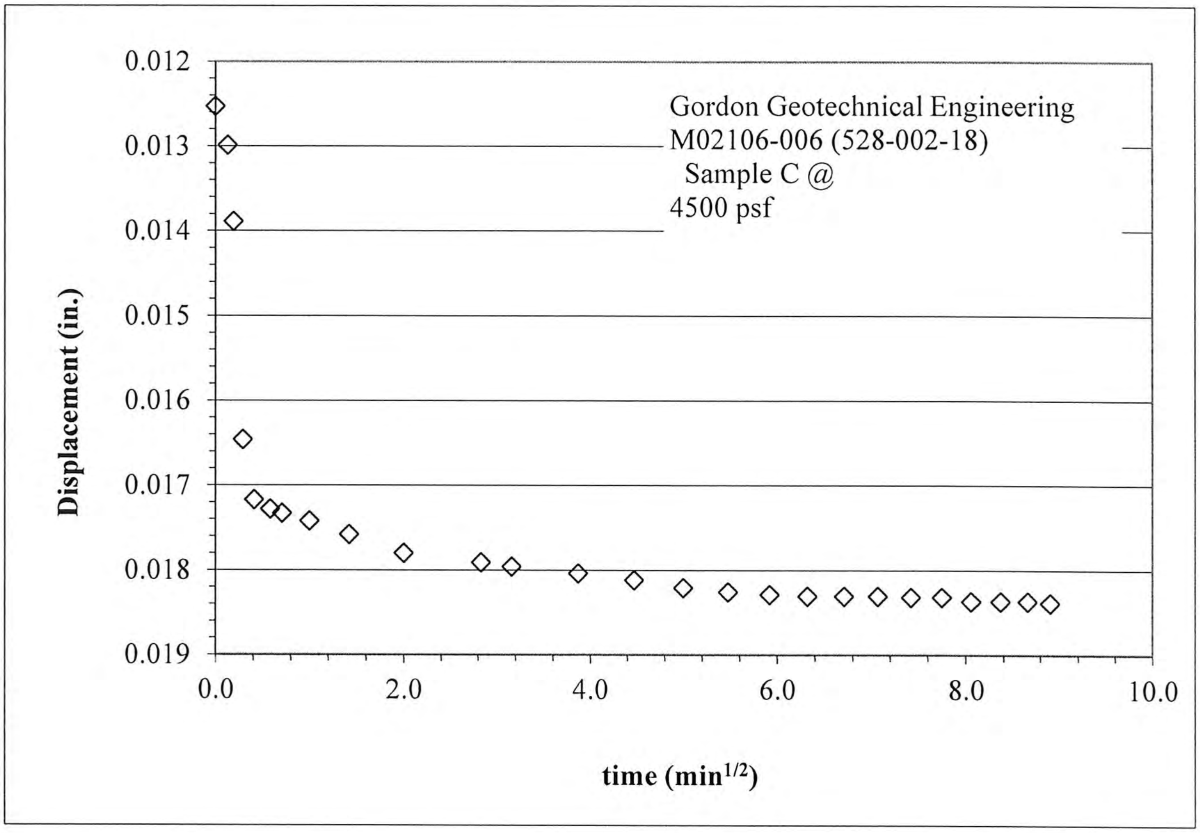
Nominal normal stress = 4500 psf			Nominal normal stress = 3000 psf			Nominal normal stress = 1500 psf		
Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)
0.002	366	0.000	0.002	397	0.000	0.002	183	0.000
0.005	702	0.000	0.005	605	-0.001	0.005	364	0.000
0.007	1029	0.000	0.007	747	-0.001	0.007	538	0.000
0.010	1337	0.000	0.010	1018	-0.001	0.010	669	-0.001
0.012	1663	0.000	0.012	1237	-0.001	0.012	803	-0.001
0.017	2220	-0.001	0.017	1680	-0.001	0.017	1086	-0.001
0.022	2676	-0.002	0.022	2085	0.000	0.022	1306	0.000
0.027	3097	-0.002	0.027	2397	0.001	0.027	1407	0.001
0.032	3400	-0.001	0.032	2636	0.001	0.032	1496	0.002
0.037	3614	0.000	0.037	2803	0.003	0.037	1553	0.003
0.042	3752	0.000	0.042	2924	0.005	0.042	1589	0.005
0.047	3844	0.002	0.047	2973	0.006	0.047	1585	0.006
0.052	3883	0.003	0.052	2980	0.007	0.052	1572	0.007
0.057	3919	0.004	0.057	2964	0.008	0.057	1552	0.009
0.062	3917	0.005	0.062	2937	0.009	0.062	1513	0.010
0.067	3901	0.006	0.067	2897	0.010	0.067	1473	0.011
0.072	3888	0.006	0.072	2819	0.012	0.072	1438	0.011
0.077	3836	0.008	0.077	2752	0.013	0.077	1396	0.012
0.082	3776	0.008	0.082	2681	0.013	0.082	1355	0.013
0.087	3674	0.009	0.087	2617	0.013	0.087	1326	0.013
0.092	3585	0.010	0.092	2569	0.013	0.092	1299	0.013
0.097	3476	0.010	0.097	2546	0.013	0.097	1284	0.013
0.102	3400	0.011	0.102	2526	0.014	0.102	1273	0.013
0.107	3337	0.011	0.107	2490	0.014	0.107	1266	0.013
0.112	3288	0.011	0.112	2479	0.014	0.112	1230	0.014
0.117	3243	0.011	0.117	2480	0.014	0.117	1233	0.014
0.122	3196	0.011	0.122	2478	0.014	0.122	1254	0.014
0.127	3175	0.011	0.127	2472	0.014	0.127	1263	0.014
0.132	3152	0.011	0.132	2493	0.014	0.132	1248	0.014
0.137	3165	0.011	0.137	2496	0.014	0.137	1265	0.014
0.142	3152	0.011	0.142	2497	0.014	0.142	1279	0.014
0.147	3141	0.011	0.147	2508	0.014	0.147	1285	0.014
0.152	3160	0.011	0.152	2513	0.014	0.152	1288	0.014
0.157	3154	0.011	0.157	2517	0.014	0.157	1291	0.014
0.162	3173	0.011	0.162	2537	0.014	0.162	1303	0.014
0.167	3175	0.010	0.167	2543	0.014	0.167	1295	0.014
0.172	3175	0.010	0.172	2548	0.014	0.172	1303	0.013
0.177	3154	0.010	0.177	2550	0.014	0.177	1303	0.013
0.182	3188	0.009	0.182	2573	0.014	0.182	1304	0.013
0.187	3186	0.009	0.187	2569	0.014	0.187	1308	0.013
0.192	3214	0.009	0.192	2570	0.014	0.192	1298	0.013
0.197	3214	0.009	0.197	2589	0.014	0.197	1295	0.013
0.202	3220	0.008	0.202	2579	0.013	0.202	1295	0.013
0.207	3238	0.008	0.207	2583	0.013	0.207	1301	0.013
0.212	3256	0.008	0.212	2572	0.013	0.212	1300	0.013
0.217	3261	0.008	0.217	2578	0.013	0.217	1313	0.012
0.222	3288	0.008	0.222	2574	0.013	0.222	1314	0.012
0.227	3314	0.007	0.227	2583	0.013	0.227	1316	0.012
0.232	3321	0.007	0.232	2579	0.013	0.232	1322	0.012
0.237	3321	0.007	0.237	2591	0.012	0.237	1327	0.012
0.242	3324	0.006	0.242	2589	0.012	0.242	1338	0.012
0.247	3332	0.006	0.247	2592	0.012	0.247	1345	0.012
0.252	3335	0.006	0.252	2602	0.011	0.252	1351	0.011
0.257	3332	0.006	0.257	2596	0.011	0.257	1363	0.011
0.262	3345	0.006	0.262	2609	0.011	0.262	1368	0.011
0.267	3371	0.005	0.267	2608	0.010	0.267	1376	0.011
0.272	3376	0.005	0.272	2619	0.010	0.272	1386	0.011
0.277	3376	0.005	0.277	2631	0.010	0.277	1390	0.011
0.282	3402	0.004	0.282	2639	0.009	0.282	1398	0.010
0.287	3371	0.004	0.287	2659	0.009	0.287	1408	0.010
0.292	3389	0.003	0.292	2674	0.009	0.292	1421	0.010
0.297	3405	0.003	0.297	2680	0.009	0.297	1426	0.010
0.299	3418	0.003	0.300	2692	0.009	0.300	1420	0.010

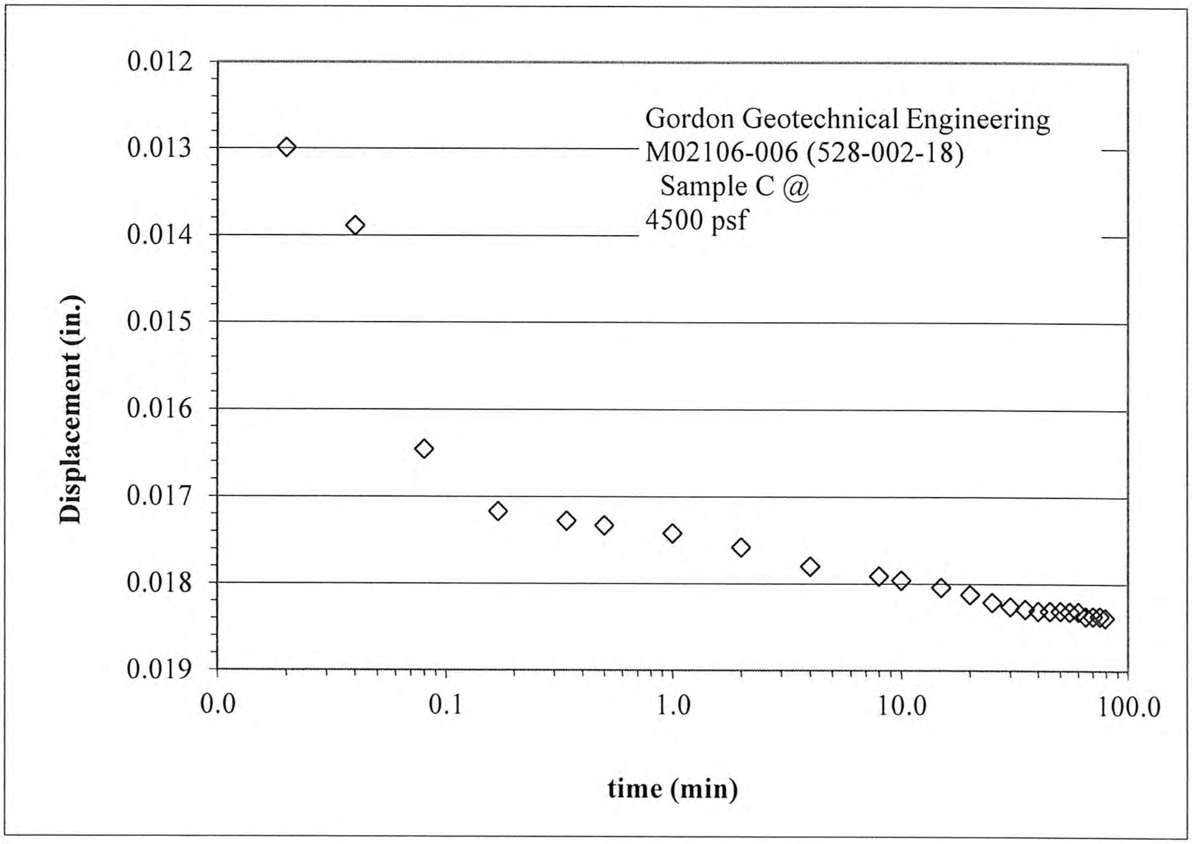
Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: **Gordon Geotechnical Engineering**
No: **M02106-006 (528-002-18)**
Location: View 62

Boring No.:
Sample: **Sample C**
Depth:





Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Gordon Geotechnical Engineering
No: M02106-006 (528-002-18)
Location: View 62
Date: 4/5/2018
By: EH

Boring No.: TP-5
Sample:
Depth:
Sample Description: Brown clay
Sample type: Laboratory compacted
Dry unit weight 118.1 pcf
at 10.8 (%) w
Compaction specifications: 95% of $\gamma_d$max

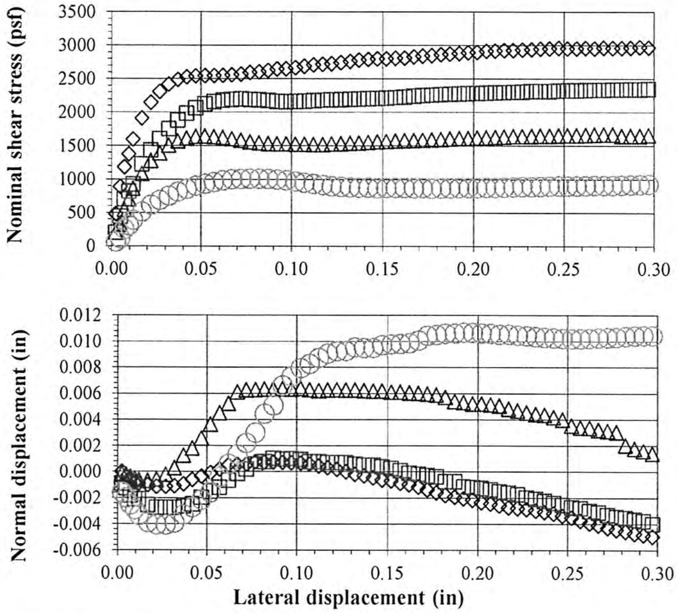
Test type: Inundated
Lateral displacement (in.): 0.3
Shear rate (in./min): 0.0007
Specific gravity, Gs: 2.70 Assumed

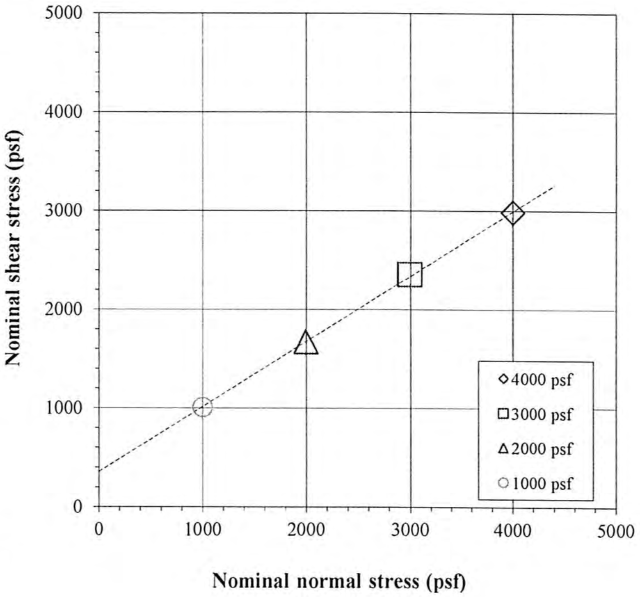
	Sample 1		Sample 2		Sample 3		Sample 4	
Nominal normal stress (psf)	4000		3000		2000		1000	
Peak shear stress (psf)	2984		2355		1672		1012	
Lateral displacement at peak (in)	0.301		0.301		0.277		0.077	
Load Duration (min)	981		931		991		46	
	Initial	Pre-shear	Initial	Pre-shear	Initial	Pre-shear	Initial	Pre-shear
Sample height (in)	1.002	0.984	0.993	0.946	0.998	0.995	1.002	0.992
Sample diameter (in)	2.423	2.423	2.416	2.416	2.422	2.422	2.417	2.417
Wt. rings + wet soil (g)	201.88	207.81	201.98	205.62	200.49	207.51	203.84	210.44
Wt. rings (g)	43.21	43.21	45.58	45.58	42.54	42.54	46.11	46.11
Wet soil + tare (g)	277.93		277.93		277.93		277.93	
Dry soil + tare (g)	263.50		263.50		263.50		263.50	
Tare (g)	128.39		128.39		128.39		128.39	
Water content (%)	10.7	14.8	10.7	13.3	10.7	15.6	10.7	15.3
Dry unit weight (pcf)	118.2	120.3	118.3	124.1	118.2	118.6	118.1	119.2
Void ratio, e, for assumed Gs	0.43	0.40	0.43	0.36	0.43	0.42	0.43	0.41
Saturation (%)*	67.7	100.0	67.8	100.0	67.8	100.0	67.5	100.0

ϕ' (deg)	33
c' (psf)	356

Average of 4 samples	Initial	Pre-shear
Water content (%)	10.7	14.7
Dry unit weight (pcf)	118.2	120.5

*Pre-shear saturation set to 100% for phase calculations





Comments:
Test specimens swelled upon inundation.

Entered by: EH
Reviewed: NB

C:\Users\nickb\Desktop\Lab_Temp\[DS_GC_4-point_v4.xlsm]1

Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

IGES

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Project: **Gordon Geotechnical Engineering**
No: **M02106-006 (528-002-18)**
Location: View 62

Boring No.: **TP-5**
Sample:
Depth:

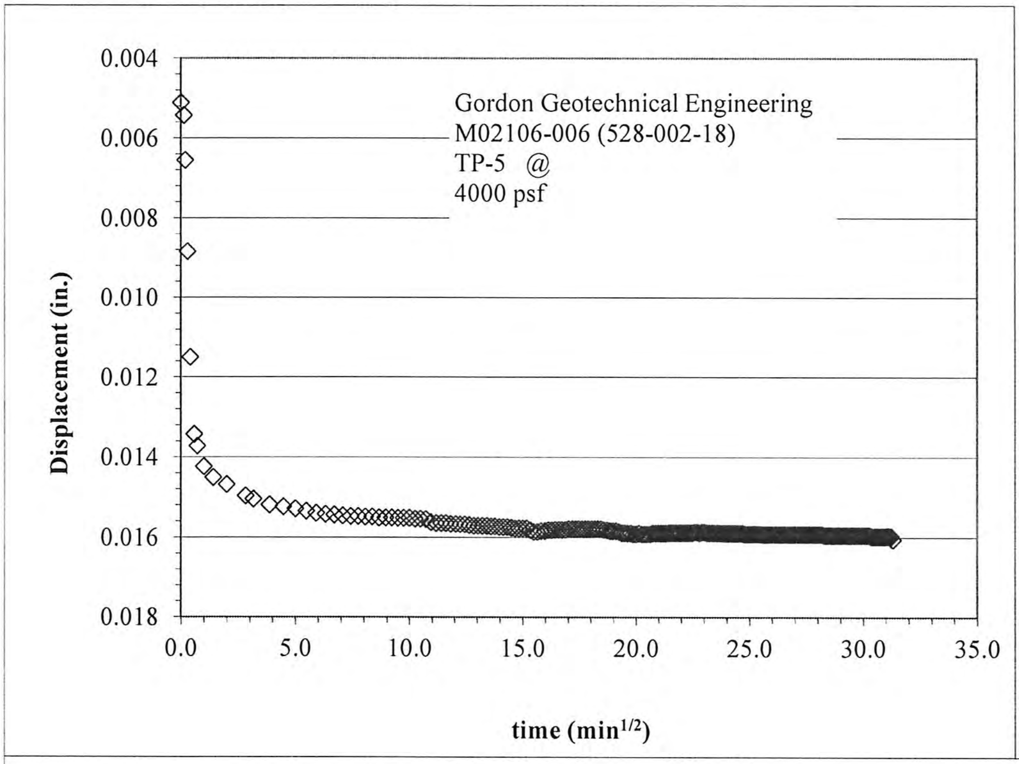
Nominal normal stress = 4000 psf			Nominal normal stress = 3000 psf			Nominal normal stress = 2000 psf			Nominal normal stress = 1000 psf		
Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)
0.002	480	0.000	0.002	205	-0.001	0.002	203	0.000	0.002	67	-0.002
0.005	888	0.000	0.005	442	-0.001	0.005	327	0.000	0.005	120	-0.002
0.007	1182	-0.001	0.007	704	-0.002	0.007	545	0.000	0.007	254	-0.003
0.010	1373	-0.001	0.010	821	-0.002	0.010	704	-0.001	0.010	311	-0.003
0.012	1595	-0.001	0.012	926	-0.002	0.012	860	-0.001	0.012	391	-0.003
0.017	1913	-0.001	0.017	1223	-0.003	0.017	1083	-0.001	0.017	514	-0.004
0.022	2147	-0.001	0.022	1435	-0.003	0.022	1274	-0.001	0.022	617	-0.004
0.027	2310	-0.001	0.027	1606	-0.003	0.027	1392	0.000	0.027	689	-0.004
0.032	2419	-0.001	0.032	1754	-0.003	0.032	1505	0.000	0.032	755	-0.004
0.037	2488	-0.001	0.037	1884	-0.003	0.037	1572	0.001	0.037	812	-0.003
0.042	2527	-0.001	0.042	1984	-0.002	0.042	1614	0.002	0.042	857	-0.003
0.047	2542	0.000	0.047	2067	-0.002	0.047	1638	0.003	0.047	904	-0.002
0.052	2548	0.000	0.052	2124	-0.001	0.052	1643	0.004	0.052	933	-0.002
0.057	2550	0.000	0.057	2160	-0.001	0.057	1636	0.005	0.057	961	0.000
0.062	2553	0.001	0.062	2182	-0.001	0.062	1625	0.005	0.062	984	0.001
0.067	2560	0.001	0.067	2196	0.000	0.067	1604	0.006	0.067	1002	0.001
0.072	2568	0.001	0.072	2201	0.000	0.072	1582	0.006	0.072	1007	0.002
0.077	2594	0.001	0.077	2195	0.001	0.077	1558	0.006	0.077	1012	0.003
0.082	2602	0.001	0.082	2188	0.001	0.082	1543	0.006	0.082	1009	0.004
0.087	2625	0.001	0.087	2175	0.001	0.087	1532	0.006	0.087	1006	0.005
0.092	2646	0.001	0.092	2164	0.001	0.092	1527	0.006	0.092	998	0.007
0.097	2653	0.001	0.097	2160	0.001	0.097	1526	0.006	0.097	985	0.007
0.102	2669	0.001	0.102	2163	0.001	0.102	1525	0.006	0.102	967	0.008
0.107	2690	0.001	0.107	2167	0.001	0.107	1522	0.006	0.107	952	0.008
0.112	2710	0.000	0.112	2177	0.001	0.112	1519	0.006	0.112	934	0.009
0.117	2720	0.000	0.117	2184	0.001	0.117	1521	0.006	0.117	917	0.009
0.122	2728	0.000	0.122	2189	0.001	0.122	1526	0.006	0.122	900	0.009
0.127	2746	0.000	0.127	2195	0.001	0.127	1533	0.006	0.127	892	0.009
0.132	2764	0.000	0.132	2197	0.001	0.132	1540	0.006	0.132	883	0.010
0.137	2785	0.000	0.137	2203	0.001	0.137	1545	0.006	0.137	880	0.010
0.142	2793	0.000	0.142	2205	0.001	0.142	1553	0.006	0.142	878	0.010
0.147	2801	-0.001	0.147	2209	0.000	0.147	1560	0.006	0.147	872	0.010
0.152	2801	-0.001	0.152	2217	0.000	0.152	1566	0.006	0.152	874	0.010
0.157	2808	-0.001	0.157	2224	0.000	0.157	1573	0.006	0.157	874	0.010
0.162	2813	-0.001	0.162	2233	0.000	0.162	1578	0.006	0.162	874	0.010
0.167	2829	-0.001	0.167	2244	0.000	0.167	1585	0.006	0.167	872	0.010
0.172	2844	-0.001	0.172	2254	0.000	0.172	1586	0.006	0.172	874	0.010
0.177	2852	-0.002	0.177	2261	0.000	0.177	1592	0.006	0.177	867	0.011
0.182	2865	-0.002	0.182	2269	-0.001	0.182	1596	0.006	0.182	871	0.011
0.187	2873	-0.002	0.187	2274	-0.001	0.187	1606	0.005	0.187	872	0.011
0.192	2883	-0.002	0.192	2279	-0.001	0.192	1611	0.005	0.192	874	0.011
0.197	2893	-0.002	0.197	2282	-0.001	0.197	1614	0.005	0.197	876	0.011
0.202	2899	-0.002	0.202	2288	-0.001	0.202	1621	0.005	0.202	876	0.011
0.207	2914	-0.002	0.207	2292	-0.001	0.207	1626	0.005	0.207	878	0.011
0.212	2919	-0.003	0.212	2297	-0.002	0.212	1631	0.005	0.212	879	0.011
0.217	2927	-0.003	0.217	2300	-0.002	0.217	1636	0.005	0.217	883	0.011
0.222	2930	-0.003	0.222	2303	-0.002	0.222	1641	0.005	0.222	887	0.010
0.227	2935	-0.003	0.227	2308	-0.002	0.227	1643	0.005	0.227	889	0.010
0.232	2942	-0.003	0.232	2318	-0.002	0.232	1647	0.004	0.232	891	0.010
0.237	2948	-0.003	0.237	2318	-0.002	0.237	1651	0.004	0.237	897	0.010
0.242	2955	-0.003	0.242	2318	-0.002	0.242	1652	0.004	0.242	900	0.010
0.247	2961	-0.003	0.247	2325	-0.003	0.247	1656	0.004	0.247	902	0.010
0.252	2955	-0.004	0.252	2330	-0.003	0.252	1661	0.004	0.252	902	0.010
0.257	2955	-0.004	0.257	2329	-0.003	0.257	1661	0.003	0.257	905	0.010
0.262	2961	-0.004	0.262	2335	-0.003	0.262	1664	0.003	0.262	907	0.010
0.267	2955	-0.004	0.267	2338	-0.003	0.267	1667	0.003	0.267	911	0.010
0.272	2966	-0.004	0.272	2340	-0.003	0.272	1671	0.003	0.272	913	0.010
0.277	2963	-0.004	0.277	2343	-0.003	0.277	1672	0.003	0.277	914	0.010
0.282	2961	-0.005	0.282	2343	-0.004	0.282	1651	0.002	0.282	919	0.010
0.287	2966	-0.005	0.287	2346	-0.004	0.287	1652	0.002	0.287	920	0.010
0.292	2976	-0.005	0.292	2347	-0.004	0.292	1653	0.002	0.292	925	0.010
0.297	2973	-0.005	0.297	2353	-0.004	0.297	1663	0.001	0.297	930	0.010
0.301	2984	-0.005	0.301	2355	-0.004	0.300	1660	0.001	0.300	931	0.010

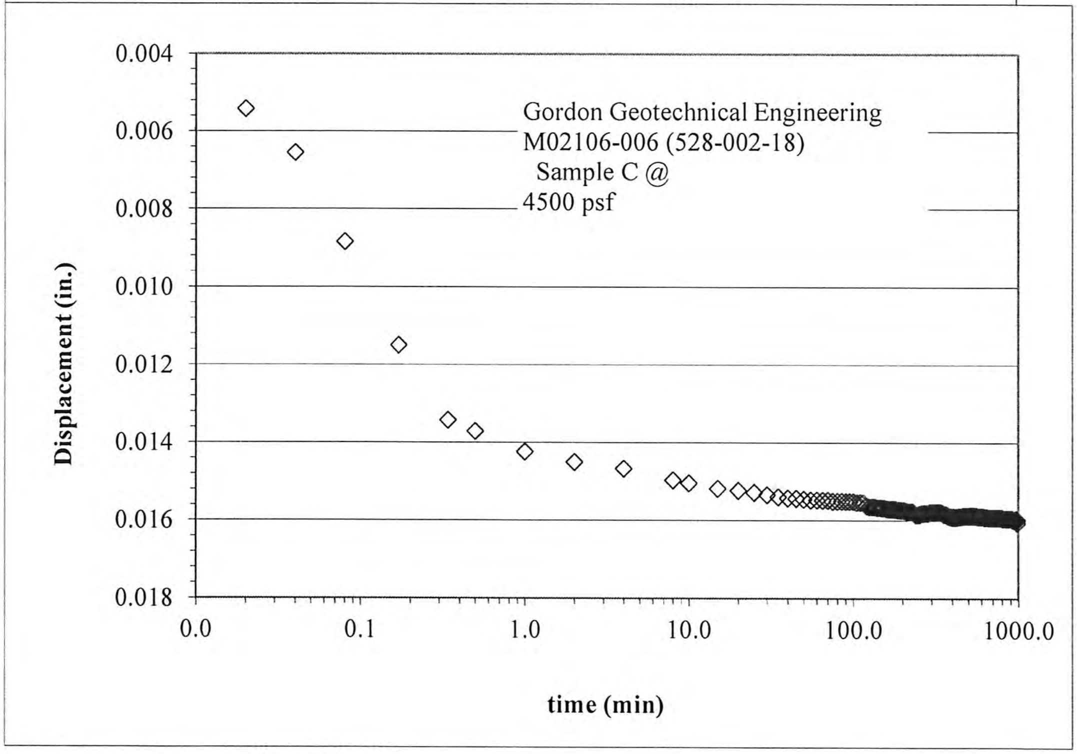
Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Gordon Geotechnical Engineering
No: M02106-006 (528-002-18)
Location: View 62

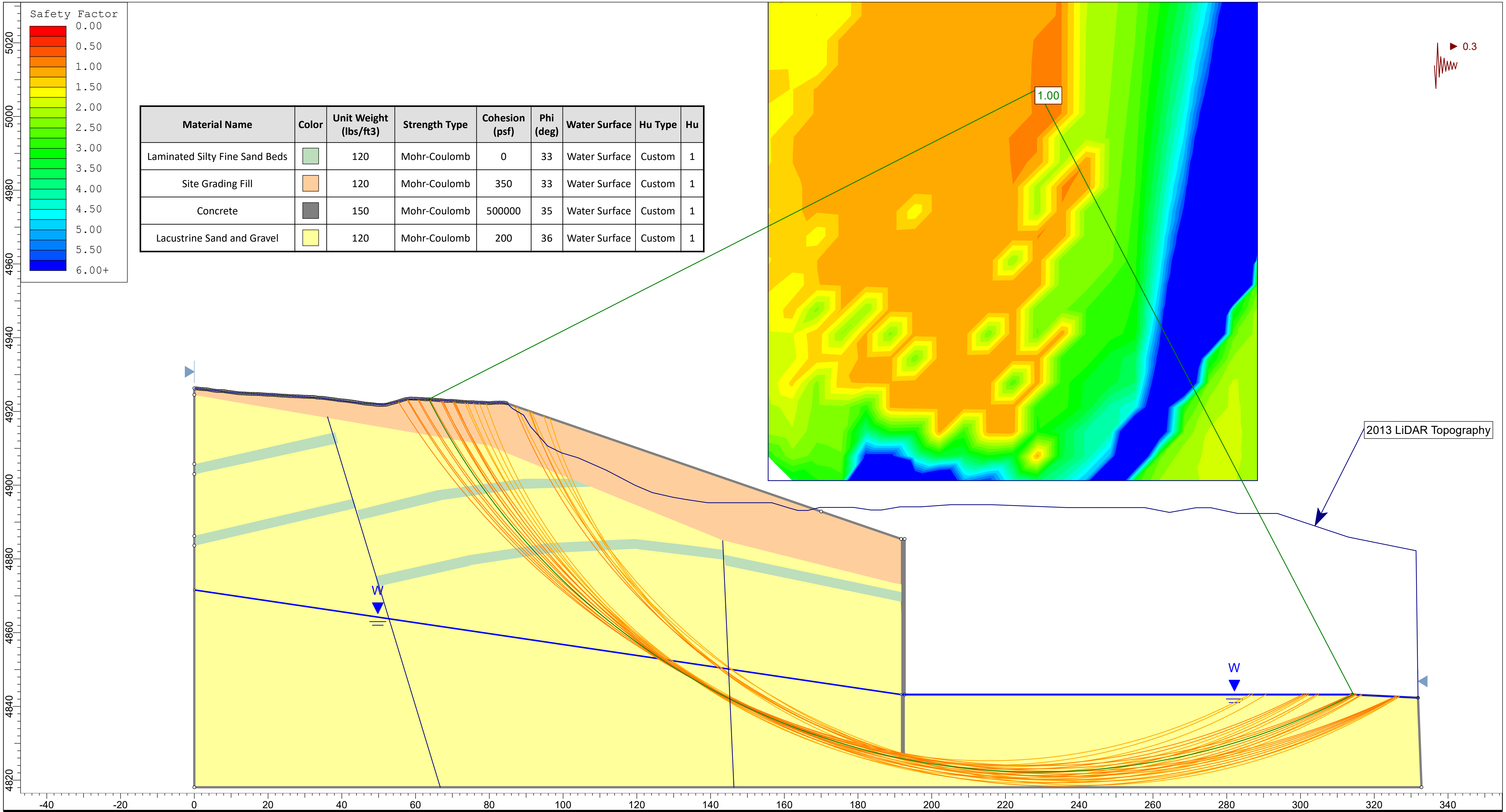
Boring No.: TP-5
Sample:
Depth:





APPENDIX D

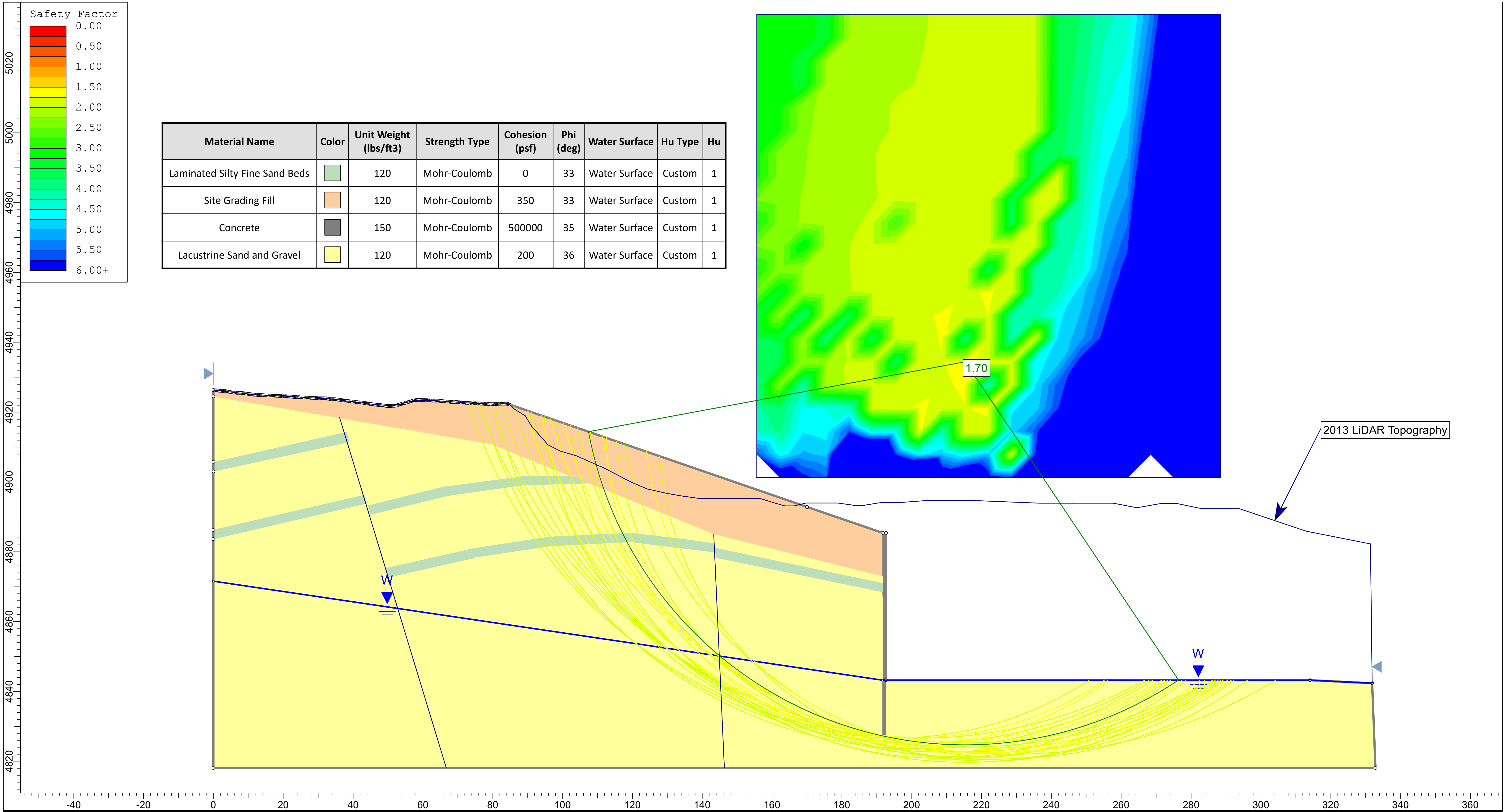
Slope Stability Analysis Results



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Laminated Silty Fine Sand Beds	🟩	120	Mohr-Coulomb	0	33	Water Surface	Custom	1
Site Grading Fill	🟧	120	Mohr-Coulomb	350	33	Water Surface	Custom	1
Concrete	⬛	150	Mohr-Coulomb	500000	35	Water Surface	Custom	1
Lacustrine Sand and Gravel	🟨	120	Mohr-Coulomb	200	36	Water Surface	Custom	1

SLIDEINTERPRET 6.039

Project	Gravel Pit Development		
Analysis Description	A-A'		
Drawn By	JKC	*Scale*	1:296
Company	G2		
Date	5/11/2020, 4:50:55 PM	*File Name*	A-A' Grading.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Laminated Silty Fine Sand Beds	🟩	120	Mohr-Coulomb	0	33	Water Surface	Custom	1
Site Grading Fill	🟧	120	Mohr-Coulomb	350	33	Water Surface	Custom	1
Concrete	⬛	150	Mohr-Coulomb	500000	35	Water Surface	Custom	1
Lacustrine Sand and Gravel	🟨	120	Mohr-Coulomb	200	36	Water Surface	Custom	1

rocscience

SLIDEINTERPRET 6.039

Project Gravel Pit Development

Analysis Description A-A'

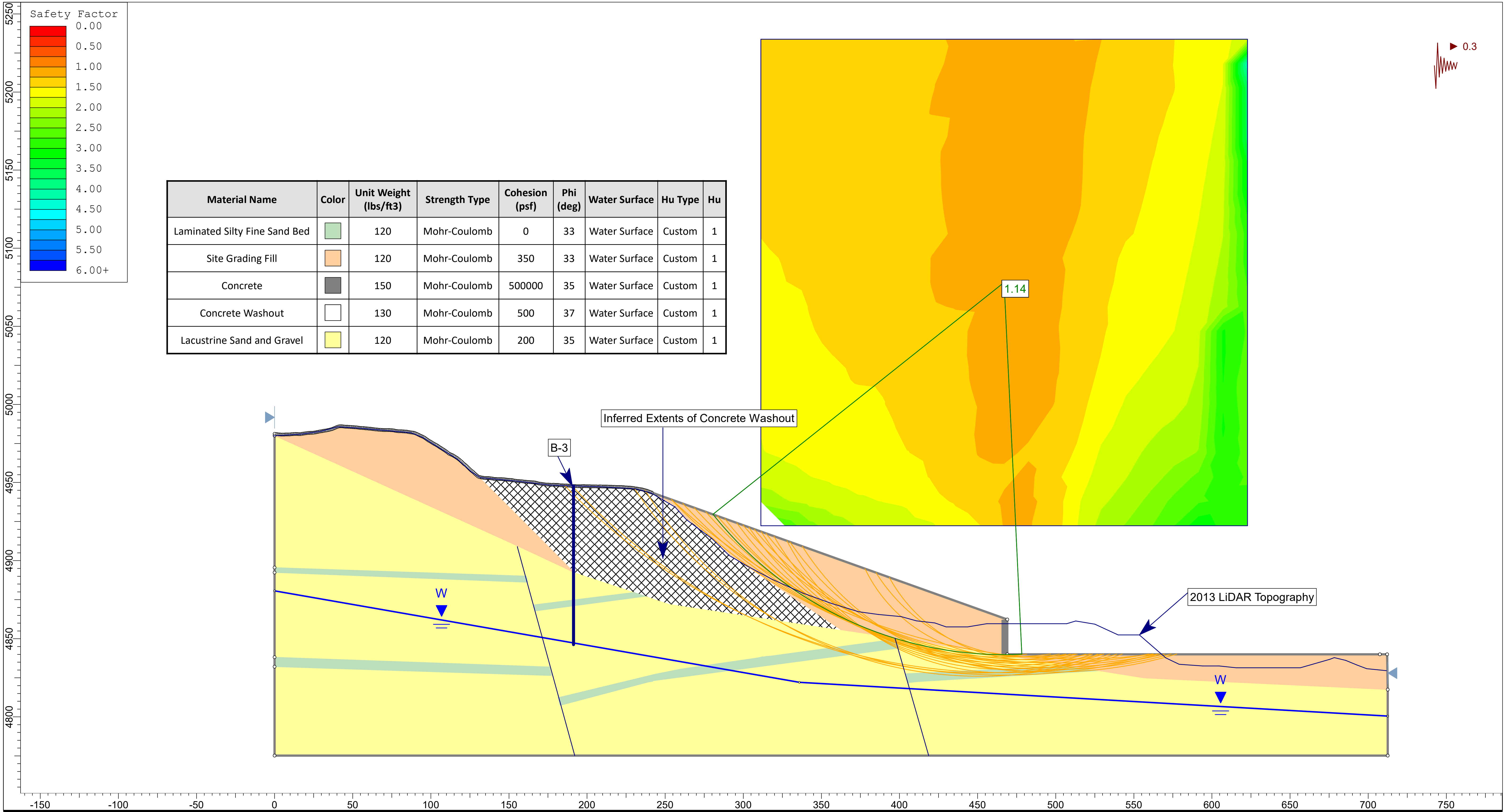
Drawn By JKC

Scale 1:312

Company G2

Date 5/11/2020, 4:50:55 PM

File Name A-A' Grading.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Laminated Silty Fine Sand Bed	🟩	120	Mohr-Coulomb	0	33	Water Surface	Custom	1
Site Grading Fill	🟧	120	Mohr-Coulomb	350	33	Water Surface	Custom	1
Concrete	⬛	150	Mohr-Coulomb	500000	35	Water Surface	Custom	1
Concrete Washout	⬜	130	Mohr-Coulomb	500	37	Water Surface	Custom	1
Lacustrine Sand and Gravel	🟨	120	Mohr-Coulomb	200	35	Water Surface	Custom	1

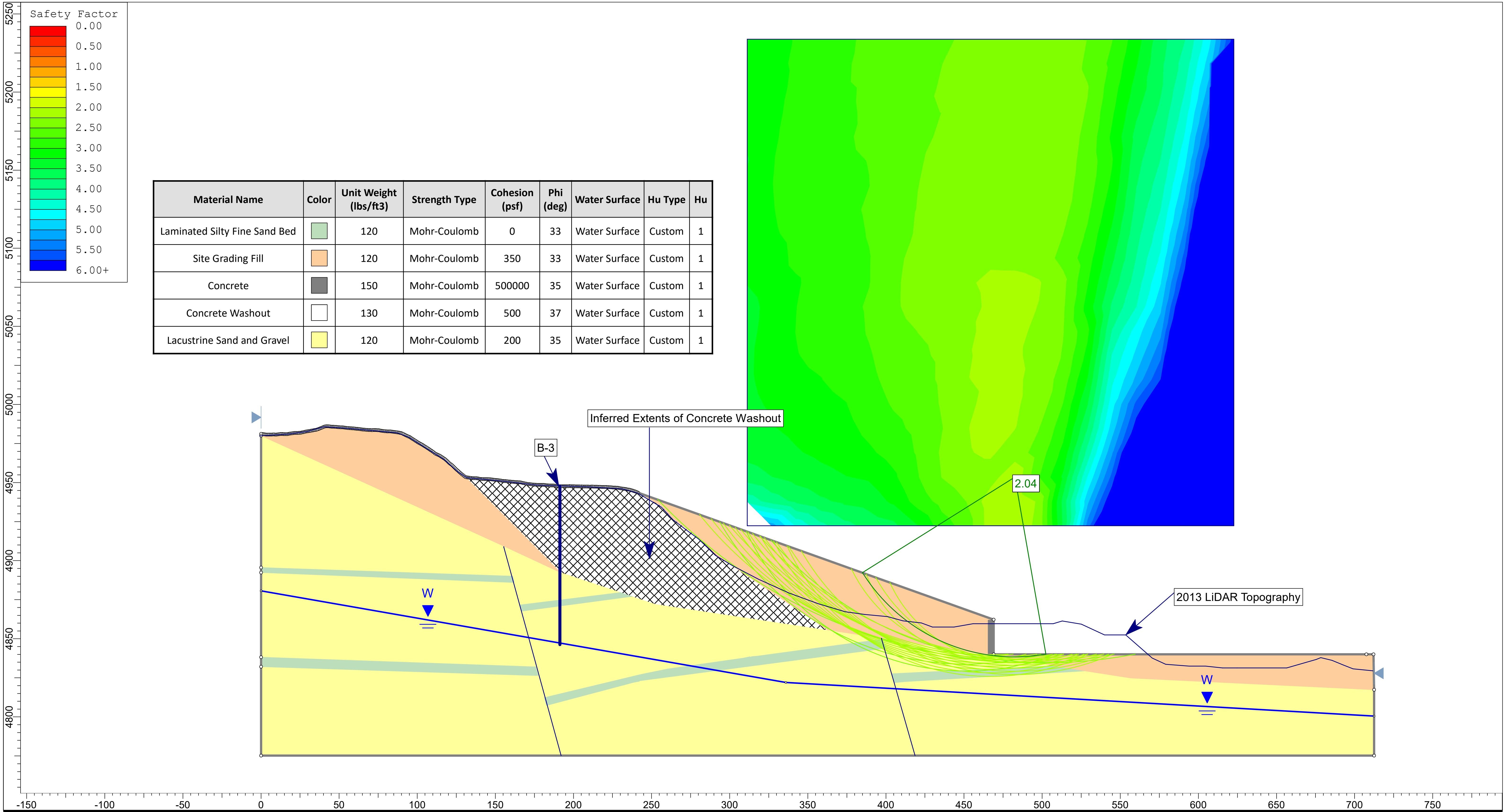
SLIDEINTERPRET 6.039

Project
Gravel Pit Development

Analysis Description
B-B'

Drawn By JKC
Scale 1:698
Company G2

Date 5/11/2020, 5:21:49 PM
File Name B-B' Grading.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Laminated Silty Fine Sand Bed	🟩	120	Mohr-Coulomb	0	33	Water Surface	Custom	1
Site Grading Fill	🟧	120	Mohr-Coulomb	350	33	Water Surface	Custom	1
Concrete	⬛	150	Mohr-Coulomb	500000	35	Water Surface	Custom	1
Concrete Washout	⬜	130	Mohr-Coulomb	500	37	Water Surface	Custom	1
Lacustrine Sand and Gravel	🟨	120	Mohr-Coulomb	200	35	Water Surface	Custom	1

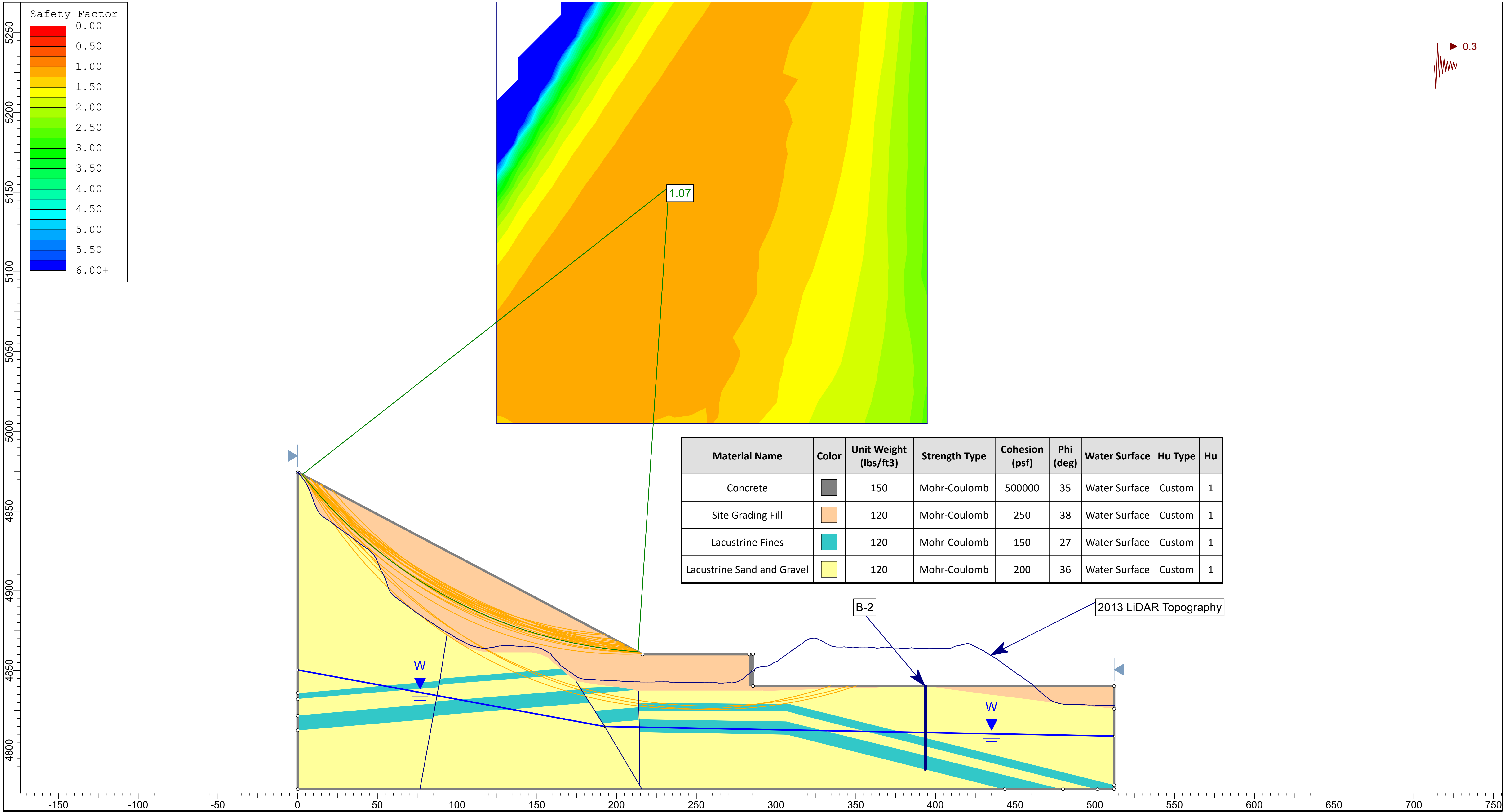
SLIDEINTERPRET 6.039

Project
Gravel Pit Development

Analysis Description
B-B'

Drawn By JKC
Scale 1:698
Company G2

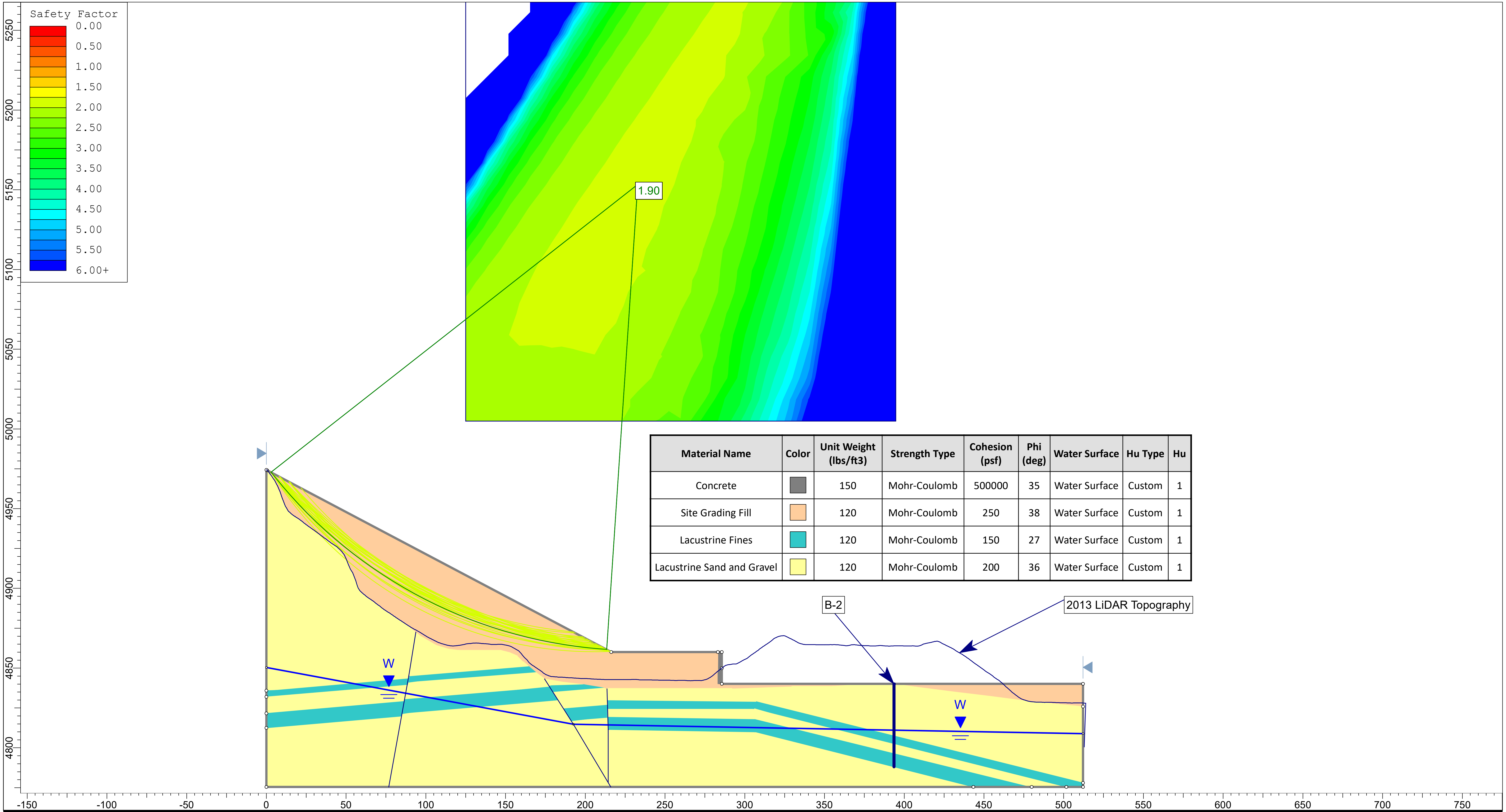
Date 5/11/2020, 5:21:49 PM
File Name B-B' Grading.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Concrete	⬛	150	Mohr-Coulomb	500000	35	Water Surface	Custom	1
Site Grading Fill	🟧	120	Mohr-Coulomb	250	38	Water Surface	Custom	1
Lacustrine Fines	🟦	120	Mohr-Coulomb	150	27	Water Surface	Custom	1
Lacustrine Sand and Gravel	🟨	120	Mohr-Coulomb	200	36	Water Surface	Custom	1

Project	Gravel Pit Development
Analysis Description	C-C'
Drawn By	JKC
Scale	1:684
Company	G2
Date	5/11/2020, 5:40:35 PM
File Name	C-C' Grading.slim

SLIDEINTERPRET 6.039



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Concrete	⬛	150	Mohr-Coulomb	500000	35	Water Surface	Custom	1
Site Grading Fill	🟧	120	Mohr-Coulomb	250	38	Water Surface	Custom	1
Lacustrine Fines	🟦	120	Mohr-Coulomb	150	27	Water Surface	Custom	1
Lacustrine Sand and Gravel	🟨	120	Mohr-Coulomb	200	36	Water Surface	Custom	1

Project
Gravel Pit Development

Analysis Description
C-C'

Drawn By JKC
Scale 1:684
Company G2

Date 5/11/2020, 5:40:35 PM
File Name C-C' Grading.slim

SLIDEINTERPRET 6.039