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COMBINED GEOTECHNICAL ENGINEERING AND GEOLOGIC STUDY

Proposed Rock Loft Ridge Subdivision

About 250 North 1450 East
Fruit Heights, Utah
CMT Project No. 12188

FOR:
Mr. Matt Lowe
Lowe Companies
5028 South Ridge Drive, #203
South Ogden, Utah 84403

November 12, 2020 (Revised February 3, 2021)

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Mr. Matt Lowe
Lowe Companies
5028 South Ridge Drive, #203
Fruit Heights, Utah 84403

Subject: Combined Geotechnical Engineering and Geologic Study
Proposed Rock Loft Ridge Subdivision
About 250 North 1450 East
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CMT Project No. 12188

Mr. Lowe:

Since 2008, various geologic and geotechnical evaluations have been conducted by Great Basin Earth Science (GBES), GSH Geotechnical (GSH), Western Geologic, and CMT Engineering Laboratories (CMT) for the Project. GBES conducted the original investigation for the Project in 2008, which was termed Cherry Ridge at that time. Thereafter, the property was split into two northern and southern portions. The southern portion, which was formerly termed the Raymond Property, was investigated by GSH and CMT, in conjunction with Western Geologic, between 2014 and 2017. The northern portion, which was formerly termed the Proposed Cherry Ridges Subdivision, was investigated by CMT in 2018. Various report reviews have been conducted by GeoStrata at the request of Fruit Heights City for these investigations. GeoStrata also provided a screening-level geologic hazard assessment for the Project area in 2018, as well as a debris flow hazard assessment of the area in March 2020 that evaluated potential impacts from the September 2019 Francis Fire.

The Proposed Rock Loft Ridge Subdivision (Project) currently includes both the Raymond Property and Proposed Cherry Ridges Subdivision. The report submitted herewith presents a geotechnical engineering and geologic hazard evaluation for the subject site that compiles and combines data and findings from the studies conducted by GBES, GSH, Western Geologic, and CMT into one report. Reports prepared by GBES for the inclusive site, and by GSH, CMT and Western Geologic with regard to the former Raymond Property, are provided in Appendix B of this report. Data, interpretations and analyses from these studies are presumed factual and accurate, except where noted, but are the responsibility of the investigators. This report contains the results of our findings and an engineering interpretation of the results with respect to the available Project characteristics based on the above and subsequent data and analyses by CMT, as provided herein. The report contains recommendations to aid in the design and construction of the earth-related phases of this Project.

Conventional spread and/or continuous footings may be utilized to support the proposed structures, provided the recommendations in this report are followed. A detailed discussion of design and construction criteria is presented in this report.

We appreciate the opportunity to work with you at this stage of the Project. CMT offers a full range of Geotechnical Engineering, Geological, Material Testing, Special Inspection services, and Phase I and II Environmental Site Assessments. With 8 offices throughout Utah and Arizona, our staff is capable of efficiently serving your Project needs. If we can be of further assistance or if you have any questions regarding this Project, please do not hesitate to contact us at (801) 870-6730.

Sincerely,
CMT Engineering Laboratories



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TABLE OF CONTENTS

1.0 INTRODUCTION 1
1.1 General 1
1.2 Objectives, Scope and Authorization 3
1.3 Description of Proposed Construction 4
1.4 Executive Summary 5
2.0 FIELD EXPLORATION 5
2.1 General 5
3.0 LABORATORY TESTING 6
3.1 General 6
3.2 One-Dimensional Consolidation Tests 9
3.3 Direct Shear Test 9
4.0 ENGINEERING GEOLOGY 11
4.1 Seismotectonic Setting 11
4.2 Lake Bonneville History 12
4.3 Surficial Geology 13
4.3 Seismic Hazards 15
4.3.1 Strong Ground Motions 15
4.3.2 Site Class 16
4.3.3 Ground Motions 16
4.3.4 Liquefaction 17
4.3.5 Surface Faulting 18
4.4 Landslide and Slump Deposits 21
4.5 Sloping Surfaces 21
4.6 Alluvial-Fan Flooding 22
4.7 Flooding Hazards 24
4.8 Rockfall Hazards 24
5.0 SITE CONDITIONS 25
5.1 Surface Conditions 25
5.2 Subsurface Soils 25
5.3 Groundwater 27
5.4 Site Subsurface Variations 27
6.0 SITE PREPARATION AND GRADING 28
6.1 General 28
6.2 Temporary Excavations 29
6.3 Permanent Cut/Fill Slopes 30
6.4 Detention Basin 30
6.5 Fill Material 30
6.6 Fill Placement and Compaction 31
6.7 Utility Trenches 32
6.8 Stabilization 33
7.0 SLOPE STATILITY 33
7.1 Input Parameters 33
7.2 Associated Testing Representing Input Parameters 34
7.2.1 Lacustrine Fines (Clay/Silt/Sand) 34
7.2.2 Mass Wasting Colluvium 35
7.2.3 Lacustrine Transgressive Sand & Gravel, Regressive Sand & Gravel, and Post-Lake Alluvium 35
7.2.4 Bedrock and Fills 35
7.3 Stability Analyses 36
7.4 Site Irrigation 40
8.0 FOUNDATION RECOMMENDATIONS 40
8.1 Continuous Spread Foundation Recommendations 41
8.1.1 Installation 41
8.1.2 Estimated Settlement 42

8.1.3 Lateral Resistance 42
9.0 LATERAL EARTH PRESSURES 42
10.0 FLOOR SLABS 43
11.0 DRAINAGE RECOMMENDATIONS 43
11.1 Surface Drainage 43
11.2 Foundation Subdrains 44
12.0 PAVEMENTS 45
13.0 QUALITY CONTROL 46
13.1 Field Observations 46
13.2 Fill Compaction 46
13.3 Excavations 46
14.0 LIMITATIONS 46

APPENDIX A
Figure 1: Vicinity Map
Figure 2: Geologic Map
Figure 3: Drainage and Alluvium
Figure 4A-4I: Slope Cross Section *A-A' – Slope Cross Section I-I'*
Figures 5A to 5Q: Geologic Test Pit Logs
Figures 6A to 6E: Trench 1 Log
Figures 7A to 7B: Trench 2 Log
Figures 8A to 8B: Trench 3 Log
Figures 9A to 9B: Trench 4 Log
Figures 10A to 10B: Trench 5 Log
Figures 11A to 11B: Trench 6 Log
Figure 12: Trench 7 Log
Figure 13: Trench 8 Log
Figures 14 to 28: Geotechnical Test Pit Logs
Figures 29 to 33: Bore Hole Logs
Figure 34: Key to Symbols
Figures 35 to 44: Stability Results
Figures 45 to 56: Direct Shear Tests Results
Plate 1: Site Evaluation
Plate 2: Site-Specific Geologic Map
Plate 3: LIDAR Analysis
Plate 4: Surface Fault Rupture Hazard
Plate 5: Debris Flow Hazard
Plate 6: Landslide Hazard
Plate 7A: Hazard Summary
Plate 7B: Hazard Summary Legend

APPENDIX B
GBES Report (Cherry Ridge)
GSH Reports (Raymond Property)
Western Geologic Report (Raymond Property)
GeoStrata Reports

APPENDIX C
Scarp Infinite Slope Analysis Letter

1.0 INTRODUCTION

1.1 General

In response to your request, CMT Engineering Laboratories (CMT) has prepared a Combined Geotechnical Engineering and Geologic Study for the proposed Rock Loft Ridge Subdivision located in Fruit Heights, Utah (the Project). The Project location is shown on **Figure 1, Vicinity Map**. Regional geology of the Project and nearby area is provided on **Figure 2, Geologic Map**. The Project consists of a multi-phase development currently termed the Rock Loft Ridge Subdivision, which includes a total of 67.03 acres proposed for development of 90 single-family residential lots. GBES (2008) previously conducted a geologic hazards evaluation for the inclusive property, and geotechnical and engineering geology studies have also been completed for the south part of the Rock Loft Ridge Subdivision, termed the Raymond Property, by GSH (2014, 2015) and Western Geologic (2017). The Western Geologic (2017) report replaced a 2015 report produced for GSH. Copies of these reports are provided in Appendix B. The current development plan for the Project and the boring, test pit and trench locations conducted for our study are provided on **Plate 1, Site Evaluation**. Site-specific surficial geology is provided on **Plate 2, Site-Specific Geologic Map**. Slope-terrain information is provided on **Plate 3, LIDAR Analysis**.

The proposed development is located on a bench bounded on the east by steep slopes at the western base of the Wasatch Range and on the west by a steep, west-facing escarpment associated with the Wasatch fault zone. Given its location, the Project is within a sensitive lands district defined by Fruit Heights City Municipal Code and governed by Title 10, Chapter 10.10A Sensitive Lands Evaluation & Development Standards (SLEDS). The purpose of the sensitive lands district is *"…to ensure that development is regulated in a manner that will minimize the potential impact from natural and man-made hazards and will reasonably preserve natural scenic beauty and ecological integrity."* Fruit Heights City SLEDS geologic hazard overlays indicate the western side of the Project is in a Surface Fault Rupture Hazard Study Area (Map 1), most of the Project is in a Debris Flow Hazard Zone (Map 4), and the uppermost northeast corner of the Project is in a Rock Fall Hazard Zone. The Project site also has steep slope areas possibly susceptible to landsliding (Map 2), a very low to moderate liquefaction potential (Map 3), and an estimated groundwater depth of about 30 feet or less (Map 11).

A steep, west-facing escarpment bounds the west side of the Project that is associated with the main trace of the active Weber segment of the Wasatch fault zone. This area has slopes that are steeper than 30%, as shown on **Plate 3**. The main fault trace trends northwestward generally following the western site boundary; an antithetic trace also crosses the detached three-lot parcel west of the main portion of the Project, as indicated on **Plate 2**. GBES (2008) excavated 12 trenches, both onsite and offsite to the west, to assess subsurface conditions and the risk of surface faulting to the inclusive property, as shown on **Plate 1**. Western Geologic (2017) also excavated three additional trenches to investigate the risk from surface faulting to the south half of the Project (former Raymond Property), as shown on **Plate 1**. Eight trenches were excavated for our study to assess subsurface conditions and the risk from surface faulting to the north half of the Project, as shown on **Plate 1**. The GBES (2008) study was conducted following guidelines in Christenson and others (2003); the Western Geologic (2017) and CMT evaluations were conducted following these guidelines and also in

accordance with Fruit Heights City Code Chapter 10.10A, Appendix B, Minimum Standards for Surface Fault Rupture Hazard Studies. Western Geologic (2017) encountered evidence for the main fault trace and the antithetic trace in two of their three trenches, but no evidence for active faulting was observed in the trenches conducted by GBES (2008) or for our study. The hazard from surface fault rupture with respect to the proposed development is shown on **Plate 4, Surface Fault Rupture Hazard**, including active fault locations and recommended non-buildable areas.

Although the site is regionally mapped as being underlain by a series of post-Lake Bonneville coalesced alluvial-fan sediments, site-specific subsurface data from our evaluation indicate the bench on which the main parcel of the Project is situated is predominantly underlain by regressive sand and gravel deposits associated with the Provo stage of Lake Bonneville. GBES (2008) similarly believed that the bench was mainly underlain by Provo-stage lacustrine sediments. However, nine alluvial fans enter portions of the site that pose a risk from alluvial-fan flooding to the current Project. Alluvial fans, drainages, and drainage basins impacting the Project are presented on **Figure 3, Drainages and Alluvium**. GBES (2008) and Western GeoLogic (2017) conducted debris-flow hazard evaluations at the inclusive site and for the former Raymond Property (south part of the current Project). GBES (2008) provided estimated debris volumes for three alluvial fans that showed evidence for Holocene activity, and Western GeoLogic (2017) provided estimates for two alluvial fans, one of which was modeled by GBES (2008). The volume estimates were provided either as a basis for debris basin control structures or to discuss the relative hazard to proposed development areas. GeoStrata (2020) conducted a comprehensive debris flow hazard assessment of the drainage basins east of the Project that evaluated potential impacts from the 2019 Francis Fire and provided debris flow volumes calculated using both empirical and post fire debris-flow methods. All of the above volume estimates are provided herein, but are mainly applicable for debris basin design. Discussions between the USFS and City of Fruit Heights suggest that use of debris basins would not likely be feasible. GeoStrata (2020) instead recommended mitigation include debris flow fencing, berms and/or other structures. Our report similarly recommends use of debris fencing, raised building pads, and channels that direct flooding around homes to reduce the risk from alluvial-fan flooding. The hazard from debris flows and alluvial-fan flooding with respect to the proposed development, as well as recommended mitigation to reduce the risk to impacted lots, is shown on **Plate 5, Debris Flow Hazard**.

Steep slopes bound the east and west sides of the Project, but the bench between these slope areas is relatively gentle. GBES (2008) identified three slumps at the site in unconsolidated sediments exposed in the steep, west-facing escarpment of the Wasatch fault along the west boundary of the Project. We identify five slumps in the west-facing escarpment, rather than three. A sixth offsite landslide that may be contemporaneous with late Pleistocene Lake Bonneville is also slightly east of the southeast site boundary and does not extend into the Project. Slope stability analyses were performed on six representative slope cross sections based on site-specific data, although one cross section mainly models offsite slopes to the east where no data could be obtained. The hazard from landsliding with respect to the proposed development is shown on **Plate 6, Landslide Hazard**.

Except for areas disturbed by prior grading and excavation, the Project mainly has a low risk from rock fall hazards. No detailed rock fall analyses were performed. GBES (2008) and Western Geologic (2017) also did not identify any potential rock fall hazard areas or bedrock source areas at the Project associated with native slopes.

1.2 Objectives, Scope and Authorization

The objectives and scope of our study were planned in discussions between Mr. Matt Lowe of Lowe Companies and Mr. Andrew Harris of CMT Engineering Laboratories (CMT), and authorized by returning a signed copy of our Proposal dated November 5, 2018 and executed on November 26, 2018. The scope was discussed and broadened in a meeting on July 22, 2020 at the offices of GeoStrata (Fruit Heights City contract reviewer).

Our objectives were to:

1) Provide the necessary personnel, equipment and materials to conduct a design-level geotechnical investigation and a geologic study for the proposed design and construction; and

2) Evaluate the site with respect to geological and geotechnical conditions, as well as provide general geotechnical analysis for site development and earthwork procedures, in accordance with the Fruit Heights City Ordinance Title 10, Chapter 10.10A Sensitive Lands Evaluation & Development Standards.

Our scope included:

1) Preparation of a Work Plan, including scope of work development, discussions, plan implementation;

2) Compilation and review of readily-available geologic maps, reports, and aerial photos, including prior geologic and/or geotechnical reports prepared by GBES, Western Geologic, GSH, and GeoStrata either for the inclusive site, former Raymond Property, or surrounding area;

3) A field reconnaissance conducted by an experienced certified engineering geologist to assess the site setting and look for adverse geologic conditions, in addition to prior field reconnaissance(s) conducted by GBES and Western Geologic either for the inclusive site or former Raymond Property;

4) A field program including advancement of five borings, excavation and logging of eight trenches, and excavation and logging of 17 test pits at the site, in addition to prior subsurface investigation conducted by GBES, Western Geologic, GSH and CMT either for the inclusive site or former Raymond Property;

5) Development of nine geologic cross sections for use in geotechnical engineering slope stability analyses, in addition to the geologic cross sections previously prepared by Western Geologic for the former Raymond Property;

6) Site-specific geologic mapping and classification to identify surficial geological conditions and the relation to proposed site improvements;

7) Slope analysis from geoprocessed LIDAR data to identify critical slope areas steeper than 25% across the site, and/or surficial features potentially affecting the proposed site improvements as well as proposed grading provided by the project civil engineer;

8) A laboratory geotechnical soils testing program of samples recovered from the test pits, trenches and borings from typical and critical geological units explored and identified in our subsurface evaluation. The laboratory testing program included (but was not limited to) moisture, density, gradation, Atterberg limits, consolidation, and direct shear tests of representative soil samples; and

9) Preparation of this geological/geotechnical engineering summary report integrating our geological hazards analyses and slope stability parametrics, combined with prior assessments conducted by GBES, Western Geologic, GSH and CMT either for the inclusive site or the former Raymond Property. Our analyses, results and findings presented herein include:

   a) A vicinity map showing the Project location relative to the regional area and topographic features;
   b) Aerial photography showing the site and all subsurface exploration locations;
   c) A surficial geologic map showing geologic conditions with respect to the development plan;
   d) Geologic and geotechnical logs of the trenches, test pits and boreholes conducted for our investigation, as located on **Plate 1**; previous geologic and geotechnical logs prepared by GBES, Western Geologic, GSH and CMT either for the inclusive site or former Raymond Property are provided in Appendix B;
   e) An assessment of potential geologic hazards at the site and their risk to proposed site improvements, including but not limited to: landsliding and site-specific slope stability; surface fault rupture; strong earthquake ground motion; liquefaction hazards; alluvial-fan flooding and debris flows; rockfall hazards; and flood hazards;
   f) Slope cross sections of slopes depicting Projected geological conditions based on available data;
   g) Analyses of site-specific slope stability for both expected static and seismic conditions; and
   h) Site development recommendations based upon our findings and professional experience.

1.3 Description of Proposed Construction

The proposed development includes a total of 67.03 acres that is mainly vacant and partly used for fruit orchards. The property is proposed for development of 90 single-family residential lots, including a main area with 87 lots and a detached parcel with three lots to the west. The development will be for single-family residences. Structures are anticipated to be 1 to 3 levels in height and anticipated to be constructed either slab on grade or with full- or partial-depth basements. Below grade, the structures will be of reinforced concrete construction. Above grade, structures will likely consist of wood-frame construction with stone, brick, stucco, or similar type veneer. Structural loads will be transmitted through bearing walls and isolated columns. We Project that the maximum wall and column loads will be on the order of 2 to 5 kips per lineal foot and 10 to 50 kips, respectively. Floor slab loads are anticipated to be light, i.e. less than 150 pounds per square foot.

Projected traffic over the primary roadways will consist of a moderate volume of automobiles and light trucks and occasional medium- and heavyweight trucks. However, during build-out, the traffic will consist of a much higher percentage and higher volume of medium- and heavyweight construction trucks.

Site development will require an extensive amount of earthwork in the form of cutting and filling for the roadways and building pads. At this time, we Project that maximum cuts and fills will be generally in the range

of 5 to 10 feet; however, in certain specific areas, cuts and fills could approach 15 to 20 feet. Where deeper cuts/fills are planned, CMT should be notified to provide additional recommendations.

1.4 Executive Summary

The proposed structures can be supported upon conventional spread and continuous wall foundations. The most significant geotechnical aspects regarding site development include the following:

1. Presently, marginally stable slopes are present along the west edge of the property and upslope to the east of the property where slopes are steeper than about 2 Horizontal to 1 Vertical (2H:1V), excluding bedrock outcrops, which could fail under seismic conditions.
2. Landslide deposits are mapped within the buildable portions of Lots 27, 28, 41, and 42. Based on our field investigation, the landslide deposits are relatively shallow in depth. Remediation of the landslide deposits within and immediately adjacent to the building footprint will be required.
3. The majority of Phase 5 is located across what appears to have been an open aggregate quarry with relatively steep slopes and significantly lower grades than the adjacent planned development. This area will need to be regraded to suitable, stable conditions prior to major construction activities.
4. Potentially large site grading fill sequences may be incorporated. Where these fills will be greater than about 10 feet, settlement monitoring methods shall be put in place prior to placement of utilities or structures.
5. Vegetation (including trees) to be removed. Root bulbs from trees will require deeper removal depths. The upper 8 to 18 inches may be loose/weathered/disturbed and thereby require removal and/or re-compaction below structural elements. Footings shall not be placed over these soils.
6. Maintaining slope stability is critical to the development of the property. Specific analyses and recommendations related to slope stability are subsequently presented within this report.

CMT must assess that topsoil, undocumented fills, debris, disturbed or unsuitable soils have been removed and that suitable soils have been encountered prior to placing site grading fills, footings, slabs, and pavements. Further, CMT must review final grading to assess if it meets the requirements as stated in this report.

In the following sections, detailed discussions pertaining to the site and subsurface descriptions, geologic/seismic setting, earthwork, slope stability, foundations, lateral resistance, lateral pressure, floor slabs, and pavements are provided.

2.0 FIELD EXPLORATION

2.1 General

Subsurface soil conditions in the northern half of the Project were explored by drilling five borings, and excavating 17 test pits and eight trenches. These borings, test pits and trenches are in addition to the previous borings, test pits and trenches conducted by GBES, Western Geologic, GSH and CMT either for the inclusive site or former Raymond Property (south half of the Project). All the subsurface exploration locations conducted at

the site and nearby are shown on **Plate 1**. The test pits and trenches conducted for the current study were excavated using a track-mounted excavator and extended to depths ranging from 4.0 to 10.7 feet below the existing ground surface, at which point excavation was either stopped or refusal was encountered. During the course of the excavating operations, a continuous log of the subsurface conditions encountered was maintained. Within the test pits and trenches, bulk samples of the typical soils encountered were obtained for subsequent laboratory testing and examination. GBES (2008), GSH (2014) and Western Geologic (2017) discuss methodology for their respective investigations.

The borings conducted for the current investigation were advanced to depths of 30.5 to 101.5 feet with hollow stem augur drilling and interval sampling using a 3.25-inch outside diameter, 2.42-inch inside diameter drive sampler (Dames & Moore) and a 2.0-inch outside diameter, 1.38-inch inside diameter drive sampler (SPT) at select depths within the borings. These samplers were typically driven 18 inches into the soils below the drill augers using a 140-pound hammer free-falling a distance of 30 inches. The number of hammer blows needed for each 6-inch interval was recorded. The sum of the hammer blows for the final 12 inches of penetration is known as a standard penetration test and this 'blow count' was recorded on the bore hole logs. Where more than 50 blows occurred before the 6-inch interval was achieved, the sampling was terminated and the number of blows and inches penetrated by the sampler were recorded. The blow count provides a reasonable approximation of the relative density of granular soils, but only a limited indication of the relative consistency of fine-grained soils because the consistency of these soils is significantly influenced by the moisture content.

The collected samples were logged and described in general accordance with ASTM standard 2488, packaged, and transported to our laboratory. 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. The subsurface conditions encountered in the field exploration are discussed below in Section 5.2. Subsurface Soil Conditions, and are graphically illustrated on geologic test pit and trench logs on **Figures 10 through 18**, the geotechnical test pit logs on **Figures 19 through 33**, and the boring logs on **Figures 34 through 38**. Sampling information and other pertinent data and observations are also included on the logs. In addition, a Key to Symbols defining the terms and symbols used on these logs is provided as **Figure 39** in this report.

The test pits and trenches were backfilled following completion of excavating and logging and a field review by the Fruit Heights City contract geologic reviewer. The backfill was not placed in uniform lifts and compacted to a specific density and therefore must be considered as non-engineered/undocumented backfill. Settlement of the backfill with time is likely to occur.

3.0 LABORATORY TESTING

3.1 General

Selected samples of the subsurface soils were subjected to various laboratory tests to assess pertinent engineering properties, as follows:

1. Moisture Content, ASTM D-2216, Percent moisture representative of field conditions
2. Dry Density, ASTM D-2937, Dry unit weight representing field conditions
3. Atterberg Limits, ASTM D-4318, Plasticity and workability
4. Gradation Analysis, ASTM D-1140/C-117, Grain Size Analysis
5. Friable Particles, AASHTO T112
6. Direct Shear Test, ASTM D-3080, Shear strength parameters
7. One-Dimensional Consolidation, ASTM D-2435

Laboratory test results are presented on the geotechnical logs (**Figures 19 through 38**) and in the following tables. A summary of laboratory testing prepared for the referenced study by GSH is also provided below:

LAB SUMMARY TABLE - Raymond Property (GSH 2014-2015 Study)

Test Pit/ Borehole	Depth (feet)	Soil Class	Sample Type	Moisture Content (%)	Dry Density (pcf)	Gradation			Atterberg Limits		
						Grav	Sand	Fines	LL	PL	PI
TP-1*	12.0	GP	Bag					4.0			
TP-2*	4.0	SP	Bag	6				6.0			
TP-3*	2.0	SP-SM	Bag	10				10.0			
TP-3*	10.5	SM	Bag	13				13.0			
TP-4*	8.0	SM	Bag	13				13.0			
TP-5*	7.5	ML	Bag	55				55.0			
TP-5*	15.0	SM	Bag	33				33.0			
TP-6*	9.0	SM	Bag	13				13.0			
TP-6*	14.0	SP-SM	Bag	7				7.0			
TP-7*	9.0	SM	Bag	16				16.0			
TP-8*	4.0	SM	Bag	15				15.0			
TP-8*	8.0	SM	Bag	31				31.0			
TP-9*	14.0	ML	Bag	69				69.0			
TP-10*	10.5	SP-SM	Bag	7				7.0			
TP-11*	2.0	SM	Bag	14				14.0			
TP-11*	12.0	SM	Bag	11				11.0			
TP-12*	3.0	SP-SM	Bag	9				9.0			
TP-12*	15.5	SP	Bag	6				6.0			
TP-13*	3.0	GM	Bag	27				27.0			
TP-14*	8.0	SP	Bag	6				6.0			
TP-15*	10.0	SM	Bag	31				31.0			
B-1*	20.0	SM	D&M	17				17.0			
B-1*	30.0	GP/Bedrock	SPT	12				12.0			
B-3*	15.0	SM/ML	D&M	69				69.0			

LAB SUMMARY TABLE - CMT Study

Test Pit/ Borehole	Depth (feet)	Soil Class	Sample Type	Moisture Content (%)	Dry Density (pcf)	Gradation			Atterberg Limits		
						Grav	Sand	Fines	LL	PL	PI
TP-1	6.5	GM	Bag	1.9		38	44	18.0			
TP-2	5.0	GP-GM	Bag	2.6		52	36	12.0			
TP-4	6.0	SM	Bag	2.3		21	61	18.0			
TP-5	10.0	GP-GM	Bag	1.5		54	39	7.4			
TP-8	6.0	GM	Bag	3.3		36	51	13.0			
TP-10	10.0	SP-SM	Bag	2.6				8.1			
TP-11	9.5	SM-ML	Bag	4.8				49.0			
TP-13	8.0	ML-SM	Bag	3.6		1	44	55.0			
TP-15	7.0	CL	Bag	21.0	86				30	21	9
B-1	5.0	SP-SM	D&M	4.4	127	31	58	11.0			
	25.0	SP-SM	D&M	4.0	120			11.9			
	30.0	SP-SM	D&M	3.9	108			10.0			
B-2	10.0	SM	D&M	5.0	118			12.8			
B-3	5.0	ML	D&M	6.5	96			82.0	26	23	3
	15.0	ML	D&M	7.5				79.1			
B-4	2.5	SM	D&M	4.1		23	48	29.0			
	7.5	ML	D&M	11.0	106			80.0	25	22	3
	10.0	ML	D&M	16.6				76.4			
	20.0	SM	D&M	13.1	104			26.0			
	35.0	ML	D&M	20.0				86.0	25	22	3
B-5	2.5	Fill-ML	D&M	4.3					21	20	1
	7.5	SM	D&M	25.0	82			27.0			
	20.0	CL-ML	D&M	7.9	107			85.0	27	20	7
	25.0	ML	D&M	8.7	105			87.0	25	22	3
TR-1 Mid	5.0	ML	Block	6.0	100	0	8	92.0			
TR-1 50	4.0	SM-ML	Block	3.1				66.1			
TR-1 190	5.0	ML	Block	8.1	99						
TR-3 30	6.0	GP	Bag	0.6				1.0			
TR-4 67	5.0	GM	Bag	2.5		18	53	29.0			
TR-5 15	7.0	SM-ML	Block	2.3				48.6	23	19	4
TR-5 100	6.0	SM-ML	Block						20	18	2
TR-7	4.0	CL	Bag					88.0	27	16	11
TP-16	7.0	GM-SM	Bag	2.2		42	41	17.0			
TR-8	6.5	CL	Bag					72.9			
	8.0	SC-CL	Bag					59.3			NP
TP-17	4.5	GP-GM	Bag	1.4		50	40	10.0			

3.2 One-Dimensional Consolidation Tests

Consolidation tests were performed on two representative samples of the fine-grained soils (silt/clay) encountered within the upper 8 feet. Based upon data obtained from the consolidation tests, the soils tested are moderately over-consolidated and will exhibit moderate strength and moderate to moderately high compressibility characteristics under the anticipated loadings when wetted. Detailed results of the tests are maintained within our files and can be transmitted to you, upon your request.

3.3 Direct Shear Test

To determine the shear strength of the soils encountered at the site, laboratory direct shear tests were performed on representative samples recovered. In order to account for the site being mapped in a Debris Flow Hazard Zone (Plate 5), and with steep slope areas possibly susceptible to landsliding (Plate 6), residual shear tests were performed. All coarse-grained soils tests were screened over a No. 4 sieve and remolded.

During the direct shear test, the samples were evenly consolidated within the test ring, loaded, and saturated immediately after the load was applied. Loading was conducted at a slower rate to simulate saturated-drained condition. The results of the direct shear tests are presented in the following table and within the appendix of this report:

DIRECT SHEAR RESULTS

Raymond Property (GSH 2014-2015 Study)

Sample Location	Sample Depth (feet)	Unified Soils Classification	Results Method	Apparent Cohesion (psf)	Measured Internal Friction Angle (degrees)
TP-4*	3.5	SM	Peak	325	32
TP-8*	5.0	SM	Peak	20	36
B-2*	10.5	SM	Peak	20	36
B-3*	35.0	CL*	Peak	0	38

*Interbedded sand seams within sample

CMT Study

Sample Location	Sample Depth (feet)	Unified Soils Classification	Results Method	Apparent Cohesion (psf)	Measured Internal Friction Angle (degrees)
B-1	10.0	SP-SM	Peak	144	42
B-2	20	SP	Peak	86	39
B-3	25	ML	Residual	72	26.7
B-4	10	ML	Residual	245	28.1
B-4	30	CL	Residual	115	30.5
B-4	40	CL	Residual	99	25.7
TP-15	7	CL	Peak	204	28.7

CMT Additional Study

Sample Location	Sample Depth (feet)	Unified Soils Classification	Results Method	Apparent Cohesion (psf)	Measured Internal Friction Angle (degrees)
B-4A	80	ML-CL	Peak	340	28.8
Trench 7	4	ML-SM	Residual	120.5	33.1
TP-16	7	GP-SP	Peak	50	38
Trench 8	6.5	GC-GM	Residual	50	33
Trench 8	8	SC	Residual	541.6	32.2

Vane Shears - GSH

Sample Location	Sample Depth (feet)	Unified Soils Classification	Apparent Cohesion (psf)
B-3*	20.0	CL	4180
B-3*	30.0	CL	1300
B-3*	40.0	CL	1800

Chemical Tests - GSH

Sample Location	Sample Depth (feet)	Unified Soils Classification	pH	Total Water-Soluble Sulfate SO_4 (mg/kg-dry)
TP-3*	5.0	SP-SM	6.9	None Detected
TP-12*	3.0	SP-SM	7.0	None Detected

4.0 ENGINEERING GEOLOGY

4.1 Seismotectonic Setting

The property is located in foothills adjacent to the western base of the Wasatch Range, a major north-south trending mountain range marking the boundary between the Basin and Range physiographic province on the west and the Middle Rock Mountains province on the east (Stokes; 1977, 1986). The Basin and Range is characterized by a series of generally north-trending 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 Wasatch fault zone (WFZ) at the base of the Wasatch Range marks the eastern boundary of the Basin and Range. Late Cenozoic normal faulting, a characteristic of the Basin and Range, began between about 17 and 10 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 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 segments, each of which rupture independently and are capable of generating large-magnitude surface-faulting earthquakes (Machette and others, 1992). The central five segments 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 Project is situated along the Weber segment. Solomon (2007; Figure 2) maps a west-dipping main trace of the Weber segment generally following the western boundary of the main parcel that splits on the north into two fault traces. On the south, the main trace is mapped bending slightly westward and ending on the margin of a sequence of alluvial deposits from Bair Canyon. South of the alluvial deposits from Bair Canyon, the fault is mapped as two west-dipping traces (**Figure 2**). Solomon (2007) also maps an-dipping antithetic fault trace crossing the detached three-lot parcel west of the main parcel (**Figure 2**).

The Weber segment of the WFZ extends for about 35 miles from the southern edge of the Plain View salient near North Ogden to the northern edge of the Salt Lake salient near North Salt Lake (Machette and others,

1992). Previous paleoseismic studies indicate four large-magnitude surface-faulting earthquakes have occurred on the Weber segment since mid-Holocene time. Nelson and others (2006) report finding evidence for four events at the Garner Canyon and East Ogden sites, including what they infer was a partial segment rupture (with 1.6 feet of displacement) around 500 years ago. This partial segment rupture was not evident at the Kaysville site of McCalpin and others (1994), although chronologic intervals for the remaining three earthquakes were similar. DuRoss and others (2009) report paleoseismic data from the 2007 Rice Creek site support a preferred scenario of six surface-faulting earthquakes in Holocene time, with four events since about 5,400 years ago, and confirm Nelson and others' (2006) partial segment rupture timing. Lund (2005) indicates preferred earthquake timing for the last four surface-faulting earthquakes on the Weber segment is: (1) an event Z between 200 to 800 years ago (partial segment rupture) and/or between 500 and 1,400 years ago (complete segment rupture), (2) an event Y between 2,300 and 3,700 years ago, (3) an event X between 3,800 and 5,200 years ago, and (4) an event W between 5,400 and 6,800 years ago. The Working Group on Utah Earthquake Probabilities (2016; Table 4.1-2) indicates closed mean recurrence interval for the last five surface-faulting earthquakes on the Weber segment is 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 MS 7.5 event in 1959 near Hebgen Lake, Montana. However, 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 of these events was the 1934 Hansel Valley (MS 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 (https://earthquake.usgs.gov/earthquakes/eventpage/uu60363602/executive) consistent with an earthquake on the Wasatch fault system. Despite being less than magnitude 6.0, this earthquake was felt (https://www.ksl.com/article/46731630/) from southern Idaho to south-central Utah and damaged multiple building. No impacts were observed, noted or reported at the site from the Magna earthquake.

4.2 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).

Timing of events related to the transgression and regression of Lake Bonneville is indicated by calendar age estimates of significant radiocarbon dates in the Bonneville Basin (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. During the transgression and high stand, major drainages that emanate from within the Wasatch Range (such as the Weber River) formed large

deltaic complexes in the lake at their canyon mouths. Headward erosion of the Snake River-Bonneville basin drainage divide then caused a catastrophic incision of the threshold and the lake level lowered by roughly 360 feet in less than two months (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 between about 16,500 and 15,000 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. Drainages that fed Lake Bonneville began downcutting through stranded deltaic complexes and near-shore deposits as the lake receded from the Provo shoreline. Great Salt Lake then experienced a brief transgression around 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). The Project is located below both the Bonneville and Provo shorelines of Lake Bonneville. The Provo shoreline is indicated on **Figure 2** near the eastern boundary of the main parcel.

4.3 Surficial Geology

The site is located adjacent to the western base of the Wasatch Range in an area mapped by Solomon (2007; **Figure 2**) as underlain mainly by lacustrine sand, silt, and gravel from Pleistocene Lake Bonneville (units Qlsp and Qlgb); and post-Lake Bonneville coalesced alluvial-fan deposits (units Qaf1, Qafy and Qafp). Solomon (2007; **Figure 2**) also maps a west-dipping trace of the Weber segment of the WFZ trending generally following the western boundary of the main parcel and an antithetic fault trace crossing the western three-lot parcel. Small areas of Holocene to upper Pleistocene alluvium and colluvium (unit Qac) and landslide deposits (units Qms1 and Qmsy) are also mapped by Solomon (2007) along the main trace of the Weber segment that partly extend into main parcel (**Figure 2**).

Solomon (2007) describes surficial units in the site vicinity, from youngest to oldest in age, as follows:

> ***Qms1*** -- ***Landslide deposits, unit 1 (Historical to Middle Holocene)***. Very poorly sorted clay, silt, sand, and minor gravel; grain size and texture varies with the nature of the deposits in the source area; unit 1 landslides occur in bluffs bordering the Weber River flood plain and north of Kaysville along steep bluffs incised by drainages following regression of Lake Bonneville; characterized by moderately fresh scarps and hummocky topography, with freshest scarps in areas of historical movement. Most unit 1 landslides along the Weber River originate in sandy Lake Bonneville deltaic deposits, form gently sloping alluvial-fan shaped deposits overlying unit 2 landslide deposits on river terraces that fringe the bluffs, and apparently result from flow failure, perhaps liquefaction induced, of the bluff face; one landslide along the Weber River, the Cedar Bench landslide, is an earth slide resulting from fill failure on the slope bordering an irrigation pond (Solomon, 1999). Landslides along other drainages are found at and slightly below the gradational contact between overlying Lake Bonneville sand and silt and underlying silt and clay and are the result of a combination of rotational and translational earth slides; parts of several of these landslides have been historically reactivated, resulting in damage to roads and houses. Thickness of the deposits is highly variable.
>
> ***Qmsy*** – ***Younger landslide deposits, undivided (Historical to Upper Pleistocene).*** Very poorly sorted, clay to boulder-size gravel in a matrix of silt, sand, clay, and pebbles; grain size and texture varies with the

nature of the deposits in the source area. Younger landslide deposits are found near Adams, Hobbs, and Holmes Reservoirs and along the Wasatch Range front; deposits along the steep bluffs near the reservoirs are derived from Lake Bonneville deposits incised by drainages following regression of Lake Bonneville and resemble unit 1 landslide deposits but lack relatively fresh scarps; younger landslide deposits along the range front are derived from the Farmington Canyon Complex (Xfc) and either result from incision of drainages after Lake Bonneville regression, obscure the Bonneville shoreline, or overlie Lake Bonneville deposits; younger landslide deposits may include unit 1 and unit 2 landslide deposits that are undifferentiated because units complexly overlap; thickness of the deposits is highly variable.

Qaf1 – Modern (level 1) alluvial-fan deposits (upper Holocene). Poorly to moderately sorted, weakly to non-stratified, silt- to boulder-size sediment deposited principally by debris flows and debris floods at the mouths of mountain-front canyons; upper parts characterized by abundant boulders and debris-flow levees that radiate away from the fan apex; small, finer grained alluvial-fan deposits are found on gentle slopes in the northern part of the quadrangle, deposited by shallow, dispersive flows; equivalent to the younger part of Qafy but differentiated where deposits can be mapped separately; commonly overlies lacustrine and older alluvial-fan deposits; debris-flow and debris-flood hazards are highest on modem alluvial fans, their undivided equivalents within younger alluvial fans (Qafy), and debris-flow deposits (Qmf); less than 30 feet (9 m) thick.

Qac – Alluvial and colluvial deposits, undifferentiated (Holocene to upper Pleistocene). Poorly to moderately sorted, generally poorly stratified, clay- to boulder-size, locally derived sediment deposited in drainages along the Wasatch Range front by fluvial, rockfall, slope wash, and creep processes; generally, less than 30 feet (9 m) thick.

Qc – Colluvial deposits (Holocene to upper Pleistocene). Poorly to moderately sorted, angular, clay- to boulder-size, locally derived sediment deposited by rock fall, slope wash, and soil creep on moderate to steep mountain-front spurs above the Bonneville shoreline; most bedrock is covered by at least a thin veneer of colluvium, and only the larger, thicker deposits are mapped; maximum thickness about 20 feet (6 m).

Qafy – Younger alluvial-fan deposits, undivided (Holocene to upper Pleistocene). Poorly to moderately sorted, weakly to non-stratified, silt- to boulder-size sediment deposited principally by debris flows, debris floods, and streams; equivalent to modern and level 2 alluvial-fan deposits (Qaf1, and Qaf2.) that are undifferentiated because units are complexly overlapping or too small to show separately; also mapped where the age of Holocene alluvial-fan deposits is uncertain; upper parts of fans are locally deeply incised; occurs near canyon mouths along the mountain front; probably less than 40 feet (12 m) thick.

Qafp – Alluvial-fan deposits related to the Provo (regressive) phase of the Bonneville lake cycle (upper Pleistocene). Poorly to moderately sorted, silt- to cobble-size sediment, with local boulders, deposited principally by debris flows whose surfaces are graded to the Provo shoreline at the mouth of Bair Canyon; incised by active streams; probably less than about 40 feet (12 m) thick.

Qlsp - Lacustrine sand and silt related to the Provo (regressive) phase of the Bonneville lake cycle (upper Pleistocene). Fine- to coarse-grained lacustrine sand and silt with minor gravel deposited at and below

the Provo shoreline; locally overlain by a thin veneer of eolian sand; grades downslope into fine-grained Lake Bonneville deposits and laterally northward to sandy deltaic deposits (Qldp); typically thick bedded and well sorted; gastropods locally common; deposited in relatively shallow water in beaches and, at the mouths of larger canyons, in deltas that no longer retain distinctive morphology; as much as 50 feet (15 m) thick.

Qlgb – Lacustrine gravel and sand related to the Bonneville (transgressive) phase of the Bonneville Lake cycle (upper Pleistocene). Moderately to well-sorted, moderately to well-rounded, clast-supported, pebble to cobble gravel interbedded with lesser amounts of pebbly sand and clean sand, deposited between the Bonneville and Provo shorelines; thin to thick bedded; gastropods locally common in sandy lenses; locally partly cemented with calcium carbonate; typically forms a bench along the Bonneville shoreline near the base of the Wasatch Range; locally as much as 200 feet (60 m) thick.

Xfc – Farmington Canyon Complex (Early Proterozoic). Metamorphic and igneous rocks, not mapped in detail for this study; Bryant (1984) concluded that these rocks in the Kaysville quadrangle record a Late Archean metamorphic event, but Barnett and others (1993) and Nelson and others (2002) found evidence that metamorphism was Early Proterozoic, affecting rocks either initially deposited as Archean sediment or Early Proterozoic sediment incorporating material eroded from nearby Archean crust; Yonkee and Lowe (2004) supported this interpretation and provided further interpretations based on detailed mapping of the Farmington Canyon Complex in the adjacent Ogden 7.5-minute quadrangle.

References included in the above unit descriptions are not provided in this report, but are provided in Solomon (2007).

Detailed surficial geology of the Project at a scale of one-inch equals 150 feet (1:1,800) is indicated on **Plate 2** based on surficial observations (to the extent such areas were viewable); review of black and white aerial photography from 1937 (U.S. Geological Survey, frames 10-AAK_3-41 and 3-42), 2012 color orthophotography available from the Utah AGRC, and 2013 LIDAR imagery available from the Utah AGRC; and subsurface data from GBES (2008), Western Geologic (2017), and the trenches and test pits conducted for the current study. Units and interpretations generally follow those of Solomon (2007), except for unit Qh, which represents areas where surficial soils have been excavated and removed and/or are obscured by fill deposits transported from cuts. Our subsurface data also showed near-surface soils at the site have also been locally disturbed and mixed by agricultural operations.

4.3 Seismic Hazards

4.3.1 Strong Ground Motions

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. Mapped active faults within this distance include the East and West Cache fault zones; the Brigham City, Weber, Salt Lake, and Provo segments of the Wasatch fault zone; the East Great Salt Lake fault zone; the Morgan fault; the West Valley fault

zone; the Oquirrh fault zone; and the Bear River fault zone (Black and others, 2003).

4.3.2 Site Class

Utah has adopted the International Building Code (IBC) 2018, which determines the seismic hazard for a site based upon 2014 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). For site class definitions, IBC 2018 Section 1613.2.2 refers to Chapter 20, Site Classification Procedure for Seismic Design, of ASCE[1] 7-16 which stipulates that the average values of shear wave velocity, blow count and/or shear strength within the upper 100 feet (30 meters) be utilized to determine seismic site class. Given the subsurface soils encountered at the site to a depth to 101.5 feet, it is our opinion the site best fits Site Class D – Stiff Soil Profile (with data), which we recommend for seismic structural design.

4.3.3 Ground Motions

The 2014 USGS mapping utilized by the IBC provides values of peak ground, short period and long period spectral accelerations for the Site Class B/C boundary and the Risk-Targeted Maximum Considered Earthquake (MCE_R). This Site Class B/C boundary represents average bedrock values for the Western United States and must be corrected for local soil conditions at site grid coordinates of 41.035917 degrees north latitude and -111.901273 degrees west longitude. The following table and response spectra summarizes the peak ground, short period and long period accelerations for the MCE_R event, and incorporates appropriate soil correction factors for a Site Class D (with data) soil profile:

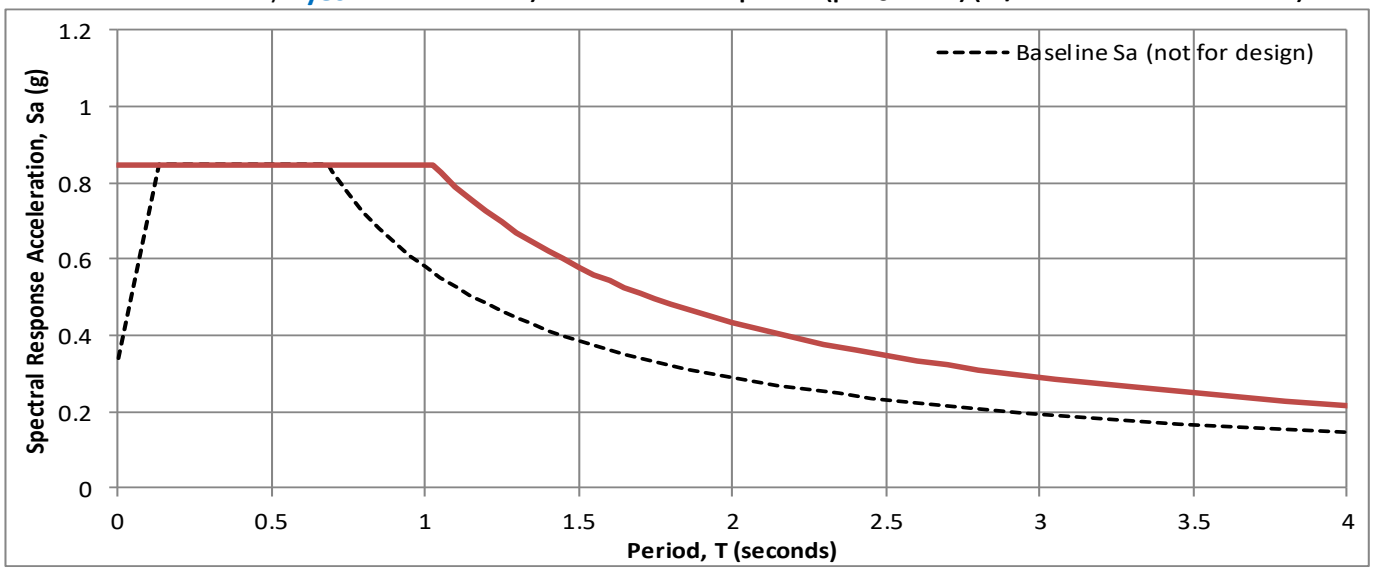
[1]American Society of Civil Engineers

SPECTRAL ACCELERATION PERIOD, T	SITE CLASS B/C BOUNDARY [mapped values] (g)	SITE COEFFICIENT	SITE CLASS D* [adjusted for site class effects] (g)	MULTI-PLIER	DESIGN VALUES (g)
Peak Ground Acceleration	PGA = **0.585**	F_{pga} = 1.100	PGA_M = 0.644	1.000	PGA_M = 0.644
0.2 Seconds (Long Period Acceleration)	S_S = **1.273**	F_a = 1.000	S_{MS} = 1.273	0.667	S_{DS} = 0.849
	(no exceptions needed)	F_a = (N/A)	S_{MS} = (N/A)	0.667	S_{DS} = (N/A)
1.0 Second (Long Period Acceleration)	S_1 = **0.477**	F_v = N/A	S_{M1} = N/A	0.667	S_{D1} = N/A
	(Exception 2:)	F_v = (1.823)	S_{M1} = (0.870)	0.667	S_{D1} = (0.580)

NOTES: 1. TL (seconds): **8** *** Site Class D With Data**

2. Site Class: **D** **4. ASCE 7-16 Requires Site-Specific Ground Motion Hazard Analysis (Since S1 ≥ 0.2 sec) - OR Can Use Exception 2 (per §11.4.8) (Sa/Cs Plot Assumes R=Ie=1.0)**

3. Have data to verify? **yes**



4.3.4 Liquefaction

Liquefaction is a phenomenon whereby loose, saturated, granular soil units lose a significant portion of their shear strength due to excess pore water pressure buildup resulting from dynamic loading, such as that caused by an earthquake. Among other effects, liquefaction can result in densification of such deposits causing settlements of overlying layers after an earthquake as excess pore water pressures are dissipated. Horizontally continuous liquefied layers may also have a potential to spread laterally where sufficient slope or free-face conditions exist. The primary factors affecting liquefaction potential of a soil deposit are: (1) magnitude and duration of seismic ground motions; (2) soil type and consistency; and (3) occurrence and depth to groundwater. Minimum Standards for Liquefaction Investigations and Evaluations are provided in Fruit Heights City Code Chapter 10.10A, Appendix D.

Anderson and others (1982) and Christenson and Shaw (2008) indicate the subject site is located in an area classified as having a "Very Low" to “Moderate” liquefaction potential. Groundwater was encountered at the site in boring B-4A (**Plate 1**) at a depth of 55.0 feet bgs, but was not encountered in any of the other CMT borings (depths of 30.5 to 51.0 feet) or the GSH borings (depths of 33.0 to 41.5 feet). Perched groundwater was also encountered in trench 8 for our investigation (**Figure 13**) with is estimated to be of shallow and limited thickness, but was not observed in any of the other trenches and test pits conducted for the current investigation or by GBES (2008) and Western Geologic (2017).

The natural soils at the encountered depth of water consisted of a cohesive, clayey silt/silty clay with some fine sand (designated as Lacustrine Transgressive Interbedded fines (Clays/Silts)). Gradation test results indicated up to 90 percent fines with a plastic index of 4 to 6 percent within this soil sequence. We strongly anticipate that well over 15 percent of the fines within this soil sequence are made up of clay and based on the "Liquefaction Susceptibility Criteria for Silts and Clays" report by Ross W. Boulanger, M. ASCE; And I.M. Idriss, M.ASCE (Journal of Geotechnical and Geoenvironmental Engineering ASCE/November 2006/1413) it is our opinion that these soil would not be suspectable to liquefaction.

Further, based on the cohesive soil type, the observed water depth, and the projected water levels down slope, interpolated from the study findings and existing well data, the shallowest projected water depths less than about 35 feet and generally more than 20 feet, are located near the existing fault line and downslope of the fault line. This is well downslope and west of the designated buildable areas (see cross section plates 4A through 4I of the report). It is our opinion that the liquefaction potential at the site would support a "low to very low" liquefaction potential.

4.3.5 Surface Faulting

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.

A main, west-dipping trace of the Weber segment of the WFZ trends northwestward generally following the western boundary of the main parcel and is marked by a steep, west-facing escarpment. An antithetic, east-dipping fault trace also crosses the detached three-lot parcel on the west. The faults are mapped on **Plate 2** based on air photo and subsurface evidence; the locations should be considered approximate, except near trenches where exposure confirm their location, but are not dashed on **Plate 2** for ease of viewing.

GBES (2008) excavated twelve trenches at the site and in offsite areas to the west, as located on **Plate 1**, to evaluate the risk from surface faulting to the inclusive property. The GBES (2008) study was conducted following Christenson and others (2003) prior to the current City of Fruit Heights SLEDS ordinance. A copy of the report is provided in Appendix B. No evidence for active faulting was observed in GBES' (2008) trenches, but GBES conducted no subsurface exploration to locate the main trace of the Weber segment. GBES (2008) instead inferred the fault was located within the lower half of the west-facing escarpment based on the available mapping, published reports, and their experience with similar faults. Based on a presumed displacement of 12 feet and dip of 60 degrees, a footing depth of 8 feet, and guidelines in Christenson and others (2003), GBES (2008) recommended no structures for human occupancy be located within 45 feet of the escarpment toe and within 40 feet of the escarpment crest.

To evaluate subsurface conditions and the risk from surface faulting at the former Raymond Property, Western Geologic (2017) excavated three trenches in 2015 as located on Plate 1. The investigation was conducted following guidelines in Christenson and others (2003) and in accordance with the Minimum Standards for Surface Fault Rupture Hazard Studies provided in Appendix B of Fruit Heights City Code Chapter 10.10A. A copy of the report is provided in Appendix B. Western Geologic (2017) trench 2 exposed evidence for five active faults associated with the main trace of the Weber segment of the WFZ crossing the main parcel, and trench 3 exposed evidence for active faulting associated with the antithetic fault trace crossing the detached three-lot parcel. Western Geologic (2017) inferred total displacement across the main fault was a total of about 50 feet, which based on data in DuRoss and others (2009) may be from six or more post-Lake Bonneville earthquakes.

Western Geologic (2017) provided fault-specific calculated setback distances based on guidelines in Christenson and others (2003), relevant fault data from their trenches, and a footing depth of 10 feet. The table below summarizes Western GeoLogic's (2017) fault data and calculated setback distances (in feet). Fault dips are in degrees. Western GeoLogic's (2017) trenches and active faults encountered are displayed on Plate 1. The station indicates the fault location the trench, as shown on the logs in Appendix B. Active fault locations were surveyed by Reeves & Associates on July 17, 2015. Tim Thompson, P.G. of GeoStrata reviewed on the trenches on the same day.

WESTERN GEOLOGIC (2017) FAULT DATA AND SETBACK CALCULATIONS

Trench	Fault	Station	Dip	Offset (feet)	Dip Direction	Calculated Downthrown Setback	Calculated Upthrown Setback	Downthrown Setback Distance	Upthrown Setback Distance
2	F1	0+7.2	52	3.0	West	20.7	9.0	20.7	15.0
2	F2a	0+15.8	55	0.5	West	12.0	1.5	15.0	15.0
2	F2b	0+17.7	61	8.0	West	32.3	24.0	32.3	24.0
2	F3	0+51.3	63	3.5	West	18.1	10.5	18.1	15.0
2	F4	0+61.6	58	2.6	West	17.2	7.8	17.2	15.0
2	F5	1+4.3	70	3.5	West	16.0	10.5	16.0	15.0
3	AF1	2+18.7	59	1.6	East	13.8	4.8	15.0	15.0

Eight trenches and five walk-in test pits (short trenches) were excavated in the northern half of the site for the current investigation to evaluate subsurface conditions and the risk from surface faulting. Locations are shown on **Plate 1**. The current study was conducted following Christenson and others (2003) and the Minimum Standards for Surface Fault Rupture Hazard Studies provided in Appendix B of Fruit Heights City Code Chapter 10.10A, except that no trenching was conducted in the escarpment bounding the west side of the main parcel, which was too steep for exploration or development in the northern half of the site. Logs of the test pits and trenches at a scale of one-inch equals five feet (1:60) are displayed on **Figures 5 through 13**. Due to trench lengths and space constraints, the trench logs are displayed on multiple sheets. Stratigraphic interpretations and descriptions are provided on the logs.

Portions of the Project have a high risk from surface faulting based on the subsurface data in Western Geologic (2017) and obtained for the current investigation. **Plate 4** shows the surface fault rupture hazard at the site. A crosshatched zone is shown on **Plate 4** around the main fault along the western boundary of the main parcel, and around the antithetic fault crossing the detached three-lot parcel. No structures intended for human occupancy should be located in the crosshatched setback zone on **Plate 4**. It is generally accepted practice to allow streets, driveways, yards, and other non-occupied, non-attached structures to be constructed within these areas. The setback zone for the antithetic fault (AF1, **Plate 4**) crossing the detached three-lot parcel is based on Western GeoLogic's (2017) calculated distances in the table above. The setback zone for the main fault is also based on the Western GeoLogic's (2017) calculated distances south of their trench 3, but further north the setback zone has been widened to correspond to the west end of GBES (2008) trench I; CMT trench 7; CMT walk-in test pit 11; CMT trench 5; GBES (2008) trench J; CMT walk-in test pit 12; CMT trenches 2, 3 and 4; CMT walk-in test pits 14 and 15; and CMT trench 1 (**Plate 4**). North of CMT trench 1, the setback zone east of the main fault is based on double (2X) Western GeoLogic's (2017) largest upthrown-side setback distance (thus, 48 feet; **Plate 4**). Except for portions of lot 29, lots 34 to 38, and lots 42 to 44, the eastern edge of the setback zone is generally west of the escarpment crest where steep slopes would prevent building (**Plate 4**).

4.4 Landslide and Slump Deposits

Landslides, slumps, and other mass movements are gravity-induced downslope movements of rock or soil. Such failures may be both deep and shallow seated. Deep-seated failures include rotational and translational slides and associated earthflows where the failure plane is more than 10 feet deep (Varnes, 1978; Cruden and Varnes, 1996). Landslides can develop in 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.

Plate 6 shows the landslide hazard at the site. Based on air photo evidence and subsurface data, we mapped five small slumps at the site on **Plates 2 and 6** in the west-facing escarpment associated with the main trace of the Weber segment of the WFZ. The escarpment is the result of multiple post-Lake Bonneville large-magnitude earthquakes and vertical displacement of the lacustrine sediments. GBES (2008) recognized three of the five slumps. **Plates 2 and 6** also show a landslide that may be contemporaneous with Lake Bonneville slightly east of the southeast part of the main parcel. The landslide deposit does not extend into the site, but the failure initiated in slopes similar to those east of lots 84 and 85 (**Plate 6**). No other evidence of landsliding or ongoing slope instability was identified at the Project.

4.5 Sloping Surfaces

Surface slopes at the Project developed from our LIDAR analysis are shown on **Plate 3**. Slope areas steeper than 30% are shaded in red and are not suitable for development per Fruit Heights City Code Chapter 10.10A.040. Such slopes bound the east and west sides of the Project associated with the Wasatch Range front and steep escarpment along the WFZ, and are also found locally associated with historical man-made surficial disturbance. The bench area between the range front on which most of the Project is located generally has slopes gentler than 20%, which are unshaded. Slopes between 20% to 30% steepness are found locally and shaded in yellow.

Nine geologic cross sections (*A-A', B-B', C-C', D-D', E-E', F-F, G-G', H-H' and I-I''*) are shown on **Figures 4A through 4I** across various representative slope sections at the Project based on proposed site grading from the project Civil Engineer Reeve and Associates. The cross sections were based on the subsurface soil conditions encountered within the nearby associate test pits, bore holes, and trenches completed across this project included in this study as well as the attached referenced studies completed for this site. The bedrock depths on the east are based on where it was encountered in borings, present in the gravel pit, or expressed at the surface. A dip of about 5:1 westward to the fault was estimated for the bedrock based on observed conditions. For estimated groundwater depth, a depth of 55 feet at the scarp crest (from B-4A) and 15 feet at the slope toe was utilized. Westward, the groundwater appears to be at a hydraulic gradient of 16.5% based on the Weber Basin Water Conservancy District well (233 feet bgs) at the corner of Green Street and Highway 89. Eastward, the groundwater follows topography.

Cross section C-C' was based on subsurface data from CMT test pits 6, 7 and 17; CMT boring B-2 and surficial observations. We note that much of cross section *B-B'* is unavoidably inferred because no subsurface data could be obtained east of the Project, which is owned by the USFS. Contacts and units should be considered inferred

and approximate for all the cross sections, though we consider them to generally be representative of the subsurface conditions.

Geotechnical analysis of CMT cross sections *A-A'* through *F-F'* is presented in Section 7.0 of this report following Fruit Heights City Code Chapter 10.10A, Appendix C, Minimum Standards for Slope Stability Analyses.

4.6 Alluvial-Fan Flooding

Alluvial-fan flooding refers to a continuum of processes that includes debris slides, debris flows, debris floods, and flash flooding on alluvial fans (National Research Council, 1996). Debris flows and related sediment-enriched floods and flows are fast moving flow-type landslides comprised of a slurry of rock, mud, organic matter, and water that move down drainage-basin channels onto alluvial fans (Giraud, 2005). Debris flow hazards are commonly associated with areas underlain by Holocene alluvial-fan deposits at the mouths of range-front drainages, such as those along the Wasatch Range.

Solomon (2007) maps much of the Project in areas underlain by a series of post-Lake Bonneville coalesced alluvial fan deposits (units Qaf1 and Qafy, **Figure 2**). However, our subsurface data and the site-specific surficial geologic mapping on **Plate 2** instead show that most of the Project is underlain by sand and gravel associated with the regressive (Provo) stage of Lake Bonneville. Based on our air photo review and mapping, nine alluvial fans impact the eastern part of the main parcel at the Project emanating from drainage basins to the east. Alluvial fans, drainages, and drainage basins impacting the Project are indicated on **Figure 3, Drainages and Alluvium**.

GBES (2008), Western GeoLogic (2017), and GeoStrata (2020) have conducted debris-flow hazard evaluations for the inclusive site, the former Raymond Property (south part of the Project), and in response to the September 2019 Francis Fire. GBES (2008) identified seven alluvial fans at the site that posed a risk from alluvial-fan flooding. Based on subsurface data, GBES (2008) indicated only their fans B and E showed evidence for Holocene flows. GBES' (2008) fans A and B appear to consist of two distinct alluvial fans with separate drainage basins. Western GeoLogic (2017) mapped three alluvial fans at the former Raymond Property, but indicated that additional drainages (erosional gullies) entered the site further south that either had limited alluvial fans or were indistinct. Western GeoLogic's (2017) alluvial fans 1, 2 and 3 correspond to our mapped fans D, C and B (respectively), and GBES' (2008) fans D, E and F. In September 2019, the drainage basins east of the Project were impacted by a wildfire termed the Francis Fire. GeoStrata (2020) conducted a comprehensive debris flow hazard assessment of the drainage basins east of the Project that evaluated potential impacts from this fire and provided debris flow volumes calculated using both empirical and post fire debris-flow methods. The table below provides a summary of the drainage basins and calculated debris volumes from the above studies, as well as calculations of peak flow, runoff volume, estimated velocity and runup heights provided by Reeve & Associates. All of the volume estimates are applicable mainly for debris basin control structures. However, GeoStrata (2020) reported that discussions between the USFS and City of Fruit Heights indicate the USFS will give no permission to improve or maintain areas adjoining the site on the east. Use of debris basins thus does not appear feasible. GeoStrata (2020) instead recommended mitigation include debris flow fencing, berms and/or other structures. Drainage basin and alluvial fan data are provided below.

DRAINAGE BASIN AND ALLUVIAL FAN DATA

Basin	Area (acres)	Alluvial Fan	Area (acres)	GBES Fan	Relevant Subsurface Exposure	Holocene Activity?	Flow Thicknesses
A	2.59	A	0.47	G	GBES test pit 12	No	Unknown
B	4.13	B	0.67	F	GBES test pits 10 and 11	No	Unknown
C	30.31	C	1.30	E	GBES test pits 8 and 9	Yes	1.6 ft for last flow, 6.0 ft total last 6-8 flows.
D	8.76	D	1.40	D	Western GeoLogic test pit 1	Yes	0.8-2.2 ft each, last three flows.
E	3.73	E	0.88	C	GBES test pit 5, CMT test pit 10	No	Unknown
F	41.72	F	1.66	B	GBES test pit 2, CMT test pit 8	Yes	1.8 ft for last flow; 6.5 ft total last two flows.
G	4.54	G	1.43	B	GBES test pit 1, CMT test pits 7 and 17	Yes	1.5 ft total last 2 flows; 1.2-1.6 ft each, last 3 flows.
H	51.33	H	4.28	A	CMT test pits 4 and 5	Yes	3.1-3.3 ft for last flow.
I	22.03	I	6.87	A	CMT test pit 16	Yes	2.1-2.2 ft each, last 2 flows.

DRAINAGE BASIN AND FLOW/VOLUME CALCULATIONS

					Estimated Debris Volumes				Difference[2]			Calculated Runoff Volume[5]			
Basin[1]	Basin[2]	Basin[3]	Basin[4]	Basin Area[2] acres	Pre-Fire[2] acre-ft	Pre-Fire[3] acre-ft	Pre-Fire[4] acre-ft	Post-Fire[2] acre-ft	Pre-Fire acre-ft	Post-Fire acre-ft	Peak Flow[5] cfs	cf	acre-ft	Velocity[5] ft/s	Runup Height[5] ft
A	A		G	2.59	0.05			0.3	-0.13	0.25	1.87	7655	0.18	5.48	0.47
B	B	3	F	4.13	0.06			0.5	-0.30	0.44	3.98	15734	0.36	5.48	0.47
C	C	2	E	30.31	0.63	1.24	0.87	1.8	-2.64	1.17	31.48	142315	3.27	5.48	0.47
D	D	1	D	8.76	0.04	1.11		0.8	-0.70	0.76	7.54	32403	0.74	5.48	0.47
E	E		C	3.73	0.11			0.5	-0.14	0.39	2.60	11015	0.25	5.48	0.47
F	F		B	41.72	1.23		3.72	2.2	-3.58	0.97	43.29	209454	4.81	5.48	0.47
G	G		B	4.54	2.74			0.5	2.36	-2.24	4.10	16473	0.38	5.48	0.47
H	H		A	51.33	0.36		3.47	2.4	-5.21	2.04	47.31	242438	5.57	5.48	0.47
I	I		A	22.03	0.50			1.2	-1.22	0.70	14.81	75080	1.72	5.48	0.47

cfs = cubic feet per second, cf = cubic feet, acre-ft = acre feet, ft/s = feet per second, ft = feet
[1] This report (Figure 3).
[2] From GeoStrata (2020).
[3] From Western GeoLogic (2017).
[4] From GBES (2008).
[5] Calculated by Reeve & Associates. Peak flow based on SCS TR-55 method with weighted C values, runup height based on H=((U0)^2)/2g.

The hazard from debris flows and alluvial-fan flooding with respect to the proposed development, as well as recommended mitigation to reduce the risk to impacted lots, is shown on **Plate 5, Debris Flow Hazard**. To reduce the risk from alluvial-fan flooding for lots situated on the alluvial fans on **Plate 5**, we recommend: (1) debris fencing be installed on the upslope lot boundary; (2) building pads be raised at least the distance shown (in inches) and homes include no upslope-facing, below-grade entryways that could pose a pathway for flooding to enter subgrade areas; and (3) that channels be installed along the upslope and lateral lot boundaries to direct flooding around homes. With regard to raised building pads, the recommended distance is based on the flow thicknesses shown on the upper table plus the runup determined by Reeve & Associates shown on the lower table. Given that depositional thickness would be lower on a fan margin, we recommend using half the flow thickness plus 6 inches for lots that straddle a lateral fan margin. The minimum recommended distance is 24 inches (or 15 inches for a lot on a fan margin). A typical lot mitigation schematic is shown in the upper right-hand corner of **Plate 5**. The intent of the debris fencing is to filter out large debris while allowing muddy water to enter the cutoff and side channels and flow to the storm sewer and thence to an onsite retention basin, which may be accomplished using various options. A homeowners association will reportedly own and maintain the debris basin. The fencing, cutoff channels, and side channels may also require repair or maintenance following a future flow by the homeowners association.

4.7 Flooding Hazards

No active drainages or significant waterways were evident crossing the during our reconnaissance or on air photos. However, several ephemeral drainages enter the site from higher slopes to the east that formerly continued across the property to the southwest but have been altered by man-made disturbance. These drainages are likely only active following high-intensity storms and spring snowmelt. Federal Emergency Management Agency flood insurance rate mapping (available at https://msc.fema.gov/portal/) classifies the Project mainly in "Zone X - Area of Minimal Flood Hazard". However, a narrow area of “Zone A” is mapped crossing the Project that has been modified by agricultural grading and is associated with drainage basin F on **Figure 3**. Local sheet flow, slope wash, and seasonally perched soil water should be anticipated for below-grade features on the site and for site improvements.

4.8 Rockfall Hazards

No boulders likely from rock falls were observed at the site and no detailed rockfall hazard analyses were performed. Scattered weathered outcrops of schist of the Precambrian Farmington Canyon Complex are in higher slopes east of the property, but these outcrops have low relief and are likely incapable of generating large rock fall boulders due to their composition. Except for in areas associated with man-made disturbance, GBES (2008) also did not identify any potential rock fall hazard areas within the Project boundaries.

5.0 SITE CONDITIONS

5.1 Surface Conditions

The site conditions and site geology were interpreted through an integrated compilation of data, including a review of literature and mapping from previous studies conducted in the area (GBES, 2008; Western GeoLogic, 2017); photogeologic analyses of 2012 imagery shown on **Figure 2**; historical stereoscopic 1:20,000 imagery flown in 1937 (frames AAK_3-41 and 3-42); GIS analyses of elevation and geoprocessed LIDAR terrain data as shown on **Plate 3**; field reconnaissance of the general site area; and the interpretation of the test pits, trenches and borings made on the site as part of our field program. Seismic hazards information was developed from United States Geologic Survey (USGS) databases (Peterson and others, 2008). The geologic cross sections shown on **Figures 4A through 4I** were interpolated from subsurface data and the detailed surficial geologic mapping shown on **Figure 2**. The topographic profiles were drawn from proposed grading data provided by the project Civil engineer (Reeve and Associates).

As shown on **Plate 1**, the site consists of 67.03 acres of land in size that is presently vacant and partly developed for use as orchards. Vegetative cover at the site consists primarily scattered oak and sage brush, fruit trees, and grasses. As shown on **Plates 2 and 3**, the site is bounded on the east and west by slopes exceeding 30% steepness, although the bench between these slope areas is relatively gentle. Significant portions of the eastern and northeastern parts of the site have been disturbed by historical gravel mining or other operations, and much of the site has also been disturbed by agricultural tilling/grading. The adjoining land on the north, east and west is undeveloped. The area on the west is generally developed for residential and religious uses. An easement crosses the main parcel at the Project for a buried aqueduct. No exploration was conducted across the aqueduct easement.

5.2 Subsurface Soils

Seventeen test pits and eight trenches were excavated for the current study to evaluate subsurface soil conditions. A summary of subsurface conditions encountered in these test pits and trenches is provided below. Detailed stratigraphic unit descriptions are shown on the test pit and trench logs (**Figures 5 through 13**). Trench and test pit locations were measured using a hand-held GPS unit and by trend and distance methods from known points. The trenches and test pits were logged following methodology in McCalpin (1996) and digitally photographed at five-foot intervals to document the exposures. The photos are not provided herein but are available on request. Logging followed methodology in McCalpin (1996). No evidence for active faulting was observed in the trenches. Tim Thompson, P.G. of GeoStrata reviewed trenches 1 through 6 on December 21, 2018 and trenches 7 and 8 on September 2, 2020.

Exposure	Subsurface Soils
Test pit 1	Fill overlying Lake Bonneville sand and gravel (SW/GW).
Test pit 2	Fill overlying a sequence of Lake Bonneville gravel, sand and silt units (GW, SW and SM/ML; from oldest to youngest).
Test pit 3	Fill overlying weathered schist bedrock of the Farmington Canyon Complex.

Test pit 4	Sandy alluvium (SW) overlying a sequence of Lake Bonneville sand and gravel units (GW and SW; from oldest to youngest).
Test pit 5	Sandy alluvium (SW) overlying a sequence of Lake Bonneville sand and gravel units (SW-SP, GW and SW/GW; from oldest to youngest).
Test pit 6	Lake Bonneville sand and gravel (SW/GW).
Test pit 7	Sandy alluvium (SW) overlying Lake Bonneville sand and gravel (SW/GW).
Test pit 8	Sequence comprised of an upper sandy alluvium (SW), a middle coarse granular debris flow deposit (SW/GW), and a lower Lake Bonneville sand and gravel (SW/GW).
Test pit 9	Sandy to gravelly alluvium (SW/GW).
Test pit 10	Sandy alluvium (SW) overlying Lake Bonneville sand (SW).
Test pit 11	Sequence of Lake Bonneville deposits comprised of a beach sand deposit (SW/GW) overlying laminated silt (ML).
Test pit 12	Sequence of Lake Bonneville deposits comprised of an upper silty sand (SM), a middle beach sand deposit (SW/GW) and a lower laminated silt (ML).
Test pit 13	Sequence of Lake Bonneville deposits comprised of an upper sandy gravel (SW), a middle silty sand to sandy silt (SM/ML) and a lower beach sand deposit (SW/GW).
Test pit 14	Sequence of Lake Bonneville deposits comprised of an upper silty sand (SM), a middle beach sand deposit (SW/GW) and a lower laminated silt (ML).
Test pit 15	Sequence of Lake Bonneville deposits comprised of interbedded sand and silt (SM/ML) overlying laminated silt (ML).
Test pit 16	Sequence of pose-Lake Bonneville debris flow and flood deposits comprised of a lower sandy gravel to gravelly sand (GW/SW) and an upper gravelly sand with silt (SW) overlying lacustrine sand and gravel (SW/GW).
Test pit 17	Fill overlying a sequence of Holocene debris flow and flood deposits comprised of silty sand (SM), sandy gravel to gravelly sand (GW/SW) and gravelly sand (SW), from youngest to oldest.
Trench 1	Fill overlying a sequence of several Lake Bonneville silt to gravel units (ML, SW/GW, ML, GW, SM, SW/GW and GW; from oldest to youngest)
Trench 2	Fill overlying a shallow-seated slump deposit (SW), in turn overlying a sequence of Lake Bonneville deposits (ML, SW/GW and ML; from oldest to youngest).
Trench 3	Fill overlying a shallow-seated slump deposit (SW), in turn overlying a sequence of Lake Bonneville silt to gravel units (ML, SW/GW, ML, GW, SM, SW and SW/GW; from oldest to youngest).
Trench 4	Fill overlying a shallow-seated slump deposit (SW), in turn overlying a sequence of Lake Bonneville silt to gravel units (ML, SW/GW, ML, GW, SM, SW and SW/GW; from oldest to youngest).
Trench 5	Fill overlying a sequence of Lake Bonneville silt to gravel units (ML, SW/GW, ML, SW/GW, SM, SW; from oldest to youngest).
Trench 6	Fill overlying Lake Bonneville laminated silt (ML).
Trench 7	Fill overlying landslide colluvium (SM/SW), in turn overlying a sequence of lacustrine sediments (ML and GW/SW).
Trench 8	Fill overlying landslide colluvium (GW/SW0, in turn overlying lacustrine sandy clay to clayey sand (CL/SM); groundwater at 7.5 feet bgs in east trench end.

In addition, 5 borings were completed across the site to depths of about 30 to 101.5 feet, with most of them encountering auger refusal. Highly weathered bedrock was encountered in borings B-1 and B-2 at depths of about 39 and 29 feet below the ground surface, respectively. The subsurface soils encountered varied somewhat across the site. At bore hole B-5 the upper 7 feet is blanketed with non-engineered (undocumented)

fill soils comprised of Silt and Silty Sand underlain by natural Silty Sand to a depth of about 18 feet. From 18 feet to the full depth penetrated, 30 feet, natural Silt and Clay soils were encountered with varying sand and gravel content.

At borings B-1 through B-3 the subsurface soils consisted primarily of Sand with varying Silt and Gravel content with occasional layers of Sandy Silt extending to the full depths penetrated, about 30.5 to 45.5 feet. At bore hole B-4A, (bore hole B-4A replaces bore hole B-4 as it extended deeper to 101.5 feet) natural soils encountered consisted primarily Clay soil extending to about 15 feet below the ground surface underlain by Sand soils with varying fines to about 30 feet. From about 30 feet to about 70 feet a Silt-Clay soil layer was encountered grading to Silt to a depth of about 85 feet below the ground surface. From 85 feet extending to the full depth penetrated, 101.5 feet, Silty Clay with Sand layers was encountered.

The natural sand and gravel soils were generally medium dense to dense, slightly moist to moist, brown in color, and will exhibit moderate high strength and low compressibility characteristic. The natural silt and clay soils were generally stiff to hard, moist to wet, brown and, based on limited lab testing, appear to be moderately over consolidated exhibiting moderate strength and compressibility characteristics.

5.3 Groundwater

Groundwater was encountered at depth of 55.0 feet in boring B-4A, but was not encountered in any of the other borings conducted for the current investigation to their explored depths. GSH (2014) also did not encounter groundwater in any of the borings conducted on the former Raymond Property. Perched groundwater was observed in trench 8 at a depth of 7.5 feet bgs, but was not observed in any of the other test pits or trenches conducted for our investigation, or by GBES (2008) and Western GeoLogic (2017). Groundwater levels may vary annually from climatic fluctuations and seasonally from snowmelt runoff and man-made sources such as landscape irrigation. Future seasonal and longer-term groundwater fluctuations should be anticipated for the site, with the highest seasonal levels generally occurring during the late spring and summer months. Perched conditions above less-permeable stratigraphic layers may be locally present in the subsurface that could cause localized shallower groundwater levels, as indicated by trench 8. We anticipate that groundwater will be at depths similar to that observed in boring B-4A or deeper as long as surface water infiltration from landscape irrigation or other sources is strictly controlled. We therefore recommend the homeowners association mandate use of low-irrigation landscaping.

5.4 Site Subsurface Variations

Based on the results of the subsurface explorations and our experience, some variations in the continuity and nature of subsurface conditions should be anticipated. Due to the heterogeneous characteristics of natural soils, caution should be taken in interpolating or extrapolating subsurface conditions beyond the exploratory locations. Seasonal fluctuations in ground water conditions may also occur.

In addition, once the subsurface explorations were completed the test pits and trenches were backfilled with the excavated soils but little effort was made to compact these soils. Excavation backfill soils must be considered

non-engineered. Settlement of the backfill in the excavations over time should be anticipated and caution should be exercised when constructing over these locations.

Further, fills, considered to be non-engineered, have been placed across the site and are anticipated to vary both laterally and with depth across the site.

6.0 SITE PREPARATION AND GRADING

6.1 General

Initial site preparation shall consist of the removal of surface vegetation, topsoil, any other deleterious materials, and loose/disturbed surface soils from beneath an area extending out at least 4 feet from the perimeter of the proposed buildings and 2 feet beyond pavements and exterior flatwork areas.

Further, all non-engineered fills shall be removed from below an area extending out 4 feet from the perimeter of the proposed buildings. However, existing in-situ fill soils/disturbed soils may remain below exterior pavement areas if: free of debris and any deleterious material; no more than 5 feet in total thickness; subsequent overlying site grading fills are limited to 3 feet; and they are properly prepared. Proper preparation will consist of the scarification of the upper 18 inches below asphalt concrete (flexible pavement) and 24 inches below rigid pavement followed by moisture preparation and re-compaction to the requirements of structural fill. This will require the temporary removal of 12 to 18 inches of fill, scarifying, moisture conditioning, and recompacting the underlying 6 to 9 inches and backfilling with compacted suitable fills. Where fine grained (clays/silts) fills are present, please note that such soils are moisture sensitive and inherently difficult to adequately moisture prepare and re-compact. These fine-grained soils may become near impossible to re-compact during cold and wet periods of the year. As an alternative, the fills/disturbed soil may be removed and replaced with imported granular structural fill over unfrozen, proof-rolled subgrade. Any loose fill piles across the surface must be completely removed.

Subsequent to stripping and prior to the placement of floor slabs, foundations, structural site grading fills, exterior flatwork, and pavements, the exposed subgrade must be proof rolled by passing moderate-weight rubber tire-mounted construction equipment over the surface at least twice. If excessively soft or otherwise unsuitable soils are encountered beneath footings, they must be completely removed. If removal depth required is greater than 2 feet below footings, CMT must be notified to provide further recommendations. In pavement, floor slab, and outside flatwork areas, unsuitable natural soils should be removed to a maximum depth of 2 feet and replaced with compacted granular structural fill. Non-engineered fills shall be handled as described above.

Structures may be constructed directly on bedrock, if encountered. However, in situations where structures will be partly on soil and partly on bedrock, we recommend that site grading be adjusted or additional excavation of bedrock be completed such that a minimum 12 inches of suitable natural undisturbed soil remains, or that structural fill be placed directly over the portion of exposed bedrock, to provide a uniform bearing condition for the structure.

Landslide deposits are mapped within the buildable portions of Lots 26, 27, 28, 40, 41 and 42. Based on our field investigation, the landslide deposits are relatively shallow in depth. Landslide deposits must be completely removed within the boundaries of Lots 26 through 28, and 40 through 42, and otherwise removed below proposed structures and extending downslope a minimum of about 20 feet beyond the homes. Additional site grading fills of significant thickness/greater than about 2 feet, shall not be placed across lots 26 through 29.

Further, it is our understanding that mass grading cuts will be required to achieve adequate road design which will include significant cuts across lots 41 through 49. Following the completion of the planned mass grading cuts, additional soil subgrade observations and slope stability analysis shall be completed by CMT for these lots. Where cut slopes may be steepened greater than about two and one-half horizontal to one vertical (2.5H:1V), additional individual slope stability analysis may become necessary before home construction on these lots. Our additional findings will be presented in a subsequent addendum.

A representative of CMT must verify that suitable natural soils and proper preparation of existing soils have been encountered/met prior to placing site grading fills, footings, slabs, and pavements.

Fill placed over large areas to raise overall site grades can induce settlements in the underlying natural soils. Further, if placed on lots near the west crest they may create instability. A slope stability analysis has been completed based on current planned grading. If the grading plans change, CMT must be allowed to review the new grading conditions.

Where planned site grading fills will be greater than about 10 feet, then a field settlement monitoring procedure shall be put in place. This may include the placement of a series of settlement plates installed within the backfill to facilitate monitoring settlement as it occurs; these settlement devices typically consist of steel plates placed at or near the bottom of the fill, with a 1-inch diameter rebar or steel pipe attached to the plate and extending up through the fill to the surface where they can be periodically surveyed (at least twice weekly). Further, monuments would be installed near or at the surface of the fill to monitor any settlement within the fill soil sequence. Settlement plates should be placed within areas such as roadways and utility corridors at a spacing of 500 feet or less. To mitigate the potential impact of this settlement on utilities, consideration should be given to placing structural fill as far as possible (preferably at least 3 months) in advance of construction

6.2 Temporary Excavations

Temporary construction excavations not exceeding 4-feet in depth in the fine-grained cohesive soils may be constructed with near-vertical sideslopes. Temporary excavations up to 8-feet deep in fine-grained cohesive soils, above or below the water table, may be constructed with sideslopes no steeper than three-quarter horizontal to one vertical (0.75H:1V).

For granular (cohesionless) soils, construction excavations above the water table, not exceeding 4-feet, should be no steeper than one-half horizontal to one vertical (0.5H:1V). For excavations up to 8-feet, in granular soils and above the water table, the slopes should be no steeper than one horizontal to one vertical (1H:1V). Excavations encountering saturated cohesionless soils will be very difficult and will require very flat sideslopes

and/or shoring, bracing, and dewatering. Further, if excessive sloughing occurs or where extensive layers of clean granular soils are encountered, the sideslopes should be appropriately flattened. Excavations deeper than about 8-feet are not anticipated at the site excluding initial mass grading.

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. All excavations should be made following OSHA safety guidelines.

6.3 Permanent Cut/Fill Slopes

In general, the natural soils encountered varied across the site from fine grained (silt/clay) soils with sand layers over bedrock within the southwestern portion of the site, grading to more sandy/gravelly soils overlying bedrock to the east/northeast. Based on our slope stability modeling, recommended permanent cut and fill slopes must be graded no steeper than two- and one-half horizontal to one vertical (2.5H:1V) to be considered stable. If steeper cut and fill slopes are required to facilitate development plans, these areas must be addressed on a case by case basis and retaining walls or shoring must be anticipated. If bedrock is encountered the permanent slope restrictions may be reevaluated.

6.4 Detention Basin

A detention basin is planned at the north end of the development as shown on **Plate 1**. This area is presently located with a previous aggregate quarry which will require significant grading prior to development. The overall general area slopes to the west/southwest. Where possible, the detention basin should be excavated to form the detention area, thereby reducing fill embankments and slopes. For the basin we recommend slopes no steeper than three horizontal to one vertical (3H:1V). Further the detention basin shall be lined with clay or a synthetic liner and retained water piped through a closed system down gradient to a storm sewer system.

6.5 Fill Material

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 possibly as replacement fill below footings. All structural fill must be free of sod, rubbish, topsoil, frozen soil, and other deleterious materials.

Following are our recommendations for the various fill types we Project will be used at this site:

FILL MATERIAL TYPE	DESCRIPTION | RECOMMENDED SPECIFICATION
Structural Fill	Placed below structures, flatwork and pavement. Well-graded sand/gravel mixture, with maximum particle size of 4 inches, a minimum 70% passing 3/4-inch sieve, a maximum 20% passing the No. 200 sieve, and a maximum Plasticity Index of 10.
Site Grading Fill	Placed over larger areas to raise the site grade. Sandy to gravelly soil, with a maximum particle size of 6 inches, a minimum 70% passing 3/4-inch sieve, and a maximum 50% passing No. 200 sieve.
Non-Structural Fill	Placed below non-structural areas, such as landscaping. On-site soils or imported soils, with a maximum particle size of 8 inches, including silt/clay soils not containing excessive amounts of degradable/organic material (see discussion below).
Stabilization Fill	Placed to stabilize soft areas prior to placing structural fill and/or site grading fill. Coarse angular gravels and cobbles 1 inch to 8 inches in size. May also use 1.5-inch to 2.0-inch gravel placed on stabilization fabric, such as Mirafi RS280i, or equivalent (see **Section 6.6**).

On-site natural granular soils or existing granular fills may be utilized as structural site grading fill provided, they meet with the requirements as stated within this report.

On-site silt/clay soils may be used as site grading fill and non-structural fill, but are also moisture-sensitive. Note that such moisture-sensitive soils are inherently more difficult to work with in proper moisture conditioning (they are very sensitive to changes in moisture content), requiring very close moisture control during placement and compaction. This will be very difficult, if not impossible, during wet and cold periods of the year.

All fill material should be approved by a CMT geotechnical engineer prior to placement.

6.6 Fill Placement and Compaction

The various types of compaction equipment available have their limitations as to the maximum lift thickness that can be compacted. For example, hand operated equipment is limited to lifts of about 4 inches and most "trench compactors" have a maximum, consistent compaction depth of about 6 inches. Large rollers, depending on soil and moisture conditions, can achieve compaction at 8 to 12 inches. The full thickness of each lift should be compacted to at least the following percentages of the maximum dry density as determined by ASTM D-1557 (or AASHTO[2] T-180) in accordance with the following recommendations:

[2] American Association of State Highway and Transportation Officials

LOCATION	TOTAL FILL THICKNESS (FEET)	MINIMUM PERCENTAGE OF MAXIMUM DRY DENSITY
Beneath an area extending at least 4 feet beyond the perimeter of structures, and below flatwork and pavement (applies to structural fill and site grading fill) extending at least 2 feet beyond the perimeter	0 to 5 More than 5	95 98
Site grading fill outside area defined above	0 to 5 More than 5	92 95
Utility trenches within structural areas	--	96
Roadbase and subbase	-	96
Non-structural fill	0 to 5 more than 5	90 92

Structural fills below buildings greater than 10 feet thick are not anticipated at the site. For best compaction results, we recommend that the moisture content for structural fill/backfill be within 2% of optimum. Field density tests should be performed on each lift as necessary to verify that proper compaction is being achieved.

Along existing slopes, benching of fill material into the slopes will be required extending a minimum of 2 feet into the hillside with maximum bench heights of 3 feet.

Embankment fills for roadways greater than 8 feet thick must be moisture conditioned to optimum or above and compacted to 98% with respect to the above criteria. Also, settlement monitoring will likely be required and should be evaluated on a case by case basis.

6.7 Utility Trenches

For the bedding zone around the utility, we recommend utilizing sand bedding fill material that meets current APWA[3] requirements.

Most utility companies and local governments are requiring Type A-1a or A-1b (AASHTO Designation) soils (sand/gravel soils with limited fines) be used as backfill over utilities within public rights of way, and the backfill be compacted over the full depth above the bedding zone to at least 96% of the maximum dry density as determined by AASHTO T-180 (ASTM D-1557). Existing sand and gravel soils at this site may meet these specifications with some processing.

Where the utility does not underlie structurally loaded facilities and public rights of way, on-site fill and natural soils may be utilized as trench backfill above the bedding layer, provided they are properly moisture conditioned and compacted to the minimum requirements stated above in **Section 6.6**.

[3] American Public Works Association

6.8 Stabilization

To stabilize soft subgrade conditions (if encountered), a mixture of coarse, clean, angular gravels and cobbles and/or 1.5- to 2.0-inch clean gravel should be utilized, as indicated above in **Section 6.5**. Often the amount of gravelly material can be reduced with the use of a geotextile fabric such as Mirafi RS280i or equivalent. Its use will also help avoid mixing of the subgrade soils with the gravelly material. After excavating the soft/disturbed soils, the fabric should be spread across the bottom of the excavation and up the sides a minimum of 18 inches. Otherwise, it should be placed in accordance with the manufacturer's recommendation, including proper overlaps. The gravel material can then be placed over the fabric in compacted lifts as described above.

7.0 SLOPE STATILITY

7.1 Input Parameters

The properties of the natural soils and bedrock encountered at the test pit and bore hole locations were estimated using laboratory testing, moderate comparison review to published correlations[4] (particularly for properties of fill soils), and our experience with similar soils. Accordingly, we estimated the following parameters for use in the stability analyses:

Material	Internal Friction Angle (degrees)	Apparent Cohesion (psf)	Unit Weight (pcf)*
Lacustrine Transgressive Interbedded Fines (Clays/Silts)	28.7	204	120
Mass Wasting Colluvium	33	0	120
Post Lake Alluvium	38	50	115
Lacustrine Transgressive Sand & Gravel, Regressive Sand & Gravel	39	86	120
Bedrock	39	1000	140
Onsite fine soil Fills	28	50	120
On-Site Fills Following Mass Grading/Post Grading Structural Fill	*33[4]	120[4]	120
Clayey Silt (B-4A at 80 Feet)	28.8	340	118
Very Moist Clay Layer (B-4A at 40 feet)	25	99	118

* Utilizing referenced (the Bureau of Reclamation) Values for mixed sand-slit-clay soil fills compacted as structural site grading fill.

[4] U.S. Bureau of Reclamation, 1987, "Design Standards No. 13, Embankment Dams," Denver, Colorado.

The stability analyses provided are based on nine cross sections (A-A' through I-I'. These cross sections are shown on **Figures 4A through 4I.** Each cross section is a representative slope sections of the site, showing the proposed slope conditions incorporating recommendations provided here in.

Guidelines set forth by Fruit Heights City for the evaluation of slope stability under seismic (pseudostatic) conditions indicate that peak horizontal accelerations should be based on a 2% probability of exceedance in a 50 year return period. The peak ground acceleration after adjusting for Site Class D was obtained for site (grid) locations of 41.035917 degrees latitude (north) and 111.901273 degrees longitude (west) was 0.644g. A deformation-based analysis was utilized to obtain the seismic (pseudostatic) horizontal acceleration utilizing the Bray and Travasarou[5] methodology for a maximum 10cm (4-inch) displacement at 0.21g. Accordingly, a value of 0.21g was used as the acceptable pseudostatic coefficient for the stability analysis.

In addition, and for reference, pseudostatic horizontal accelerations were obtained for 7.6cm (3-inch); 5cm (2-inch) and no displacement with calculated accelerations of .242g, .291g, and .322g respectively. The affected distance from the crest with respect to displacement utilizing the calculated accelerations are shown by dashed lines above the west slope crest on Plate 6B Post Grading Landslide Hazard.

7.2 Associated Testing Representing Input Parameters

Highlighted vales were primarily utilized in our analysis.

7.2.1 Lacustrine Fines (Clay/Silt/Sand)

Peak values were considered as these soils do not represent landslide materials.
Representative sources and lab testing.

Sample Location	Sample Depth (feet)	Unified Soils Classification	Results Method	Apparent Cohesion (psf)	Measured Internal Friction Angle (degrees)
TP-15	7	CL	Peak	204	28.7
B-4	80	ML-CL	peak	340	28.8
B-1	10	SP-SM	Peak	144	42
B-2	20	SP	Peak	86	39
Raymond B-2	10.5	SM	Peak	20	36
Raymond TP-4	3.5	SM	Peak	325	32

[5] Bray, J. D., & Travasarou, T., "Pseudostatic Coefficient for use in Simplified Seismic Slope Stability Evaluation," Journal of Geotechnical Engineering, ASCE, September 2009, p 1336-1340.

Raymond TP-8	5	SM	Peak	20	36

7.2.2 Mass Wasting Colluvium

Representative sources and lab testing.

Sample Location	Sample Depth (feet)	Unified Soils Classification	Results Method	Apparent Cohesion (psf)	Measured Internal Friction Angle (degrees)
Trench 7	4	ML-SM	Residual	*120.5	33.1
Trench 8	8	SC	Residual	541.6	32.2
Trench 8	6.5	GC-GM	Residual	*50	33

* cohesion to be omitted.

7.2.3 Lacustrine Transgressive Sand & Gravel, Regressive Sand & Gravel, and Post-Lake Alluvium

Representative sources and lab testing.

Sample Location	Sample Depth (feet)	Unified Soils Classification	Results Method	Apparent Cohesion (psf)	Measured Internal Friction Angle (degrees)
B-1	10	SP-SM	Peak	144	42
B-2	20	SP	Peak	86	39
TP-16	7	GP-SP	Peak	50	38
Raymond TP-4	3.5	SM	Peak	325	32
Raymond TP-8	5	SM	Peak	20	36
Raymond B-2	10.5	SM	Peak	20	36

7.2.4 Bedrock and Fills

These soil types were estimated based on published correlations as well as past projects with similar bedrock.

7.3 Stability Analyses

The global stability for the cross sections as shown on **Plate 6B, Post-Grading Landslide Hazard** have been complete. In addition, a generic cross section with a 100-foot-tall slope was also analyzed. Each analysis was completed using the computer program *SLIDE* version 7.0. This program uses a limit equilibrium (Simplified Bishop) method for calculating factors of safety against sliding on an assumed failure surface and evaluates numerous potential failure surfaces, with the most critical failure surface identified as the one yielding the lowest factor of safety of those evaluated.

A water (phreatic) surface was conservatively added to the cross sections A-A' through I-I' at a depth of about 8 feet below the surface which daylighted out to the slope in order to represent the perched conditions seen in trench 8. Shallow groundwater was not observed within any of the other test pits, trenches or bore holes. Static groundwater levels were utilized in our analysis based on observed conditions encountered during the course of our field investigations in conjunction with the interpretation by the signing professional geologist. The current proposed grading along these cross sections was utilized in the analysis. In **Section 6.3** above, permanent unbraced cut/fill slopes were discussed. Following are discussions with respect to the nine cross-sections we analyzed, and our evaluation results:

- Cross-section A-A' consists of an approximate 1060-foot-long horizontal cross-section with an overall elevation change of about 200-feet and with sloped portions as steep as about one and one-half horizontal to one vertical (1.5H:1V) (See **Figure 4A, Cross Section *A-A'*** in the Appendix). This cross section begins offsite to the west and comes up through Lot 60 (see **Plate 6B, Post Grading Landslide Hazard** for cross section locations). Based on the slope stability analyses (see **Figures 35-1 through 35-5 Stability Results** in the appendix), failure surfaces with a safety factor less than 1.0 for seismic/dynamic conditions extend between about 434 feet and 600 feet east of the cross section end from the west and particularly within the proposed one and one-half horizontal to one vertical (1.5H:1V) cut location across lots 46 through 49 for the new roadway(**see Plate 6B Post Grading Landslide Hazard** in the appendix). Recommended, unbraced cut slopes are discussed in section **6.3 Permanent Cut/Fill Slopes** above and must be followed or each individual lot affected must be evaluated separately.

- Cross-section B-B' consists of an approximate 500-foot-long horizontal cross-section with an overall elevation change of about 110 feet and with sloped portions as steep as about three horizontal to one vertical (3H:1V) above the fault scarp (See **Figure 4B, Cross Section B-*B'*** in the Appendix). This cross-section begins offsite to the west and comes up through Lot 41 (see **Plate 1, Site Evaluation** for cross section location). Based on the slope stability analyses (see **Figures 36-1 through 36-4 Stability Results** in the appendix), the current slope generally meets the minimum safety factor of 1.5 for static conditions and 1.0 for seismic conditions within the buildable areas. Failure surfaces with less than the required 1.5 and 1.0 for static and seismic conditions were located across the fault scarp outside of the buildable area. The buildable limit for lots represented by this cross-section will be dictated by the steepest portion of the slope which crosses the fault and is located within the "greater than 30% - no build zone" (**see Plate 6B Post Grading Landslide Hazard** in the appendix).

- Cross-section C-C' consists of an approximate 560-foot-long horizontal cross-section with an overall elevation change of about 245-feet and with sloped portions as steep as about one and one-half horizontal to one vertical (1.5H:1V) (See **Figure 4C, Cross Section C-*C'*** in the Appendix). This cross-section is almost entirely offsite and along the hillside east of the proposed development (see **Plate 6B, Post-Grading Landslide Hazard** for cross section location). Based on the slope stability analyses, the current slope is essentially stable in static conditions. When applying the seismic design acceleration, the upper, relatively shallow sand and gravel layer has a safety factor of slightly less than 1.0 along the majority of the slope section analyzed (See **Figures 37-1 through 37-4, Stability Results** in the appendix). A deformation analysis was completed to estimate the amount of movement that could occur using the Newmark Sliding Block method. The amount of movement from this analysis was about 8 inches.

- Cross-section D-D' consists of an approximate 736-foot-long horizontal cross-section with an overall elevation change of about 150 feet and with sloped portions as steep as (2.7 Horizontal to 1 Vertical) (See **Figure 4D, Cross Section *D-D'*** in the Appendix). This cross-section begins offsite to the west and comes up through Lot 39 (see **Plate 6B, Post Grading Landslide Hazard** for cross-section locations). Based on the slope stability analyses (see **Figures 38-1 through 38-4 Stability Results** in the appendix), the current slope generally meets the minimum safety factor of 1.5 for static conditions and 1.0 for seismic conditions within the buildable area. Failure surfaces with less than the required 1.5 and 1.0 for static and seismic conditions were located across the fault scarp outside of the buildable area. Further, more extensive seismic related failure surfaces with a safety factor less than 1.0 for seismic/dynamic conditions extend up through the full cross section with increased accelerations related to less deformation (see **Figures 38-1 through 38-4 Stability Results** in the appendix)

- Cross-section E-E' consists of an approximate 636-foot-long horizontal cross-section with an overall elevation change of about 128 feet and with sloped portions as steep as about two horizontal to one vertical (2H:1V) (See **Figure 4E, Cross Section *E-E'*** in the Appendix). This cross-section begins offsite to the west and comes up through Lot 34 (see **Plate 6B, Post Grading Landslide Hazard** for cross-section locations) and assumes that no site grading fills will be placed on current lots 33 and 34. Based on the slope stability analyses (see **Figures 39-1 through 39-3 Stability Results** in the appendix), the home on lot 34 must be placed further uphill beyond the crest as shown on Plate 6B, Post Grading Landslide Hazard.

- Cross-section F-F' consists of an approximate 567-foot-long horizontal cross-section with an overall elevation change of about 110 feet and with sloped portions as steep as about one horizontal to one vertical (1.7H:1V) (See **Figure 4F, Cross Section *F-F'*** in the Appendix). This cross-section begins offsite to the west and comes up through Lot 27 (see **Plate 6B, Post Grading Landslide Hazard** for cross-section locations) and assumes that no site grading fills will be placed on current lots 27, 28, and 29. Based on the slope stability analysis (see **Figures 40-1 through 40-4 Stability Results** in the appendix), failure surfaces for seismic condition having up to 10cm of displacement there were failure surfaces with less than a 1.0 safety factor extend about 47 feet east of the scarp crest, (**see Plate 6B Post Grading Landslide Hazard** in the appendix). Failure surfaces continue to move upslope and across the planned development with increased accelerations related to less deformation (see **Figures 40-1 through 40-4 Stability Results** in the appendix).

- Cross Section G-G consists of an approximate 485-foot-long horizontal cross-section with an overall elevation change of about 116 feet and with sloped portions as steep as about 1.8 Horizontal to 1 Vertical) (See **Figure 4F, Cross Section *F-F'*** in the Appendix). This cross-section begins offsite to the west and comes up through Lot 26 (see **Plate 6B, Post Grading Landslide Hazard** for cross-section locations) and assumes that no site grading fills will be placed on current lots 25 and 26. Based on the slope stability analysis (see **Figures 41-1 through 41-3 Stability Results** in the appendix), failure surfaces for both the static condition and the seismic condition having up to 10cm of displacement with a safety factor less than 1.5 and 1.0 extend about 65 and 124 feet east of the scarp crest respectively, (**see Plate 6B Post Grading Landslide Hazard** in the appendix). Failure surfaces continue to move upslope and across the planned development with increased accelerations related to less deformation (see **Figures 41-1 through 41-3 Stability Results** in the appendix).

- Cross-section H-H' consists of an approximate 1428-foot-long horizontal cross-section with an overall elevation change of about 261-feet and with sloped portions as steep as about (2.7 Horizontal to 1 Veritical) (See **Figure 4H, Cross Section H-*H'*** in the Appendix). This cross section begins offsite to the west and comes up through Lot 23 (see **Plate 6B, Post Grading Landslide Hazard** for cross section locations) and spans the entire width of the property. Based on the slope stability analyses (see **Figures 42-1 through 42-4 Stability Results** in the appendix), the current slope generally meets the minimum safety factor of 1.5 for static conditions and 1.0 for seismic conditions having a displacement of 3 and 4 inches. However, more extensive seismic related failure surfaces with a safety factor less than 1.0 for seismic/dynamic conditions extend up through about 980 feet from the west end with increased accelerations related to less deformation (see **Figures 42-1 through 42-4 Stability Results** in the appendix)

- Cross-section I-I' consists of an approximate 325-foot-long horizontal cross-section with an overall elevation change of about 60-feet and with sloped portions as steep as about (6.3 Horizontal to 1 Veritical) (See **Figure 4I, Cross Section I-I'** in the Appendix). This cross section begins at the proposed roadway (Street C) to the west and comes up through Lot 86 (see **Plate 6B, Post Grading Landslide Hazard** for cross section locations). Based on the slope stability analyses (see **Figures 43-1 through 43-4 Stability Results** in the appendix), the current slope generally meets the minimum safety factor of 1.5 for static conditions and 1.0 for seismic conditions having a displacement of 0 inches up to 4 inches.

- A generic slope with a similar soil cross-section to that considered for cross-section C-C' and up to 100 feet tall was analyzed to obtain a working slope cut for un-braced grading conditions. When laid back to a slope of about two- and one-half horizontal to one vertical (2.5H: 1V), this cross-section meets the minimum safety factors for both static and seismic conditions (see **Figures 44-1 and 44-2, Stability Results** in the Appendix).

The following values provided in the table represent the static condition and the dynamic/seismic condition with up to 10cm of displacement based on the acceleration reduction utilizing the Bray and Travasarou[6] methodology referenced above. Typically, the required minimum factors of safety are 1.5 for static conditions and 1.0 for seismic (pseudostatic) conditions. The lowest stability results of our stability analyses along the nine cross sections representing proposed slope conditions as well as the generic 100-foot-tall slope, utilizing the estimated soil properties described previously, are tabulated below. For proposed buildable area as it relates to calculated stability results, please refer to **Plate 6B Post Grading Landslide Hazard.** The below, tabulated minimum factors of safety **do not** necessarily represent the entire cross section but only portions. For further discussion see itemized paragraphs above as well as the graphical representations on Figures 35 through 44 in the appendix.

Slope Cross Section	Condition	Seismic Coefficient	Lowest Factor of Safety (F.S.)	Minimum Allowable F.S.
A-A'	Static	---	1.588	1.5
A-A'	Seismic	.211	1.013	1.0
B-B'	Static	---	1.098	1.5
B-B'	Seismic	.211	0.711	1.0
C-C'	Static	---	1.493	1.5
C-C'	Seismic	.211	0.998	1.0
D-D'	Static	---	1.123	1.5
D-D'	Seismic	.211	0.724	1.0
E-E'	Static	---	1.033	1.5
E-E'	Seismic	.211	0.672	1.0
F-F'	Static	---	1.163	1.5
F-F'	Seismic	.211	0.746	1.0
G-G'	Static	---	1.189	1.5
G-G'	Seismic	.211	0.760	1.0
H-H'	Static	---	1.635	1.5
H-H'	Seismic	.211	1.074	1.0
I-I'	Static	---	2.34	1.5
I-I'	Seismic	.211	1.534	1.0
Generic 2.5H:1V, 100' tall Slope	Static	---	1.985	1.5
Generic 2.5H:1V, 100' tall Slope	Seismic	.211	1.101	1.0

Graphical representation of the above table results are provided on **Figures 35 through 44** in the Appendix.

[6] Bray, J. D., & Travasarou, T., "Pseudostatic Coefficient for use in Simplified Seismic Slope Stability Evaluation," Journal of Geotechnical Engineering, ASCE, September 2009, p 1336-1340.

Slope movements or even failure can occur if the slope soils are undermined or become saturated. Any retaining walls must be properly engineered to maintain stability of the slopes. Any changes to the grading at the site must be reviewed by CMT prior to initiation of any construction in order to assess if our findings and recommendations remain applicable. During construction, CMT must observe grading to ensure suitable soil conditions are encountered. Following grading at the site, the slope surface must be re-vegetated as soon as possible to limit erosion and potential undermining of the slope. The property owner and the owner's representatives should be made aware of the risks involved should these or other conditions occur that could saturate or erode/undermine the slope soils.

Further, we recommend that a cutoff drain be installed along the crest of the lower slope as discussed in Section 11.0, **Drainage**, of this report.

7.4 Site Irrigation

Proper site drainage is important to maintaining slope stability at the site. Saturation of soils at the site may result in slope movement or failure. Therefore, we recommend that sprinklers and sprinkler/irrigation lines not be placed on the steep fault scarp slope along the west side of the property nor within 30 feet from the crest of the slope. Landscaping at the site should be planned to utilize drought resistant plants that require minimal watering. Plants or lawn may be placed up to the crest of the slope, with plants watered using direct drip systems targeted only for each plant, and any lawn areas watered using sprinklers placed a minimum of 30 feet from the crest of the slope. Overwatering should be strictly avoided. The surface of the site should be graded to prevent the accumulation or ponding of surface water at the site. The property owner and the owner's representatives should be made aware of the risks should these or other conditions occur that could saturate or erode/undermine the slope soils. It is our understanding that the development will be part of an association that will help monitor site irrigation.

To reduce the potential for saturation of the site soils, overwatering at the site should be strictly avoided. Watering at the site should be limited to a maximum equivalent rainfall of 0.5 inches per week. Irrigation at the site should be strictly avoided during periods of natural precipitation.

We understand that the recommendations above for site irrigation will be further defined by the developer through the development agreement with Fruit Heights City, including the enforcement mechanisms for the site irrigation recommendations.

8.0 FOUNDATION RECOMMENDATIONS

The following recommendations have been developed on the basis of the previously described Project characteristics and loading in section **1.3 Description of Proposed Construction**, the subsurface conditions observed in the field and the laboratory test data, slope stability analysis, and with respect to common geotechnical engineering practice.

The results of our analyses indicate that the proposed structures may be supported upon conventional spread and/or continuous wall foundations established upon suitable natural soils or granular structural fill extending

to suitable natural soils. Homes built on the lots along the western slope of the property must be offset from the slope crest as discussed in section **7.3 Stability Analysis** above.

8.1 Continuous Spread Foundation Recommendations

For design, with respect to projected construction and anticipated loading, the following parameters are recommended:

ITEM	RECOMMENDATION
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, foundations established on suitable natural soils or structural fill extending to suitable natural soils	2,000 psf
Bearing pressure increase for seismic loading	30%

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.

8.1.1 Installation

Under no circumstances shall foundations be placed directly on undocumented fill, topsoil, rubbish, construction debris, other deleterious materials, frozen soils, or within ponded water. If bedrock is encountered at the footing elevation, then building grades must be adjusted or the excavation extended to facilitate a minimum of 12 inches of structural fill beneath all footings.

Deep, large roots may be encountered where trees and larger bushes are located or were previously located at the site; such large roots should be removed. If unsuitable soils are encountered, they must be completely removed and replaced with properly compacted structural fill. Excavation bottoms should be examined by a qualified geotechnical engineer to confirm that suitable bearing materials soils have been exposed.

Landslide deposits are mapped within the buildable portions of Lots 26, 27, 28, 41 and 42. Based on our field investigation, the landslide deposits are relatively shallow in depth. Landslide deposits must be removed below proposed structures and extending downslope a minimum of 20 feet beyond the homes. This may require up to 10 feet of soil be removal and replacement with controlled structural fill.

All structural fill shall meet the requirements for such, and should be placed and compacted in accordance with **Section 6** above. The width of structural replacement fill below footings should be equal to the width of the footing plus 1 foot for each foot of fill thickness. For instance, if the footing width is 2 feet and the structural fill depth beneath the footing is 2 feet, the fill replacement width should be 4 feet, centered beneath the footing.

8.1.2 Estimated Settlement

Foundations designed and constructed in accordance with our recommendations could experience some settlement, but we anticipate that total settlements of footings founded as recommended above will not exceed 1 inch, with differential settlements on the order of 0.5 inches over a distance of 25 feet. We expect approximately 50% of the total settlement to initially take place during construction.

8.1.3 Lateral Resistance

Lateral loads imposed upon shallow 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, a coefficient of 0.30 for natural silt/clay soils or 0.40 for natural sand/gravel soils and structural fill, may be utilized for design. Passive resistance provided by properly placed and compacted structural fill above the water table may be considered equivalent to a fluid with a density of 300 pcf. A combination of passive earth resistance and friction may be utilized if the friction component of the total is divided by 1.5.

9.0 LATERAL EARTH PRESSURES

The lateral pressure parameters, as presented herein, are for backfills which will consist of suitable, drained on-site granular soil.

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), backfill may be considered equivalent to a fluid with a density of 40 pounds per cubic foot in computing lateral pressures. For more rigid walls (moderately yielding), backfill may be considered equivalent to a fluid with a density of 50 pounds per cubic foot. For very rigid non-yielding walls, 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 fill within 4 feet of the wall will be compacted with hand-operated compacting equipment.

For seismic loading of retaining/below-grade walls, the following uniform lateral pressures, in pounds per square foot (psf), should be added based on wall depth and wall case.

UNIFORM LATERAL PRESSURES			
WALL HEIGHT (FEET)	ACTIVE PRESSURE CASE (PSF)	MODERATELY YIELDING CASE (PSF)	AT REST/NON-YIELDING CASE (PSF)
4	27	56	85
6	40	84	128
8	54	112	171

The given values for design are based on the natural granular soils in place behind walls. The values above may be linearly interpolated between heights given. If the natural soils will be excavated and other soils placed as backfill, we recommend that this office review the materials and determine if the above design earth pressures are still appropriate.

10.0 FLOOR SLABS

Floor slabs may be established upon suitable, undisturbed, natural soils and/or on structural fill extending to suitable natural soils (same as for foundations). Under no circumstances shall floor slabs be established directly on topsoil, non-engineered fills, loose or disturbed soils, sod, rubbish, construction debris, other deleterious materials, frozen soils, or within ponded water.

In order to facilitate curing of the concrete, we recommend that floor slabs be directly underlain by at least 4 inches of "free-draining" fill, such as "pea" gravel or 3/4-inch quarters to 1-inch minus, clean, gap-graded gravel. To help control normal shrinkage and stress cracking, the floor slabs may include the following features:

1. Adequate reinforcement for the anticipated floor loads with the reinforcement continuous through interior floor joints;
2. Frequent crack control joints; and
3. Non-rigid attachment of the slabs to foundation walls and bearing slabs.

11.0 DRAINAGE RECOMMENDATIONS

11.1 Surface Drainage

It is important to the long-term performance of foundations and floor slabs that water not be allowed to collect near the foundation walls and infiltrate into the underlying soils. Additionally, some of the on-site near-surface silty/clayey soils may potentially be collapsible or expansive when subjected to water, thus it is very important to the long-term performance of foundations and floor slabs that water not be allowed to collect near the foundation walls and infiltrate into the underlying soils. We recommend the following:

1. All areas around the structure should be sloped to provide drainage away from the foundations. We recommend a minimum slope of 6 inches in the first 10 feet away from the structure. This slope should be maintained throughout the lifetime of the structure.

2. All roof drainage should be collected in rain gutters with downspouts designed to discharge at least 10 feet from the foundation walls or well beyond the backfill limits, whichever is greater. Further drainage shall be controlled such that it does not affect adjacent properties.

3. Adequate compaction of the foundation backfill should be provided. We suggest a minimum of 90% of the maximum laboratory density as determined by ASTM D-1557. Water consolidation methods should not be used under any circumstances.

4. Landscape sprinklers should be aimed away, and maintained a distance of at least 4 feet, from the foundation walls. The sprinkling systems should be designed with proper drainage and be well-maintained. Over watering should be avoided.

5. As discussed in section **7.3 Site irrigation**, Saturation of soils at the site may result in slope movement or failure. Therefore, sprinklers and sprinkler/irrigation lines must not be placed on the steep fault scarp slope along the west side of the property nor within 30 feet from the crest of the slope. Landscaping at the site should be planned to utilize drought resistant plants that require minimal watering. Overwatering at the site should be strictly avoided and limited to a maximum equivalent rainfall of 0.5 inches per week. Irrigation at the site should be strictly avoided during periods of natural precipitation.

6. Other precautions that may become evident during construction.

11.2 Foundation Subdrains

Due to the potential for localized “perched” groundwater conditions, especially during the spring and early summer months against foundations, we recommend the installation of foundation subdrains around structures with subgrade levels extending more than 4 feet below the adjacent ground surface. The subdrain must gravity daylight well beyond and down-gradient of the home and retaining walls, or deliver any captured water to another suitable down-gradient control source.

Foundation subdrains should consist of a 4-inch diameter perforated or slotted solid-plastic or PVC pipe enclosed in clean gravel comprised of three-quarter- to one-inch minus gap graded gravel and/or “pea” gravel. The invert of a subdrain should be at least 18 inches below the top of the lowest adjacent habitable floor slab. The gravel portion of the drain should extend 2 inches laterally and below the perforated pipe and at least 1 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 foundation subdrain, a minimum 4-inch-wide zone of “free-draining” sand or gravel (chimney) should be placed adjacent to the foundation walls and extend to within 1.5 feet of final grade. The sand/gravel fill must be separated from adjacent native or backfill soils with geotextile fabric (Mirafi 140N or equivalent). The upper

1.5 feet of soils should consist of a compacted clayey soil cap to reduce surface water infiltration into the drain. As an alternative to the zone of permeable sand and gravel, a prefabricated "drainage board," such as Miradrain or equivalent, may be placed against the exterior below-grade walls. Prior to the installation of the footing subdrain, the below-grade walls should be adequately dampproofed. The slope of the subdrain should be at least 0.3 percent. The foundation subdrains shall be discharged to a suitable down gradient location (i.e. area subdrains, storm drains, etc.).

12.0 PAVEMENTS

We anticipate the surficial prepared existing undocumented fill soils and natural fine-grained clay/silt soils will exhibit poor pavement support characteristics when saturated or nearly saturated. Based on our laboratory testing experience with similar soils, our pavement design utilized a California Bearing Ratio (CBR) of 5 for the prepared existing undocumented fill soils.

Following proper preparation of surficial soils as described in **Section 6.1** above, the exposed subgrade must proofrolled by passing moderate-weight rubber tire-mounted construction equipment over the surface at least twice. If excessively soft or loose soils are encountered, they must be removed (up to a maximum depth of 2 feet) and replaced with structural fill.

Given the Projected traffic as discussed above in **Section 1.3**, the following pavement sections are recommended for approximately 6 ESAL's (18-kip equivalent single-axle loads) per day:

MATERIAL	PAVEMENT SECTION THICKNESS (inches)	
Asphalt	**3**	**3**
Road-Base	**10**	**6**
Subbase	**0**	**6**
Total Thickness	**13**	**15**

Untreated base course (UTBC) should conform to city specifications, or to 1-inch-minus UDOT specifications for A–1-a/NP, and have a minimum CBR value of 70%. Material meeting our specification for structural fill can be used for subbase, as long as the fines content (percent passing No. 200 sieve) does not exceed 15%. Roadbase and subbase material should be compacted as recommended above in **Section 6.4**. Asphalt material generally should conform to APWA requirements, having a ½-inch maximum aggregate size, a 75-gyration Superpave mix containing no more than 15% of recycled asphalt (RAP) and a PG58-28 binder.

Exterior concrete should be designed in accordance with the American Concrete Institute (ACI) and joint details should conform to the Portland Cement Association (PCA) guidelines. The concrete should have a minimum 28-day unconfined compressive strength of 4,000 pounds per square inch and contain 6% ±1% air-entrainment.

13.0 QUALITY CONTROL

We recommend that CMT be retained to as part of a comprehensive quality control testing and observation program. With CMT on-site we can help facilitate implementation of our recommendations and address, in a timely manner, any subsurface conditions encountered which vary from those described in this report. Without such a program CMT cannot be responsible for application of our recommendations to subsurface conditions which may vary from those described herein. This program may include, but not necessarily be limited to, the following sections.

13.1 Field Observations

Observations should be completed during all phases of construction such as site preparation, foundation excavation, structural fill placement and concrete placement.

13.2 Fill Compaction

Compaction testing by CMT is required for all structural supporting fill materials. Maximum Dry Density (Modified Proctor, ASTM D-1557) tests should be requested by the contractor immediately after delivery of any fill materials. The maximum density information should then be used for field density tests on each lift as necessary to ensure that the required compaction is being achieved.

13.3 Excavations

All excavation procedures and processes should be observed by a geotechnical engineer from CMT or their representative. In addition, for the recommendations in this report to be valid, all backfill and structural fill placed in trenches and all pavements should be density tested by CMT. We recommend that freshly mixed concrete be tested by CMT in accordance with ASTM designations.

14.0 LIMITATIONS

The recommendations provided herein were developed by evaluating the information obtained from the subsurface explorations and soils encountered therein. The exploration logs reflect the subsurface conditions only at the specific location at the particular time designated on the logs. Soil and ground water conditions may differ from conditions encountered at the actual exploration locations. The nature and extent of any variation in the explorations may not become evident until during the course of construction. If variations do appear, it may become necessary to re-evaluate the recommendations of this report after we have observed the variation.

Our professional services have been performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all other warranties, either expressed or implied.

We appreciate the opportunity to be of service to you on this Project. If we can be of further assistance or if you have any questions regarding this Project, please do not hesitate to contact us at (801) 870-6730. To schedule materials testing, please call (801) 381-5141.

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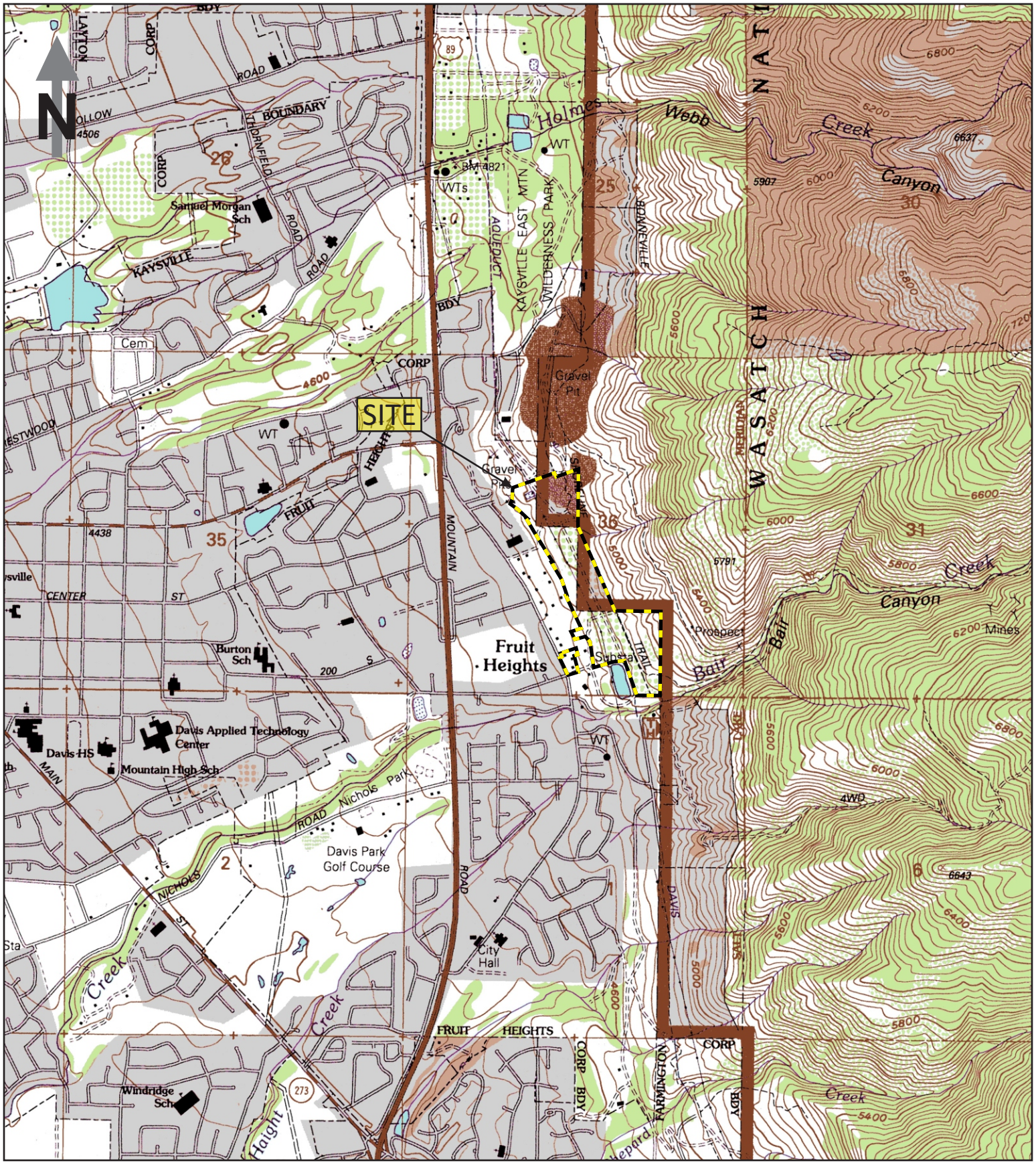
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APPENDIX A

SUPPORTING
DOCUMENTATION



Base:
USGS 7.5-minute topographic quadrangles, Utah; KAYSVILLE and PETERSON, 1998.

0 2,000 ft 4,000 ft

1:24,000 (1 inch equals 2,000 feet)

Proposed Rock Loft Ridge Subdivision

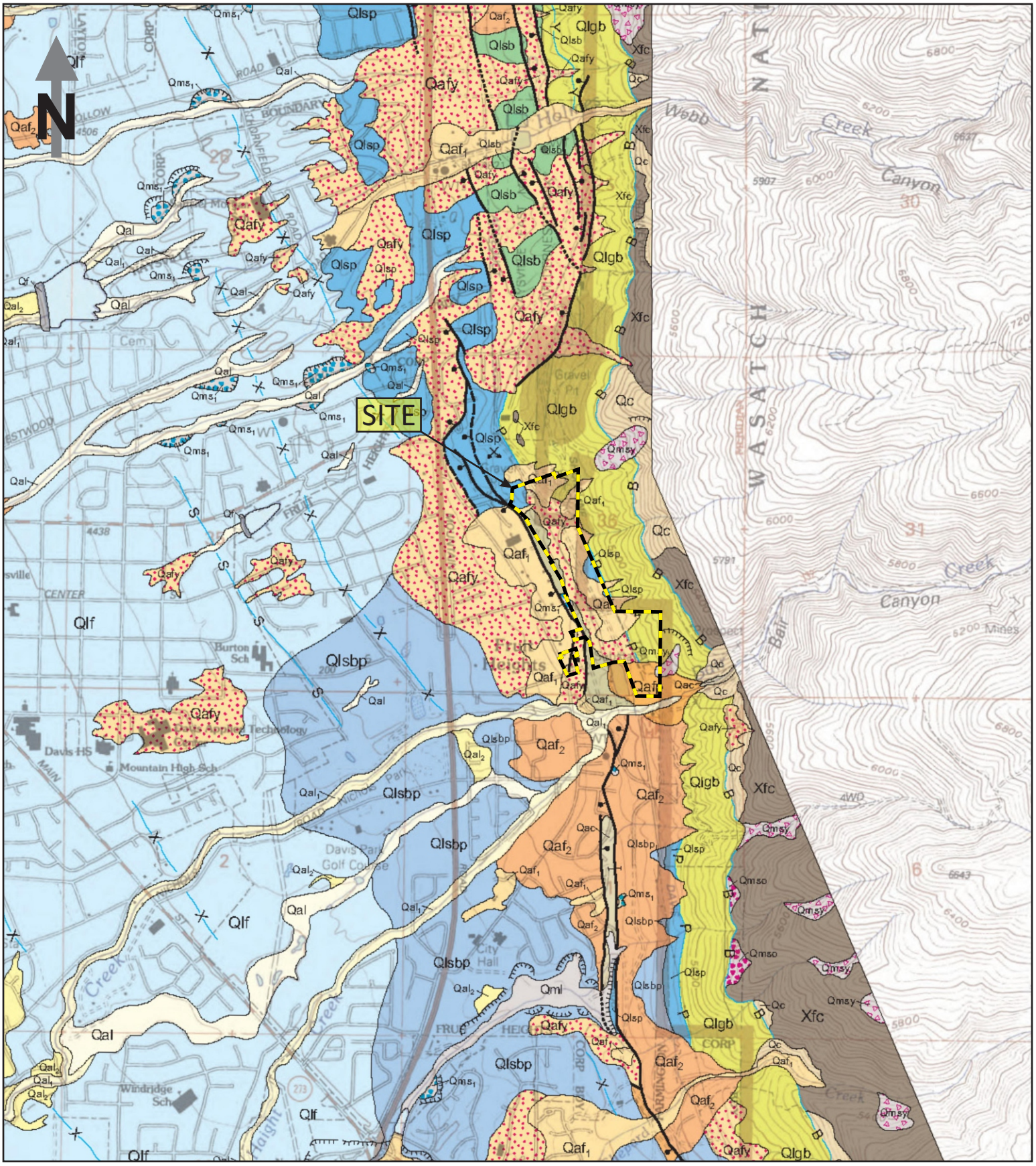
About 250 North 1450 East, Fruit Heights, Utah

CMT ENGINEERING LABORATORIES

Vicinity Map

Date:	08-Oct-2020
Job #	12188

Figure 1



Base:
Surficial Geologic Map of part of the Kaysville Quadrangle (Solomon, 2007).

1:24,000 (1 inch equals 2,000 feet)

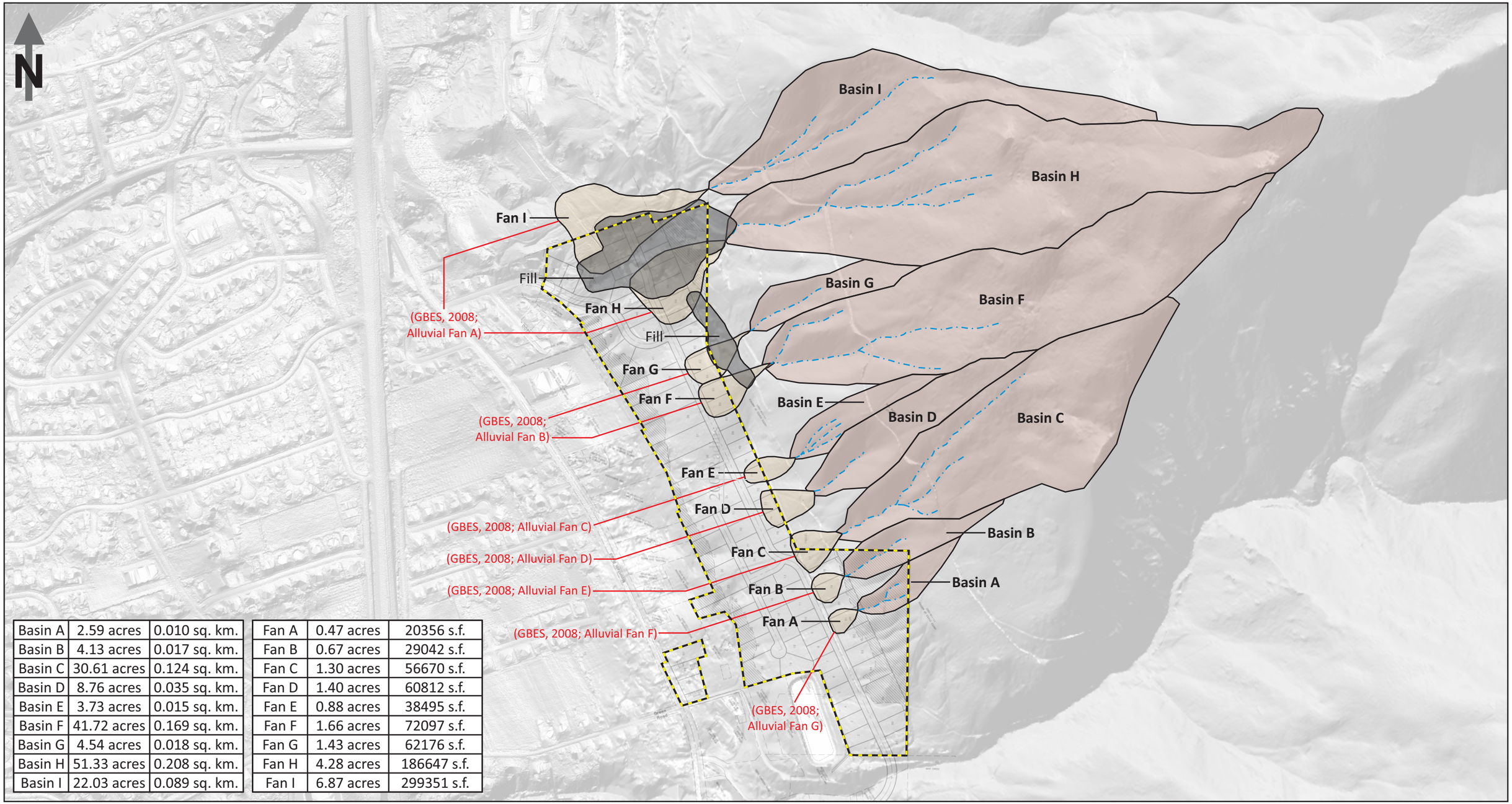
Proposed Rock Loft Ridge Subdivision
About 250 North 1450 East, Fruit Heights, Utah

CMT ENGINEERING LABORATORIES

Geologic Map

Date:	08-Oct-2020
Job #	12188

Figure
2



Basin A	2.59 acres	0.010 sq. km.	Fan A	0.47 acres	20356 s.f.
Basin B	4.13 acres	0.017 sq. km.	Fan B	0.67 acres	29042 s.f.
Basin C	30.61 acres	0.124 sq. km.	Fan C	1.30 acres	56670 s.f.
Basin D	8.76 acres	0.035 sq. km.	Fan D	1.40 acres	60812 s.f.
Basin E	3.73 acres	0.015 sq. km.	Fan E	0.88 acres	38495 s.f.
Basin F	41.72 acres	0.169 sq. km.	Fan F	1.66 acres	72097 s.f.
Basin G	4.54 acres	0.018 sq. km.	Fan G	1.43 acres	62176 s.f.
Basin H	51.33 acres	0.208 sq. km.	Fan H	4.28 acres	186647 s.f.
Basin I	22.03 acres	0.089 sq. km.	Fan I	6.87 acres	299351 s.f.

Base:
2013 50cm and 2006 2m bare earth LIDAR DEM available from Utah AGRC; site plan from Reeve & Associates preliminary grading plan cover sheet, July 2020.

0 600 ft 1,200 ft

1:7,200 (1 inch equals 600 feet)

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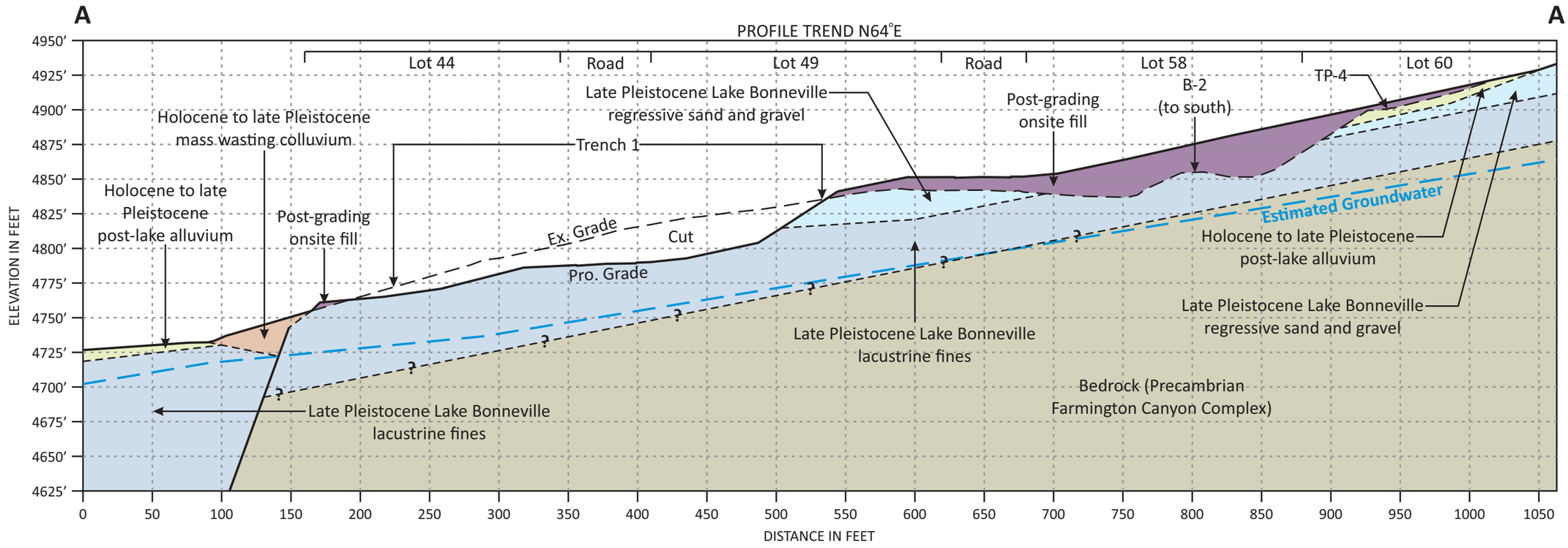
Proposed Rock Loft Ridge Subdivision

About 250 North 1450 East, Fruit Heights, Utah

Drainages and Alluvium

Date:	24-Oct-2020
CMT No.:	12188

Figure 3



Scale 1 inch equals 80 feet (1:960) with no vertical exaggeration. Existing and proposed grades are from Reeve & Associates. Contact and units are inferred and approximate.

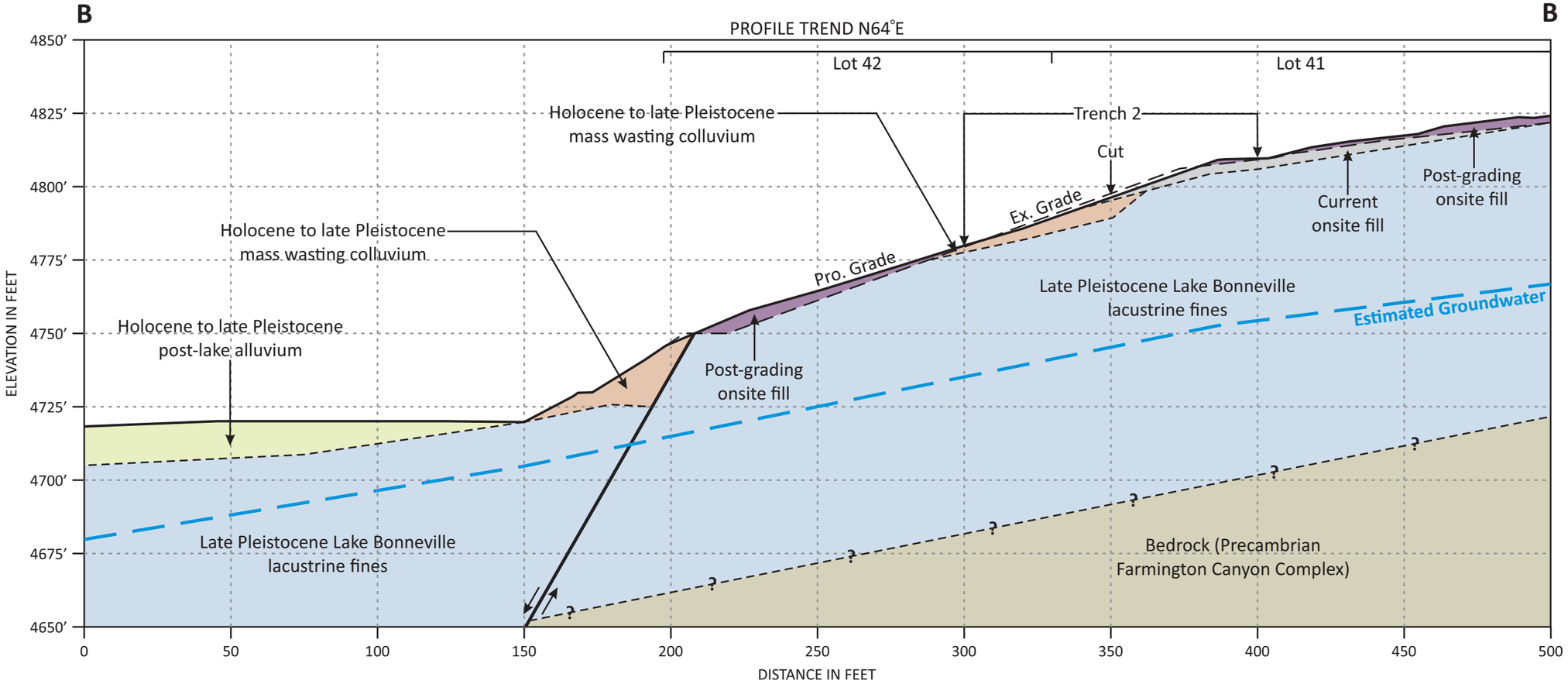
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Rock Loft Ridge Subdivision
About 250 North 1450 East, Fruit Heights, Utah

Cross Section A-A

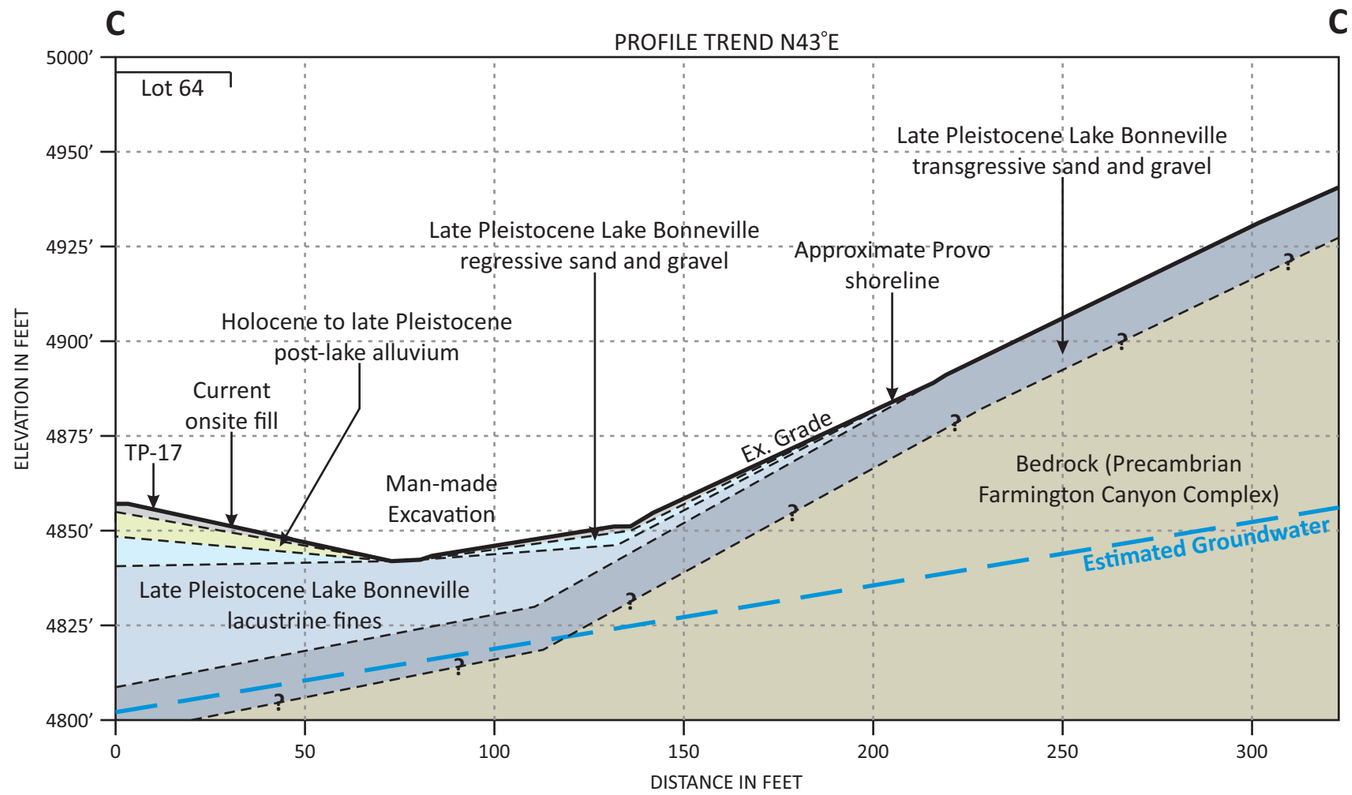
Date:	15-Jan-2021
CMT No.:	12188

Figure 4A



Scale 1 inch equals 40 feet (1:480) with no vertical exaggeration. Existing and proposed grades are from Reeve & Associates. Contact and units are inferred and approximate.

CMT ENGINEERING LABORATORIES	**Rock Loft Ridge Subdivision** About 250 North 1450 East, Fruit Heights, Utah	**Cross Section B-B**	Date:	15-Jan-2021	Figure 4B
			CMT No.:	12188	



Scale 1 inch equals 40 feet (1:480) with no vertical exaggeration. Existing and proposed grades are from Reeve & Associates. Contact and units are inferred and approximate.

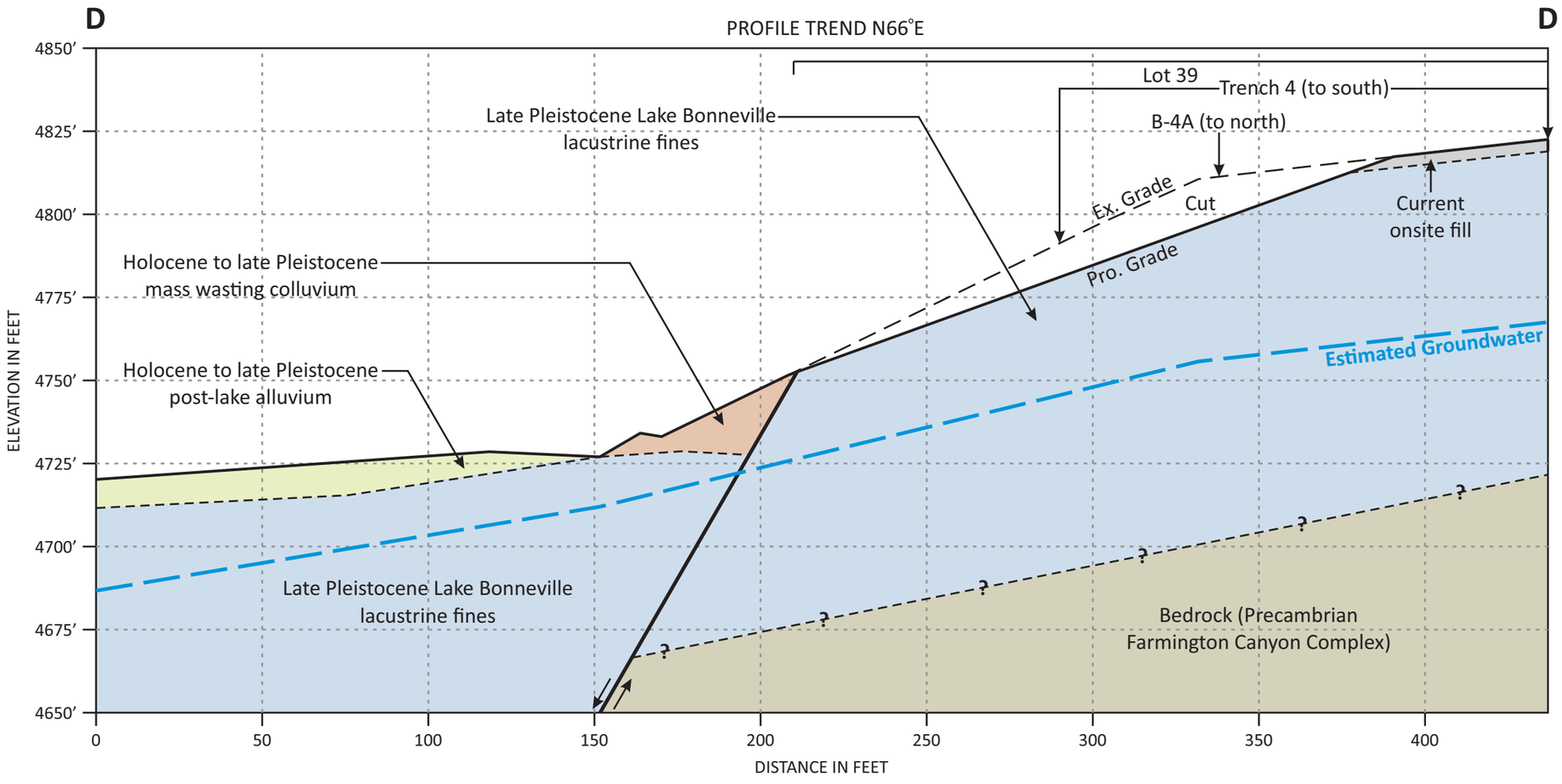
CMT ENGINEERING LABORATORIES

Rock Loft Ridge Subdivision
About 250 North 1450 East, Fruit Heights, Utah

Cross Section C-C

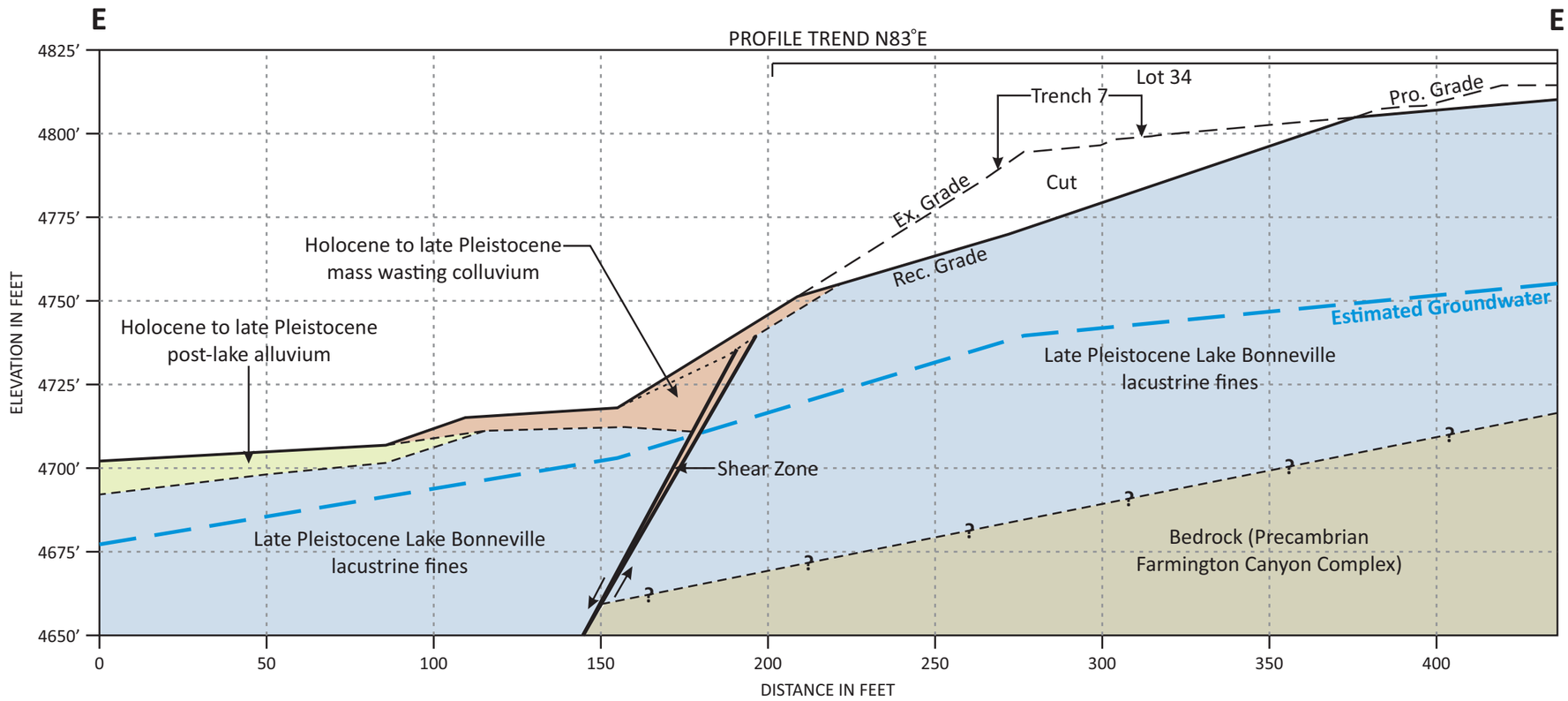
Date:	15-Jan-2021
CMT No.:	12188

Figure 4C



Scale 1 inch equals 40 feet (1:480) with no vertical exaggeration. Existing and proposed grades are from Reeve & Associates. Contact and units are inferred and approximate.

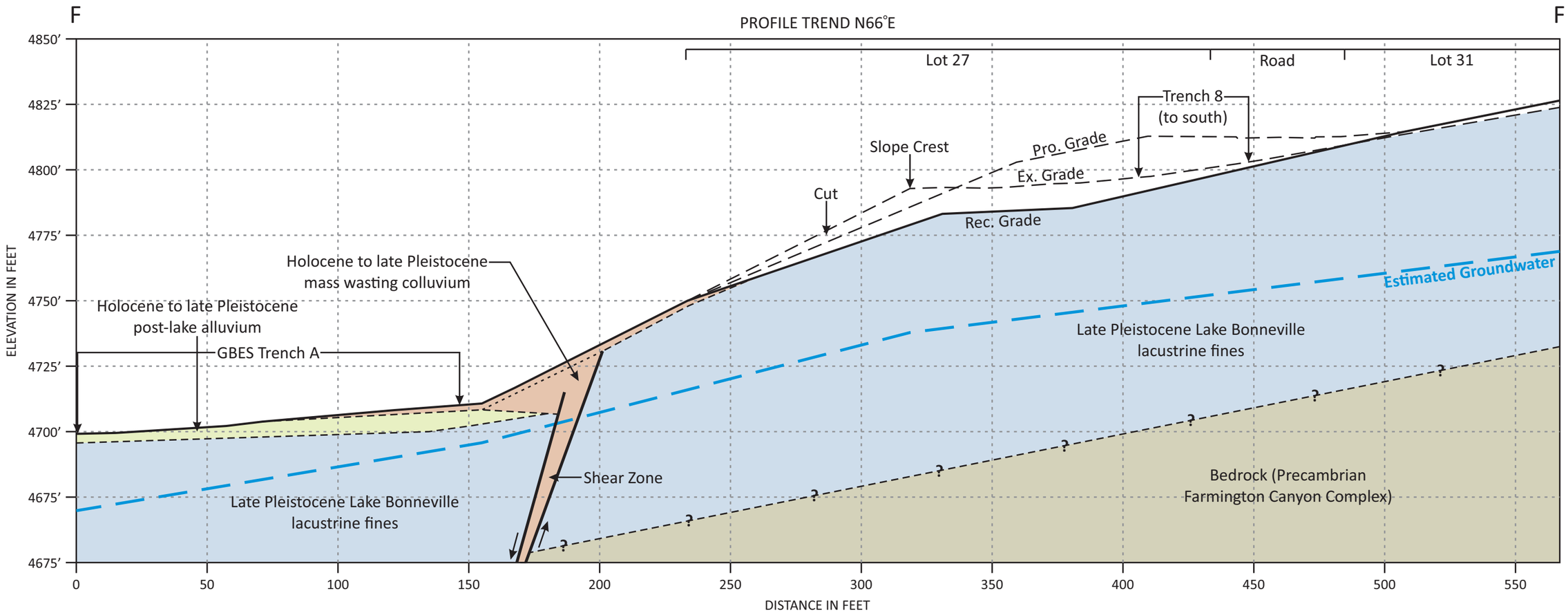
CMT ENGINEERING LABORATORIES	**Rock Loft Ridge Subdivision** About 250 North 1450 East, Fruit Heights, Utah	**Cross Section** **D-D**	Date:	15-Jan-2021	Figure 4D
			CMT No.:	12188	



The recommended grade assumes no emplacement of post-grading fill, and removal of mass movement deposits from beneath home footprints where present.

Scale 1 inch equals 40 feet (1:480) with no vertical exaggeration. Existing and proposed grades are from Reeve & Associates. Contact and units are inferred and approximate.

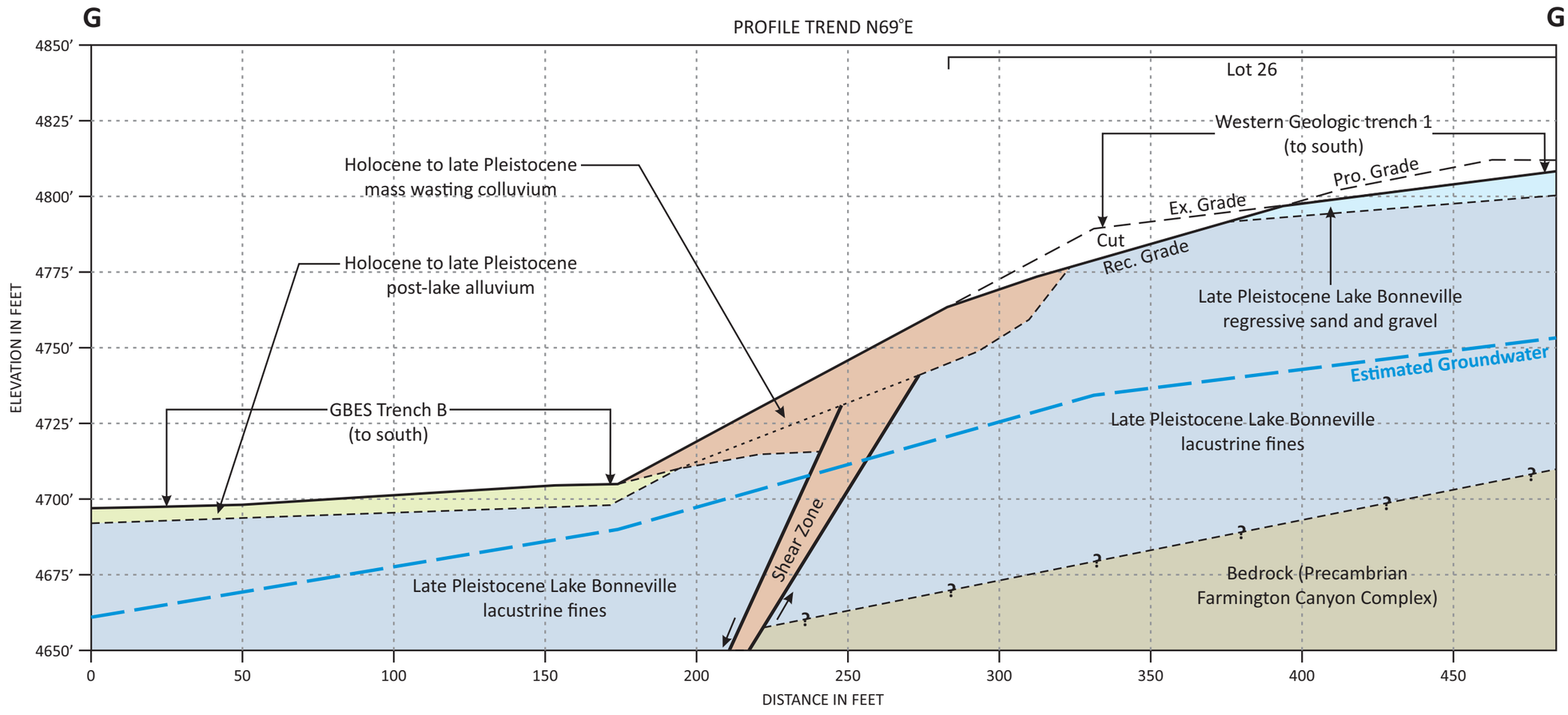
CMT ENGINEERING LABORATORIES	**Rock Loft Ridge Subdivision** About 250 North 1450 East, Fruit Heights, Utah	**Cross Section E-E**	Date:	15-Jan-2021	Figure 4E
			CMT No.:	12188	



The recommended grade assumes no emplacement of post-grading fill, and removal of mass movement deposits from beneath home footprints where present.

Scale 1 inch equals 40 feet (1:480) with no vertical exaggeration. Existing and proposed grades are from Reeve & Associates. Contact and units are inferred and approximate.

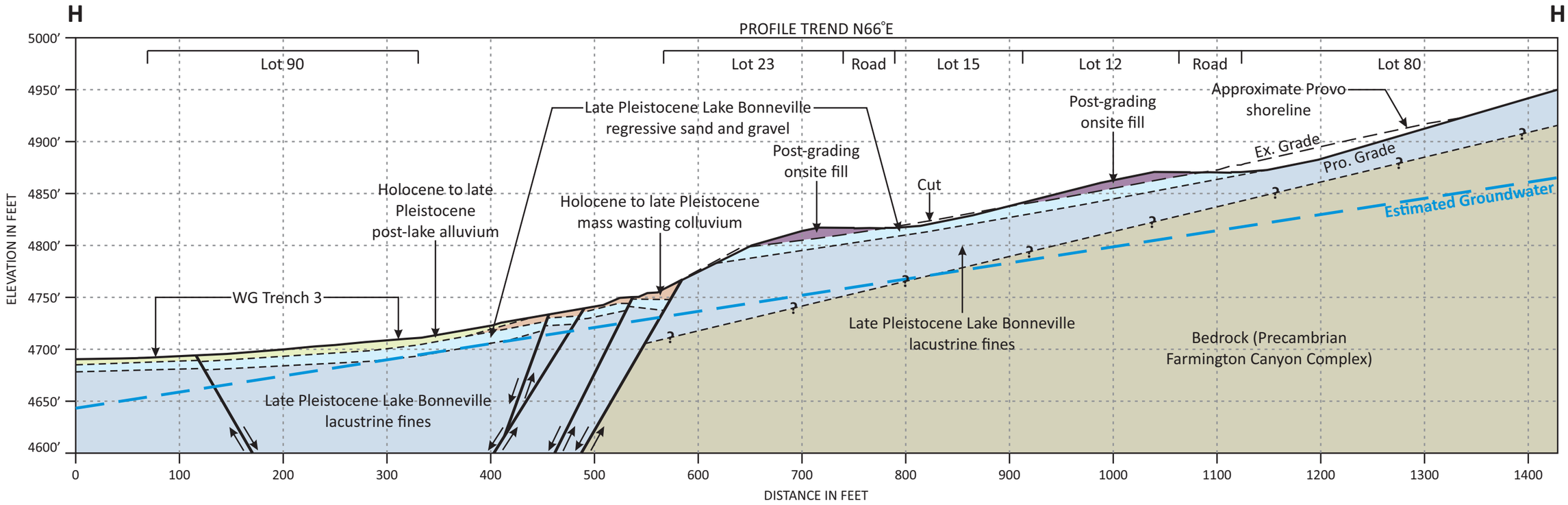
CMT ENGINEERING LABORATORIES	**Rock Loft Ridge Subdivision** About 250 North 1450 East, Fruit Heights, Utah	**Cross Section** **F-F**	Date:	15-Jan-2021	Figure 4F
			CMT No.:	12188	



The recommended grade assumes no emplacement of post-grading fill, and removal of mass movement deposits from beneath home footprints where present.

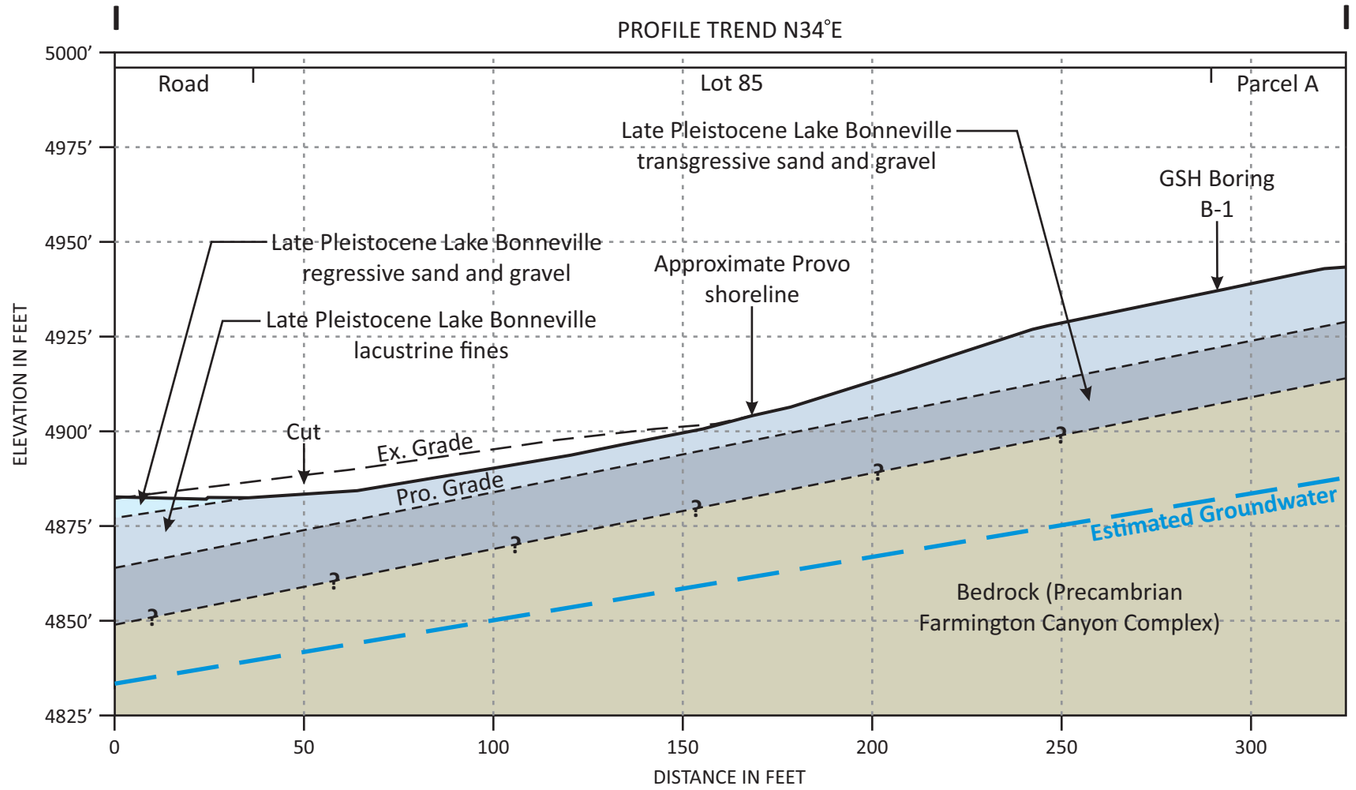
Scale 1 inch equals 40 feet (1:480) with no vertical exaggeration. Existing and proposed grades are from Reeve & Associates. Contact and units are inferred and approximate.

CMT ENGINEERING LABORATORIES	**Rock Loft Ridge Subdivision** About 250 North 1450 East, Fruit Heights, Utah	**Cross Section** **G-G**	Date:	15-Jan-2021	Figure 4G
			CMT No.:	12188	



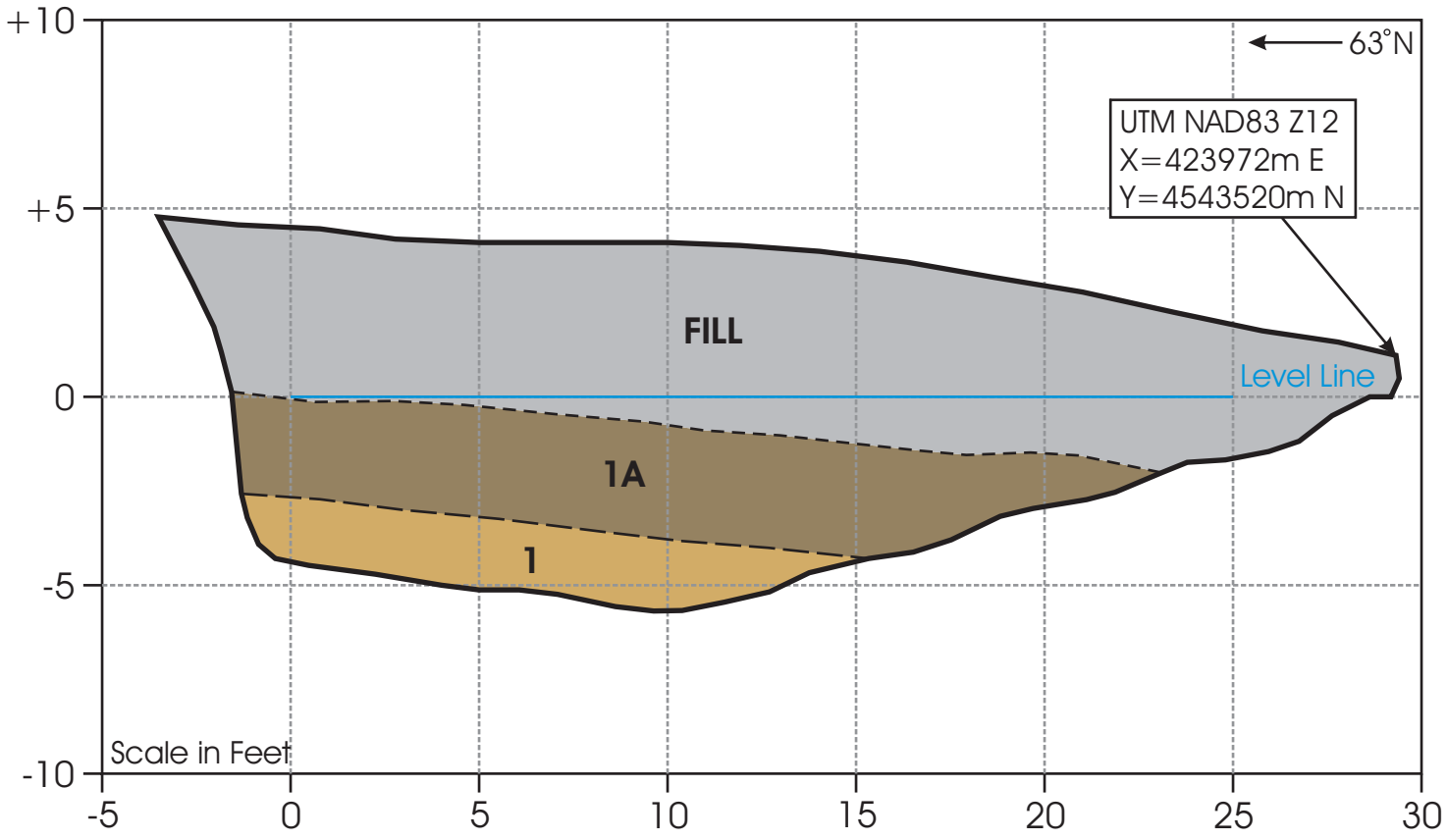
Scale 1 inch equals 100 feet (1:1,200) with no vertical exaggeration. Existing and proposed grades are from Reeve & Associates. Contact and units are inferred and approximate.

CMT ENGINEERING LABORATORIES	**Rock Loft Ridge Subdivision** About 250 North 1450 East, Fruit Heights, Utah	**Cross Section** **H-H**	Date:	15-Jan-2021	Figure 4H
			CMT No.:	12188	



Scale 1 inch equals 40 feet (1:480) with no vertical exaggeration. Existing and proposed grades are from Reeve & Associates. Contact and units are inferred and approximate.

CMT ENGINEERING LABORATORIES	**Rock Loft Ridge Subdivision** About 250 North 1450 East, Fruit Heights, Utah	**Cross Section** **I-I**	Date: 15-Jan-2021 CMT No.: 12188	Figure 4I



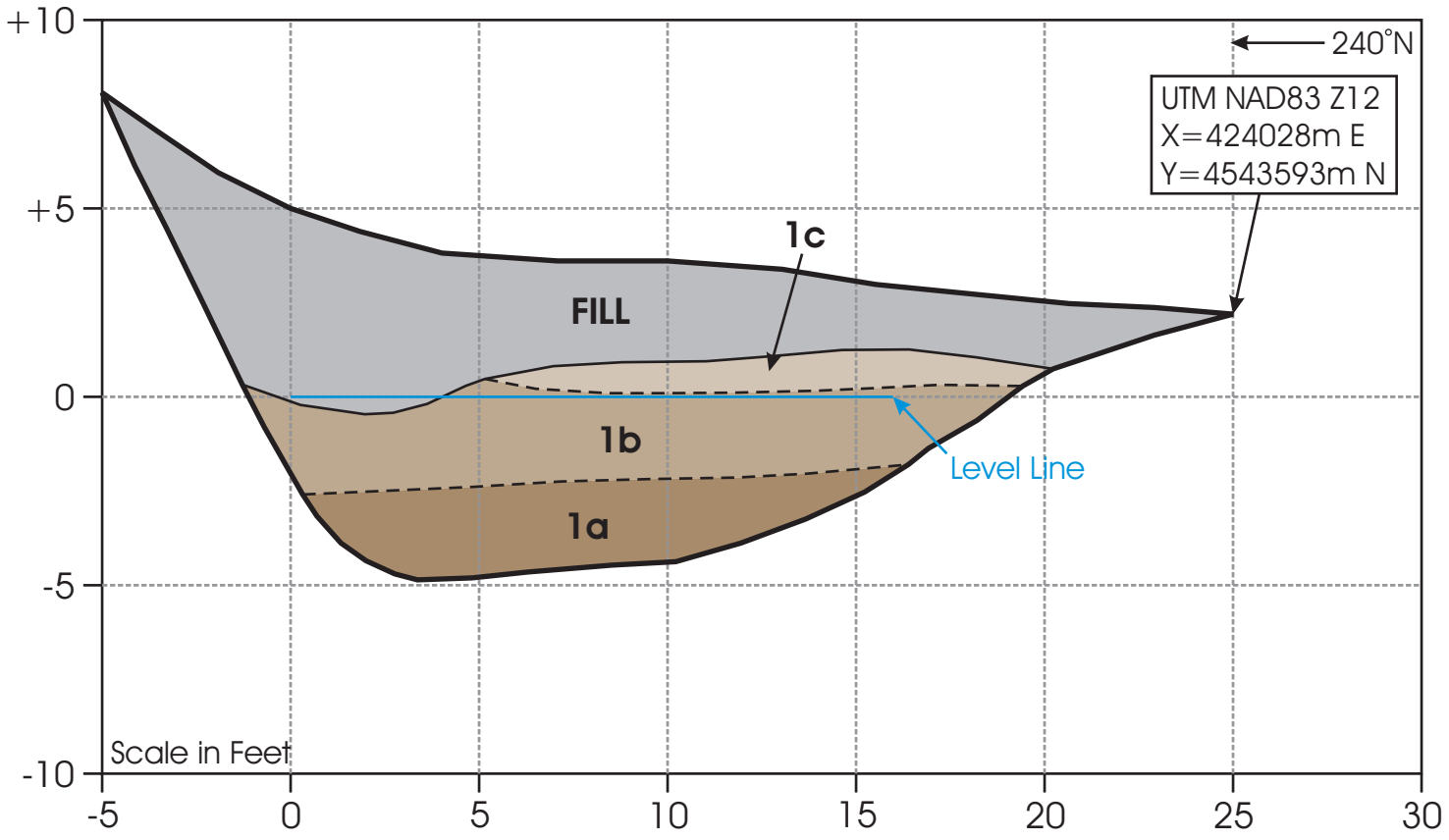
UNIT DESCRIPTIONS

Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleistocene Lake Bonneville* - Yellowish- to dark-grayish-brown, poorly bedded, moderate density, gravelly sand to sandy gravel (SW/GW) with cobbles and trace boulders; clasts subangular with stage II carbonate, includes a schist boulder about 2.5 feet across.

1A. A-horizon paleosol formed in unit 1, overlain by fill from gravel pit operations.

Logged by Bill D. Black, P.G. on December 12, 2018.
South wall logged, east to west. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

Rock Loft Ridge Subdivision About 250 North 1450 East, Fruit Heights, Utah	CMT ENGINEERING LABORATORIES **Test Pit 1 Log**	Date: 24-Oct-2020 Job # 12188	Figure 5A

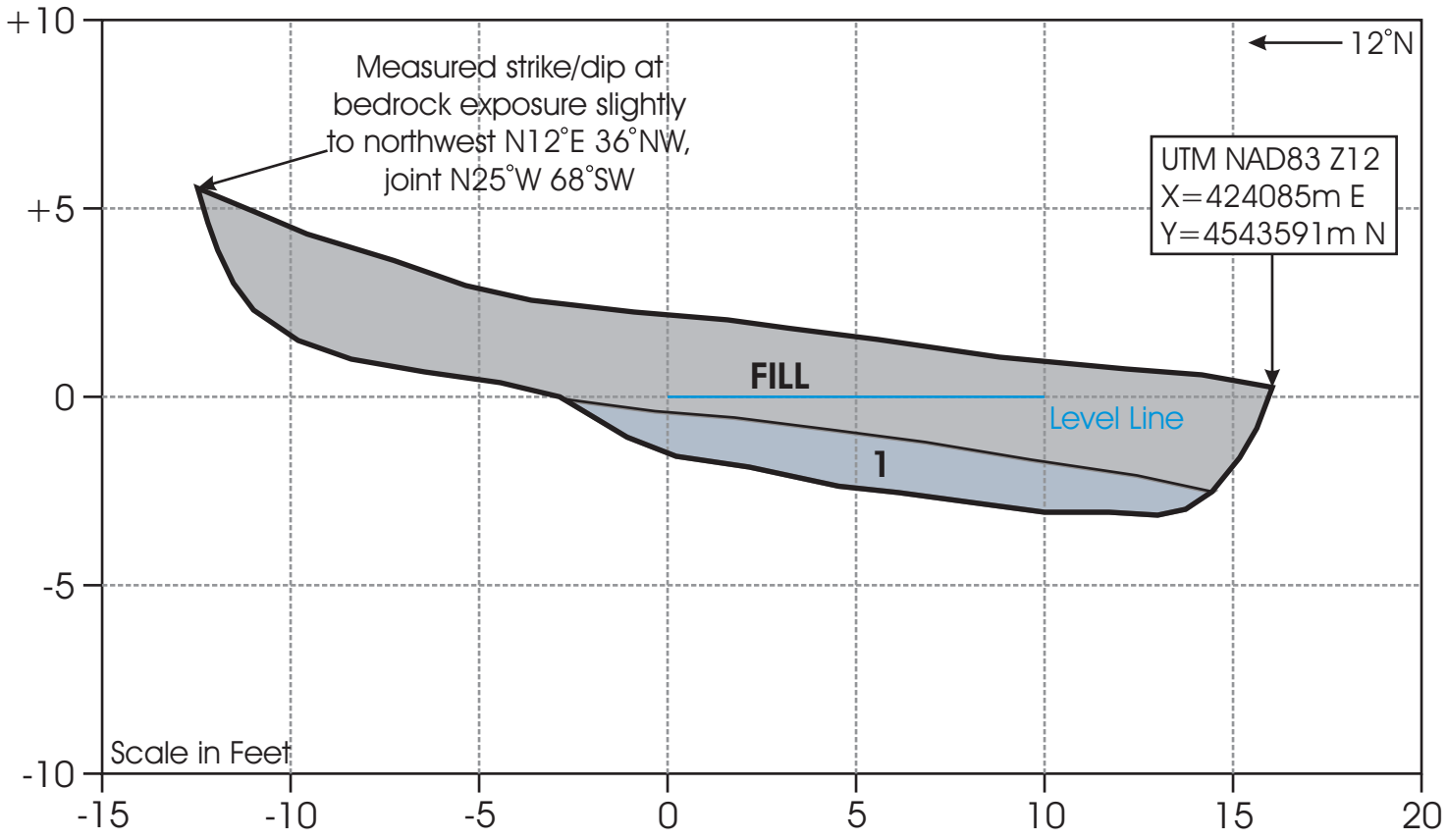


UNIT DESCRIPTIONS

Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleistocene Lake Bonneville* - Generally well to poorly bedded, moderate density, fining-upward sequence comprised of a lower (**1a**), brown, sandy gravel (GW) with cobbles and boulders; a middle (**1b**), brown to pale-brown, gravelly sand (SW) with trace cobbles; and an upper (**1c**), pale-brown, silty fine sand to sandy silt (SM/ML); clasts round to subangular with stage II carbonate; lower subunit includes a fractured schist boulder about 2 feet across.

Logged by Bill D. Black, P.G. on December 12, 2018.
North wall logged, west to east. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

Rock Loft Ridge Subdivision About 250 North 1450 East, Fruit Heights, Utah	CMT ENGINEERING LABORATORIES **Test Pit 2 Log**	Date: 24-Oct-2020 Job # 12188	Figure **5B**

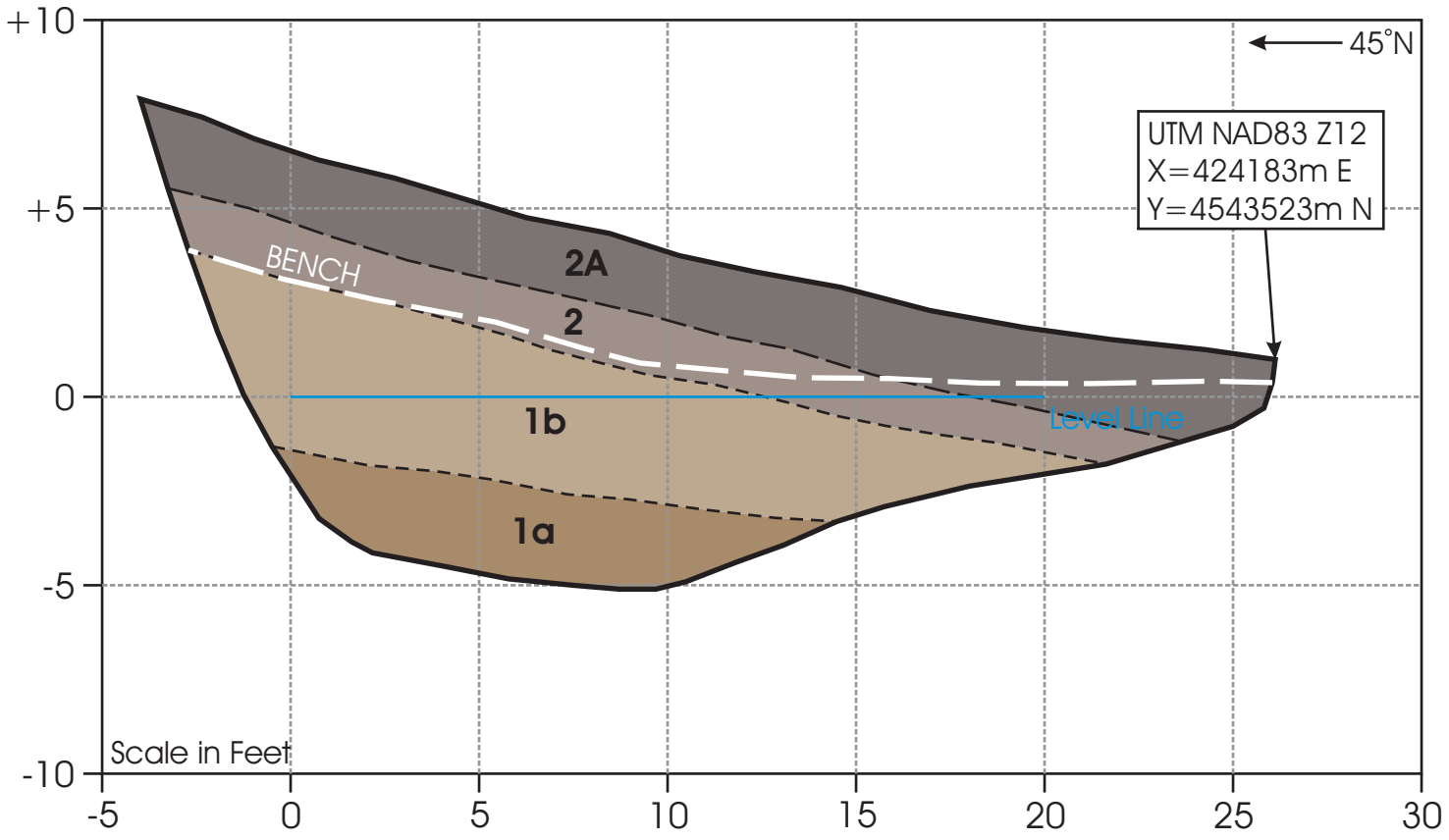


UNIT DESCRIPTIONS

Unit 1. Bluish-gray, very dense, thinly bedded, weathered schist bedrock of the Farmington Canyon Complex; trackhoe refusal in unit.

Logged by Bill D. Black, P.G. on December 12, 2018.
East wall logged, north to south. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

Rock Loft Ridge Subdivision About 250 North 1450 East, Fruit Heights, Utah	CMT ENGINEERING LABORATORIES **Test Pit 3 Log**	Date: 24-Oct-2020 Job # 12188	Figure 5C



UNIT DESCRIPTIONS

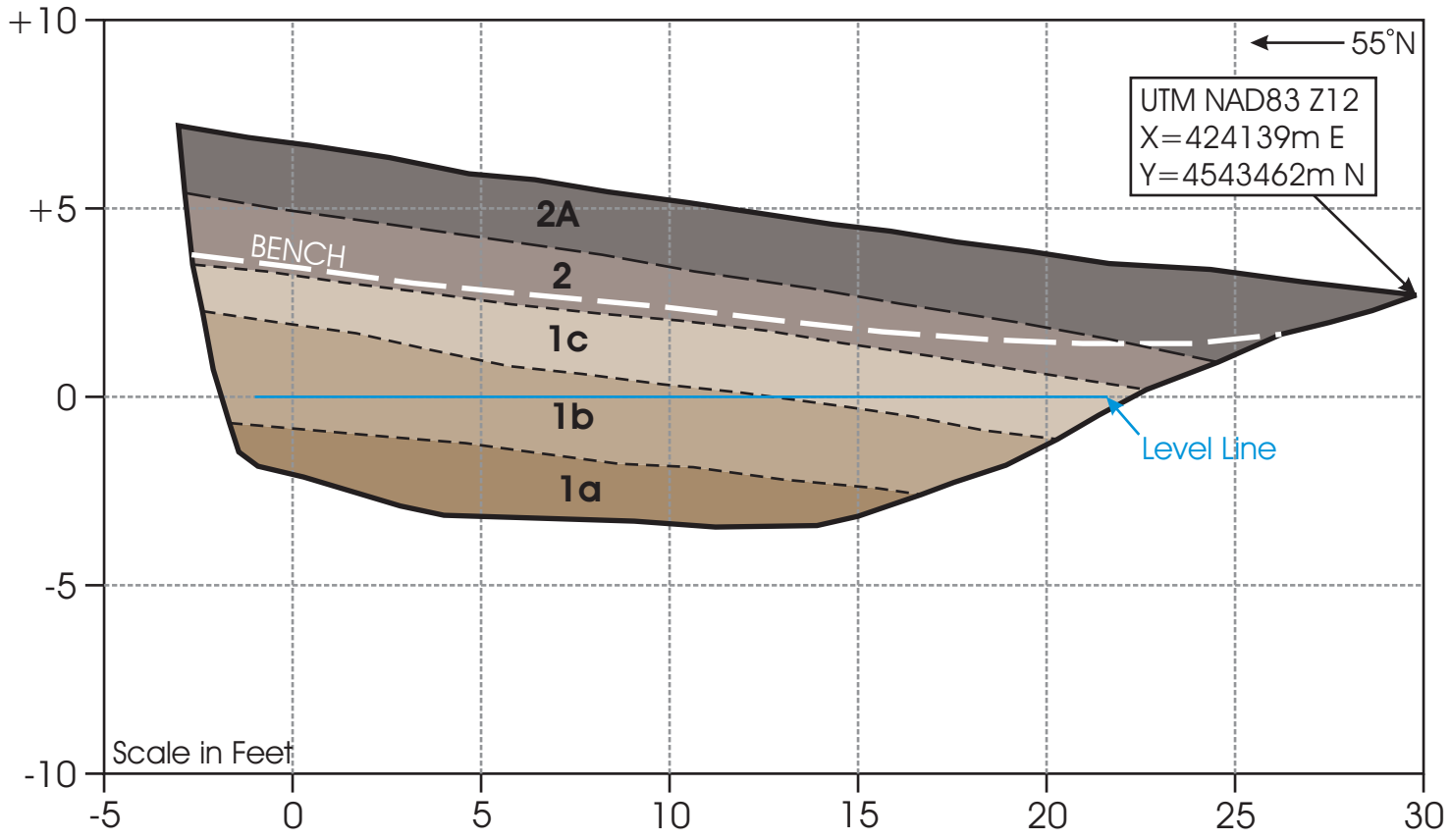
Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleistocene Lake Bonneville* - Generally well to poorly bedded, moderate density, fining-upward sequence comprised of a lower (**1a**), brown, sandy gravel (GW) with cobbles and boulders; and an upper (**1b**), brown to pale-brown, gravelly sand (SW) with trace cobbles; clasts round to subangular with stage II carbonate.

Unit 2. *Post-Lake Bonneville alluvium* - Organic-enriched, brown to dark-grayish-brown, low to moderate density, generally massive, gravelly sand (SW) with cobbles and trace boulders; clasts subround to angular with stage II carbonate; upper part disturbed and reworked by historical gravel mining and/or agricultural operations.

2A. Partly disturbed modern A-horizon soil formed in unit 2.

Logged by Bill D. Black, P.G. on December 12, 2018.
South wall logged, east to west. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

Rock Loft Ridge Subdivision About 250 North 1450 East, Fruit Heights, Utah	CMT ENGINEERING LABORATORIES **Test Pit 4 Log**	Date: 24-Oct-2020 Job # 12188	Figure 5D



UNIT DESCRIPTIONS

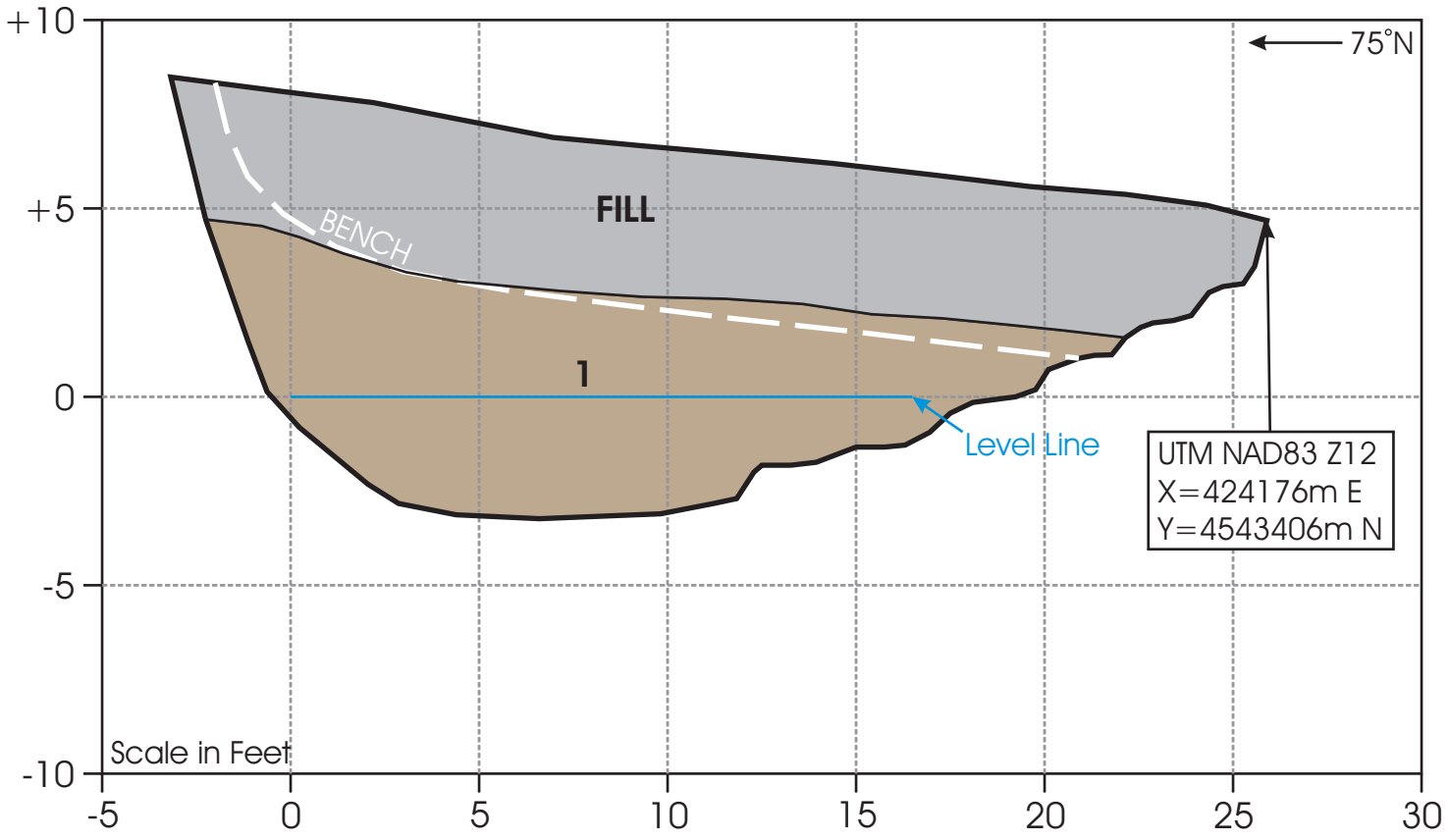
Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleistocene Lake Bonneville* - Interbedded sequence of generally pale-brown, moderate density, sand to gravel; comprised of a lower (**1a**), gravelly, mainly coarse but well-graded sand (SW-SP) with trace cobbles; a middle (**1b**), sandy gravel (GW) with cobbles; and an upper (**1c**), gravelly sand to sandy gravel (SW/GW) with trace cobbles; clasts round to subangular with stage II carbonate.

Unit 2. *Post-Lake Bonneville alluvium* - Organic-enriched, dark-grayish-brown, low to moderate density, root penetrated, moist, generally massive, sand (SW) with silt, gravel, cobbles and boulders up to 3' across; clasts angular to subround with stage II carbonate; upper part disturbed and reworked by historical gravel mining and/or agricultural operations.

2A. Partly disturbed modern A-horizon soil formed in unit 2.

Logged by Bill D. Black, P.G. on December 13, 2018.
South wall logged, east to west. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

Rock Loft Ridge Subdivision About 250 North 1450 East, Fruit Heights, Utah	CMT ENGINEERING LABORATORIES **Test Pit 5 Log**	Date: 24-Oct-2020 Job # 12188	Figure 5E

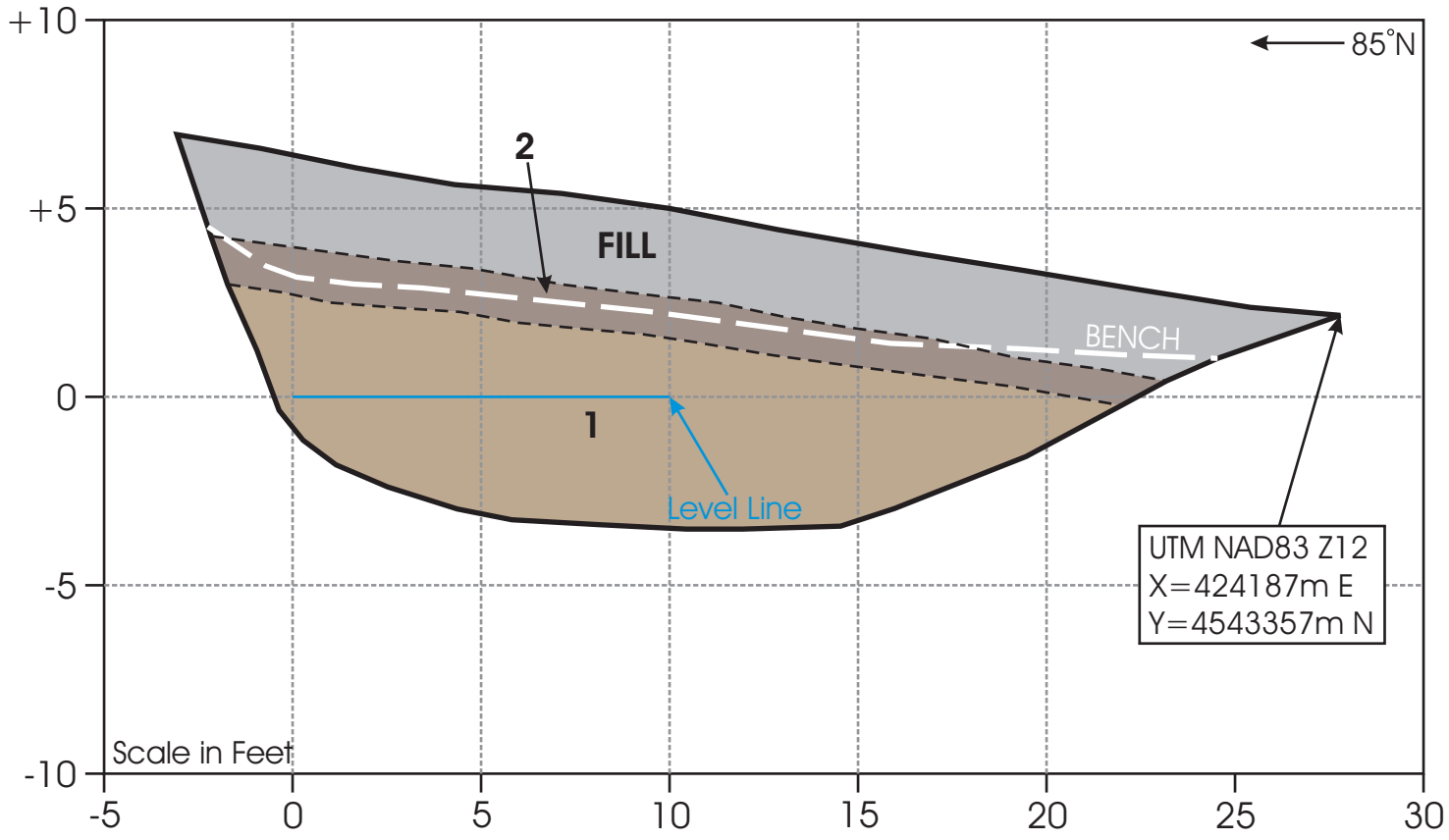


UNIT DESCRIPTIONS

Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleistocene Lake Bonneville* - Brown to pale-brown; poorly to well-bedded, moderate density, gravelly sand to sandy gravel (SW/GW) with trace small cobbles; bedding dips to west at from 6-8 degrees.

Logged by Bill D. Black, P.G. on December 13, 2018.
South wall logged, east to west. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

Rock Loft Ridge Subdivision About 250 North 1450 East, Fruit Heights, Utah	CMT ENGINEERING LABORATORIES **Test Pit 6 Log**	Date: 24-Oct-2020 Job # 12188	Figure 5F



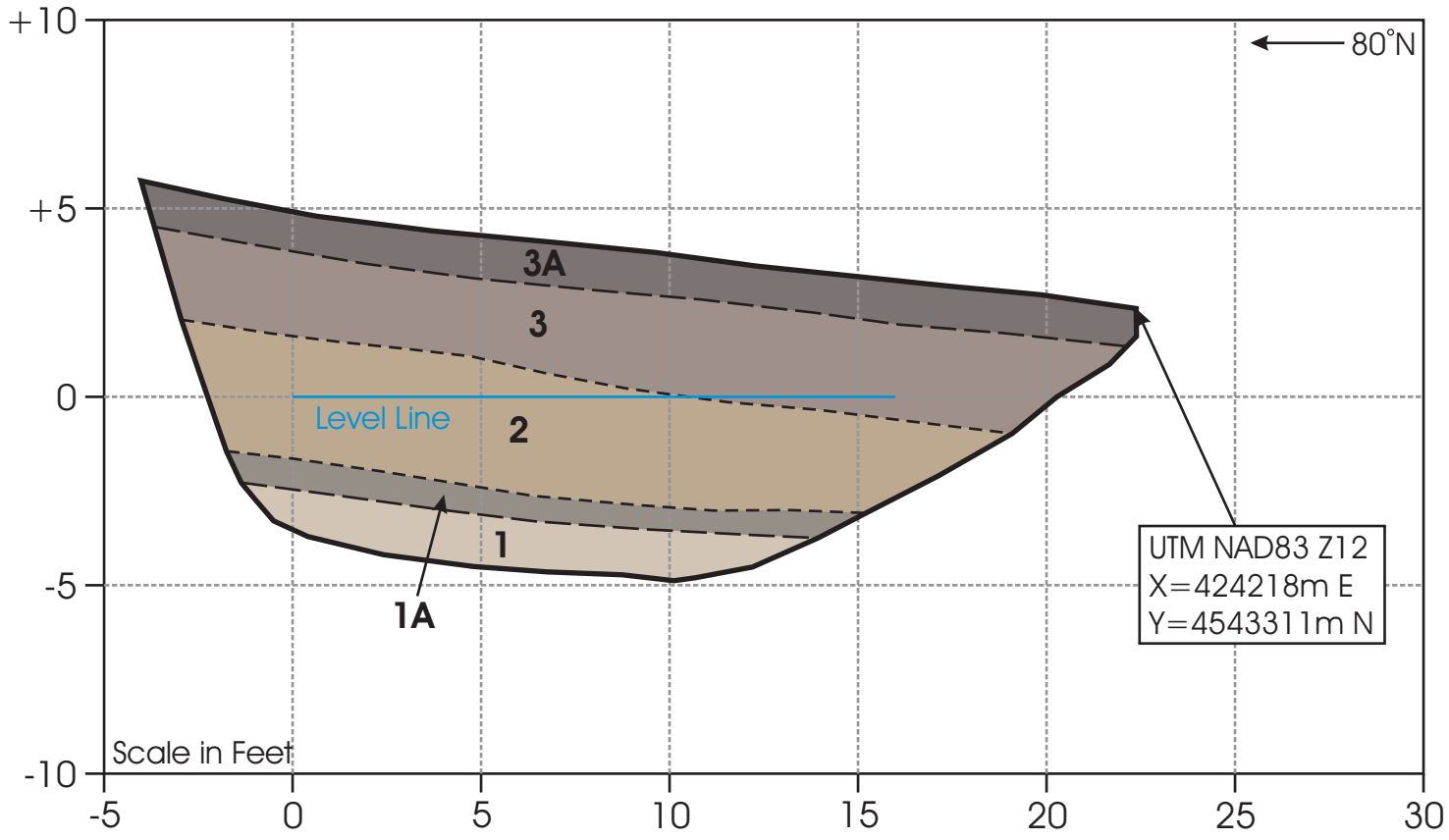
UNIT DESCRIPTIONS

Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleitsocene Lake Bonneville* - Brown to pale brown, poorly bedded, moderate density, gravelly sand to sandy gravel (SW/GW) with trace cobbles and boulders; clasts round to subangular with stage II carbonate.

Unit 2. *Post-Lake Bonneville alluvium* - Organic-enriched, brown to dark-grayish-brown, moist, low to moderate density, massive, gravelly sand (SW) with trace cobbles; clasts subangular with stage II carbonate; upper part disturbed and reworked by historical gravel mining and/or agricultural operations.

Logged by Bill D. Black, P.G. on December 13, 2018.
South wall logged, east to west. Scale 1 inch equals
5 feet (1:60) with no vertical exaggeration.

Rock Loft Ridge Subdivision About 250 North 1450 East, Fruit Heights, Utah	CMT ENGINEERING LABORATORIES **Test Pit 7 Log**	Date: 24-Oct-2020 Job # 12188	Figure 5G



UNIT DESCRIPTIONS

Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleitsocene Lake Bonneville* - Brown to pale brown, poorly bedded, moderate density, gravelly sand to sandy gravel (SW/GW) with trace cobbles and boulders; clasts round to subangular with stage II carbonate.

1A. Paleosol A horizon formed in unit 1.

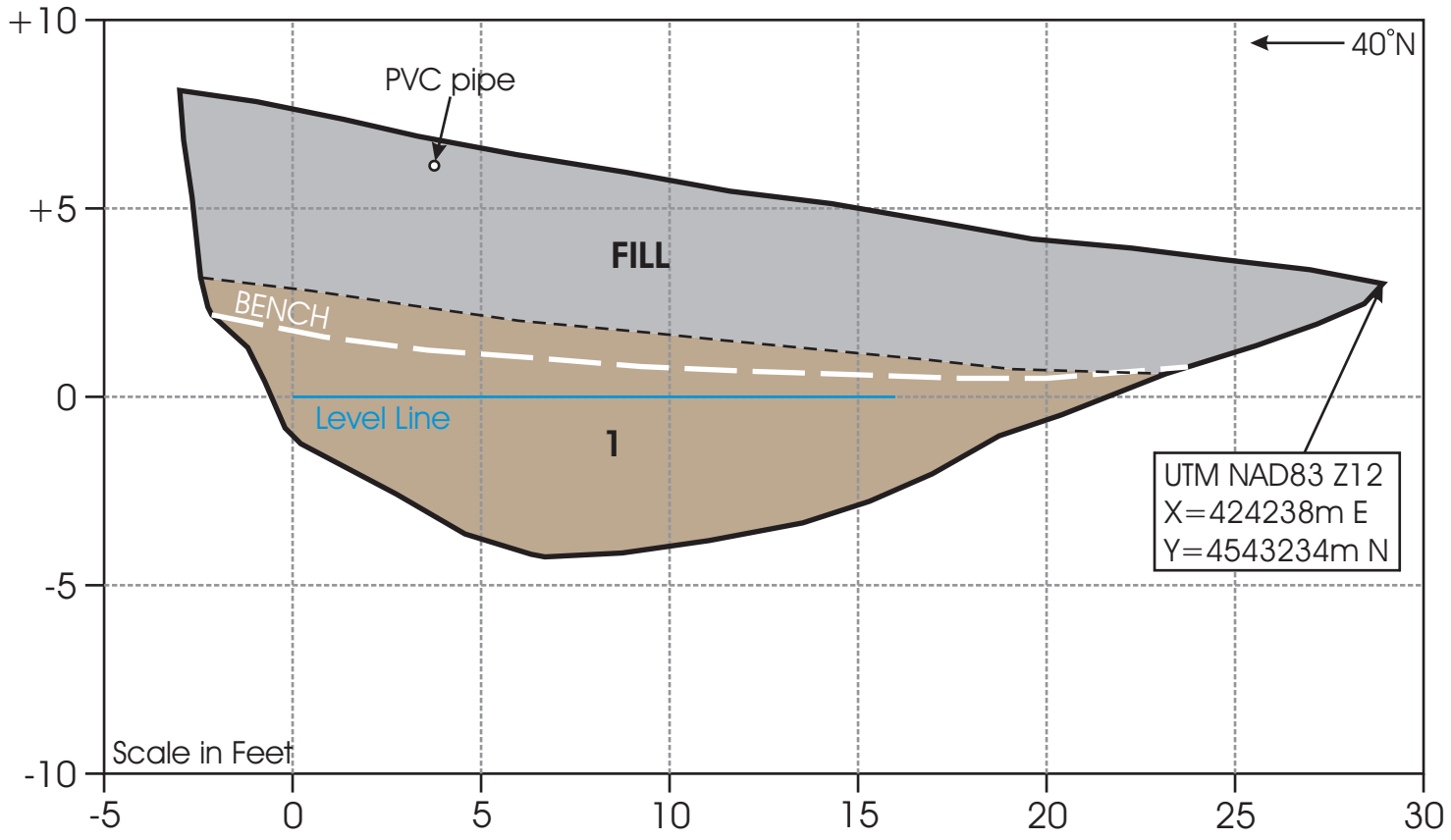
Unit 2. *Post-Lake Bonneville debris flow deposit (?)* - Brown to pale brown, massive, moderate density, gravelly sand to sandy gravel (SW/GW) with cobbles and boulders up to several feet across; clasts subangular to subround with stage II carbonate.

Unit 3. *Younger post-Lake Bonneville alluvium (?)* - Organic-enriched, brown to dark-grayish-brown, moist, low to moderate density, generally massive, gravelly sand (SW) with trace cobbles; clasts subangular with stage II carbonate; upper part disturbed and reworked by historical gravel mining and/or agricultural operations.

3A. Disturbed modern A-horizon soil formed in unit 3.

Logged by Bill D. Black, P.G. on December 13, 2018.
South wall logged, east to west. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

Rock Loft Ridge Subdivision About 250 North 1450 East, Fruit Heights, Utah	CMT ENGINEERING LABORATORIES **Test Pit 8 Log**	Date: 24-Oct-2020 Job # 12188	Figure **5H**



UNIT DESCRIPTIONS

Unit 1. *Post-Lake Bonneville alluvium* - Brown, massive, moderate density, gravelly sand to sandy gravel (SW/GW) with trace cobbles and boulders up to 3' across; clasts angular to subround with stage II carbonate; overlain by fill from historical gravel mining and/or agricultural operations that may be derived from younger alluvial deposits previously reworked and disturbed at the surface.

Logged by Bill D. Black, P.G. on December 13, 2018.
South wall logged, east to west. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

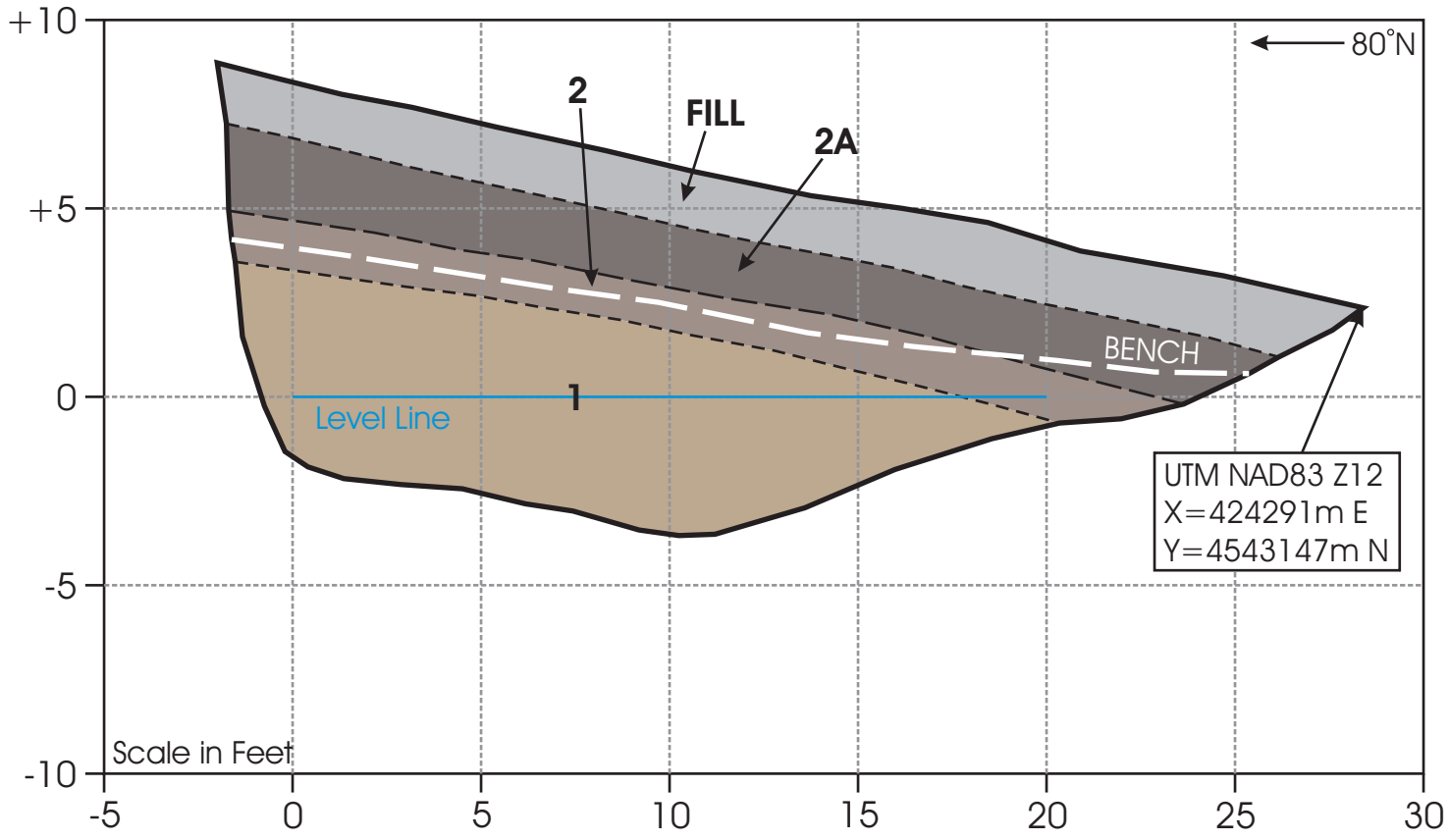
Rock Loft Ridge Subdivision
About 250 North 1450 East, Fruit Heights, Utah

CMT ENGINEERING LABORATORIES

Test Pit 9 Log

Date:	24-Oct-2020
Job #	12188

Figure 5I



UNIT DESCRIPTIONS

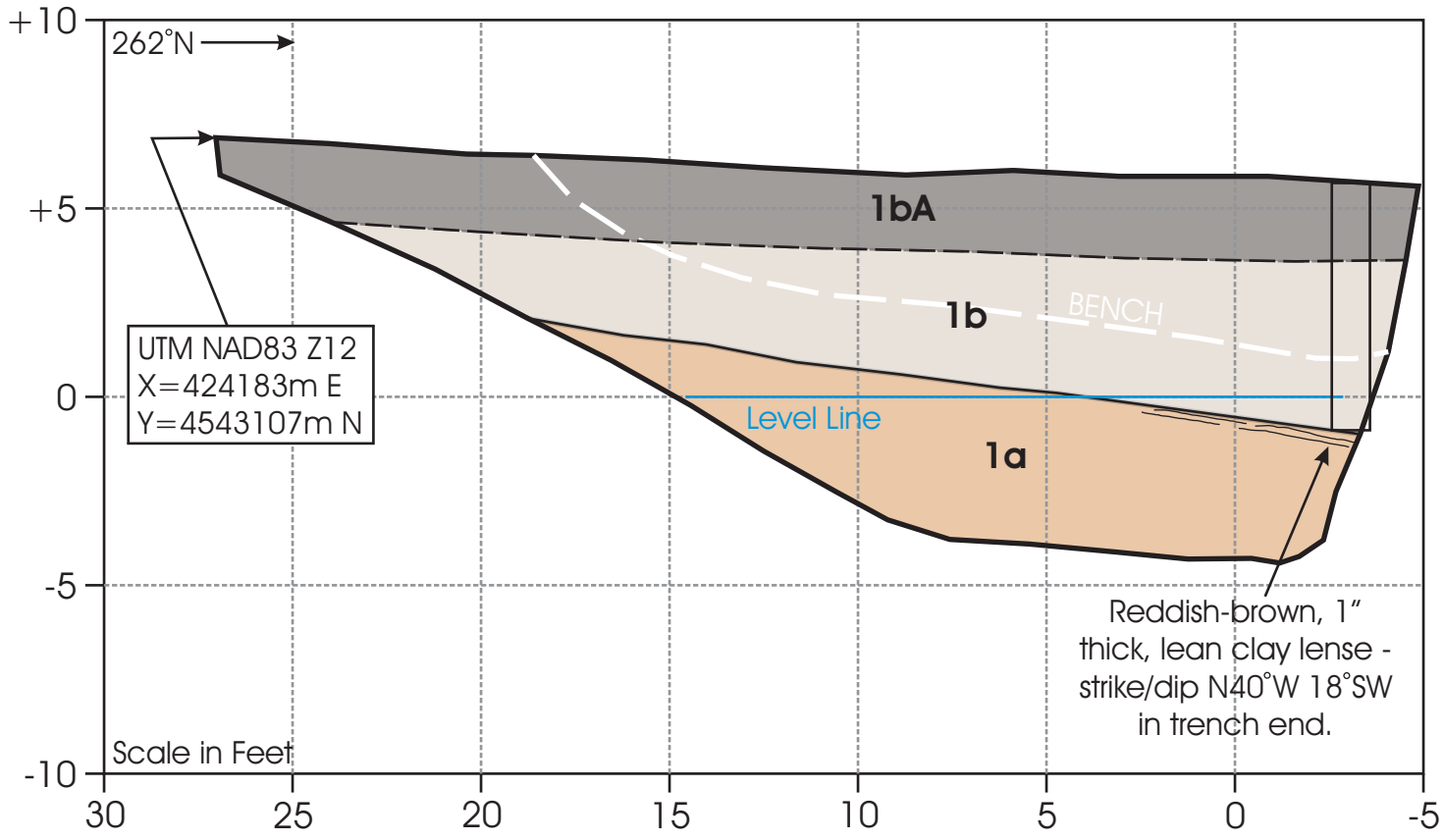
Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleistocene Lake Bonneville* - Grayish-brown, moderate density, poorly bedded, gravelly sand (SW) with trace cobbles; clasts round to subround with stage II carbonate.

Unit 2. *Post-Lake Bonneville alluvium* - Organic-enriched, brown to dark-brown, low to moderate density, massive, gravelly sand (SW) with silt; upper part disturbed and reworked by historical gravel mining and/or agricultural operations.

2A. Partly disturbed modern A-horizon soil formed in unit 2.

Logged by Bill D. Black, P.G. on December 13, 2018.
South wall logged, east to west. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

Rock Loft Ridge Subdivision About 250 North 1450 East, Fruit Heights, Utah	CMT ENGINEERING LABORATORIES **Test Pit 10 Log**	Date: 24-Oct-2020 Job # 12188	Figure 5J



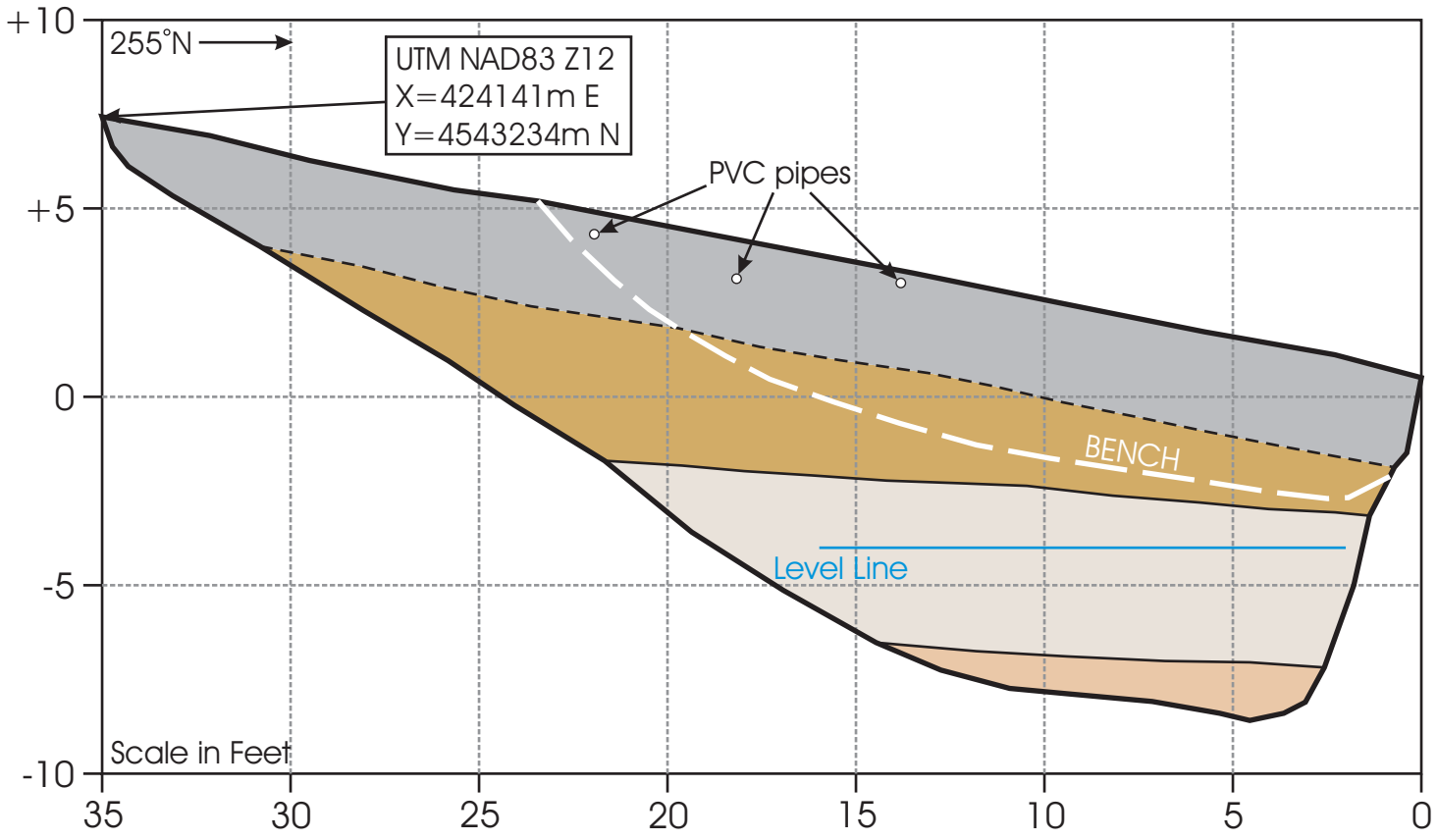
UNIT DESCRIPTIONS

Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleistocene Lake Bonneville* - Sequence of well-bedded, moderate density, silt to gravel comprised of a lower (**1a**), Pale-reddish-brown, silt (ML) with thin interbeds of reddish-brown lean clay and fine sand, including a topset clay lense; and an upper (**1b**), sub-Provo beach deposit comprised of interbedded fine to coarse sand and sandy pea gravel (SW/GW) with ostracods, generally poorly graded within beds but well graded overall.

1bA. Modern A-horizon soil formed in unit 1b.

Logged by Bill D. Black, P.G. on December 14, 2018.
South wall logged, west to east. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

Rock Loft Ridge Subdivision About 250 North 1450 East, Fruit Heights, Utah	CMT ENGINEERING LABORATORIES **Test Pit 11 Log**	Date: 24-Oct-2020 Job # 12188	Figure 5K

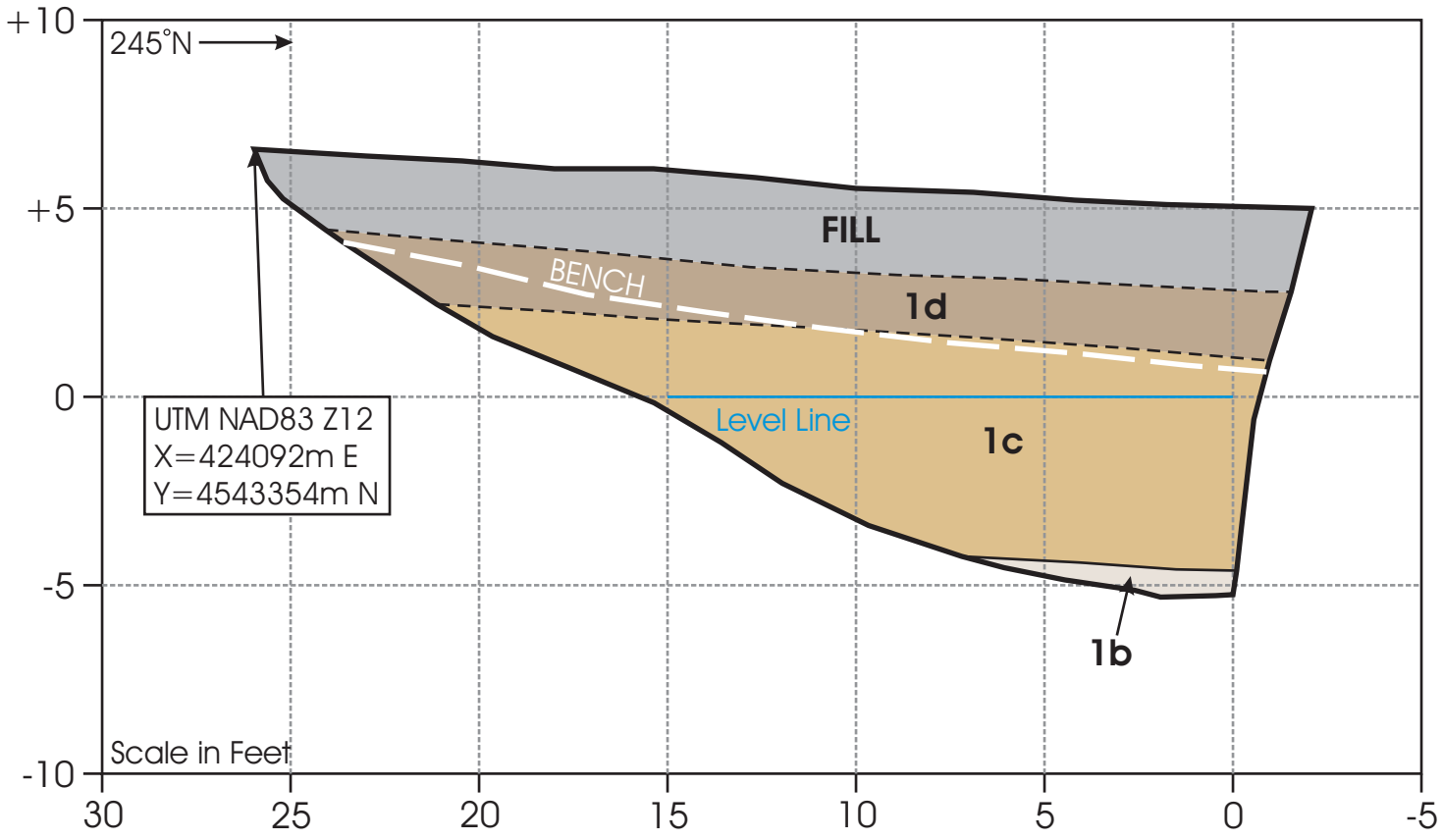


UNIT DESCRIPTIONS

Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleistocene Lake Bonneville* - Sequence of poorly to well-bedded, mainly silt to sand comprised of a lower (**1a**), pinkish-brown, moderate density, silt (ML) with thin fine sand interbeds; a middle (**1b**), sub-Provo beach deposit comprised of interbedded fine to medium sand and sandy pea gravel (SW/GW) with ostracods, generally poorly graded within beds but well graded overall; and an upper (**1c**) orange-brown, moderate density, silty sand (SM) with pebble gravel, upper part disturbed and overlain by fill.

Logged by Bill D. Black, P.G. on December 14, 2018.
South wall logged, west to east. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

Rock Loft Ridge Subdivision About 250 North 1450 East, Fruit Heights, Utah	CMT ENGINEERING LABORATORIES **Test Pit 12 Log**	Date: 24-Oct-2020 Job # 12188	Figure 5L

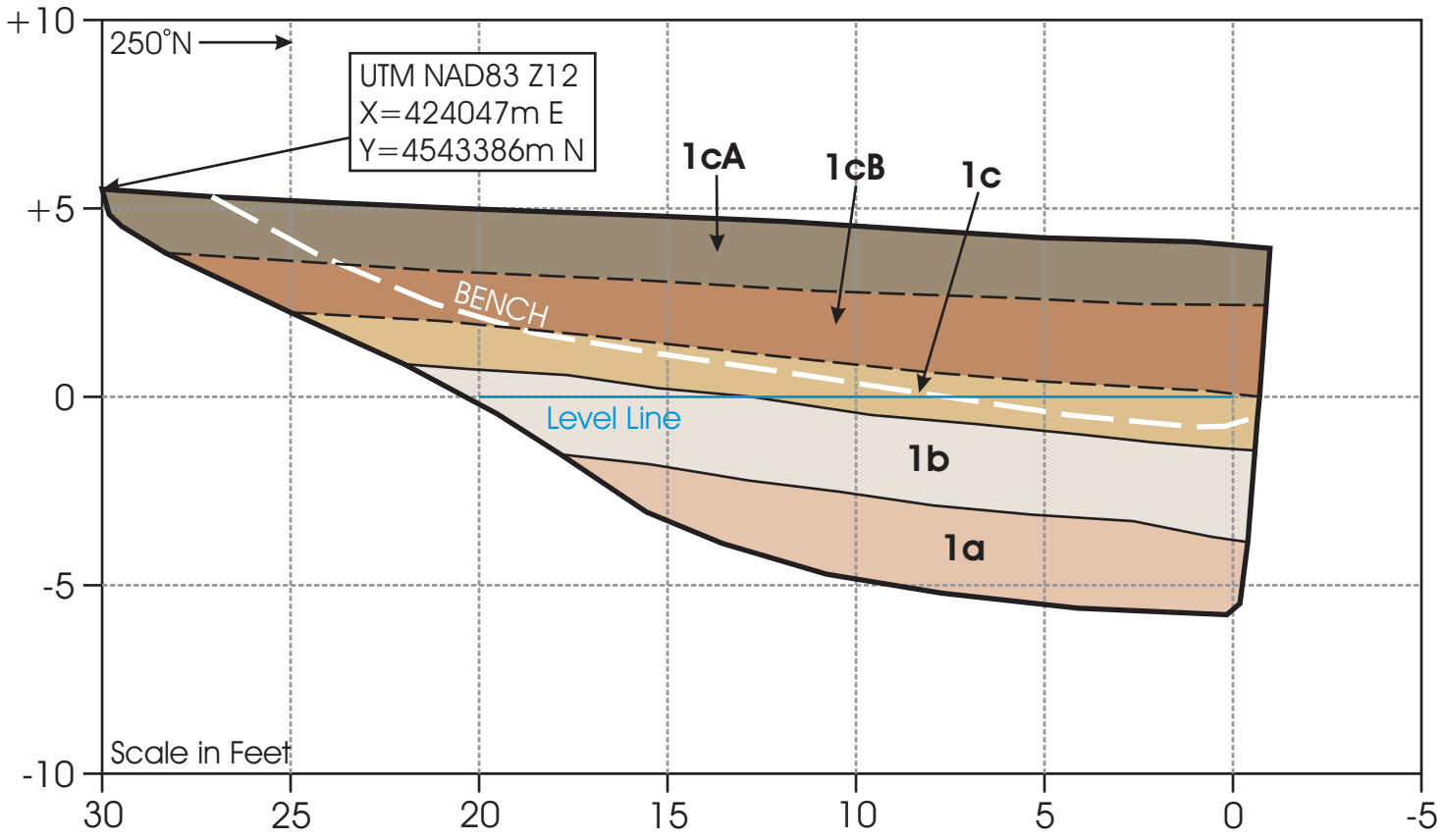


UNIT DESCRIPTIONS

Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleistocene Lake Bonneville* - Sequence of well-bedded, silt to sand comprised of a lower (**1b**), sub-Provo beach deposit comprised of interbedded fine to coarse sand and sandy pea gravel (SW/GW) with ostracods, generally poorly graded within beds but well graded overall; a middle (**1c**), orange-brown, silty sand to sandy silt (SM/ML) with trace pebble gravel; and an upper (**1d**), brown, sandy gravel (GW) with subangular to round cobbles and trace boulders with stage II carbonate.

Logged by Bill D. Black, P.G. on December 17, 2018.
South wall logged, west to east. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

Rock Loft Ridge Subdivision About 250 North 1450 East, Fruit Heights, Utah	CMT ENGINEERING LABORATORIES **Test Pit 13 Log**	Date: 24-Oct-2020 Job # 12188	Figure 5M



UNIT DESCRIPTIONS

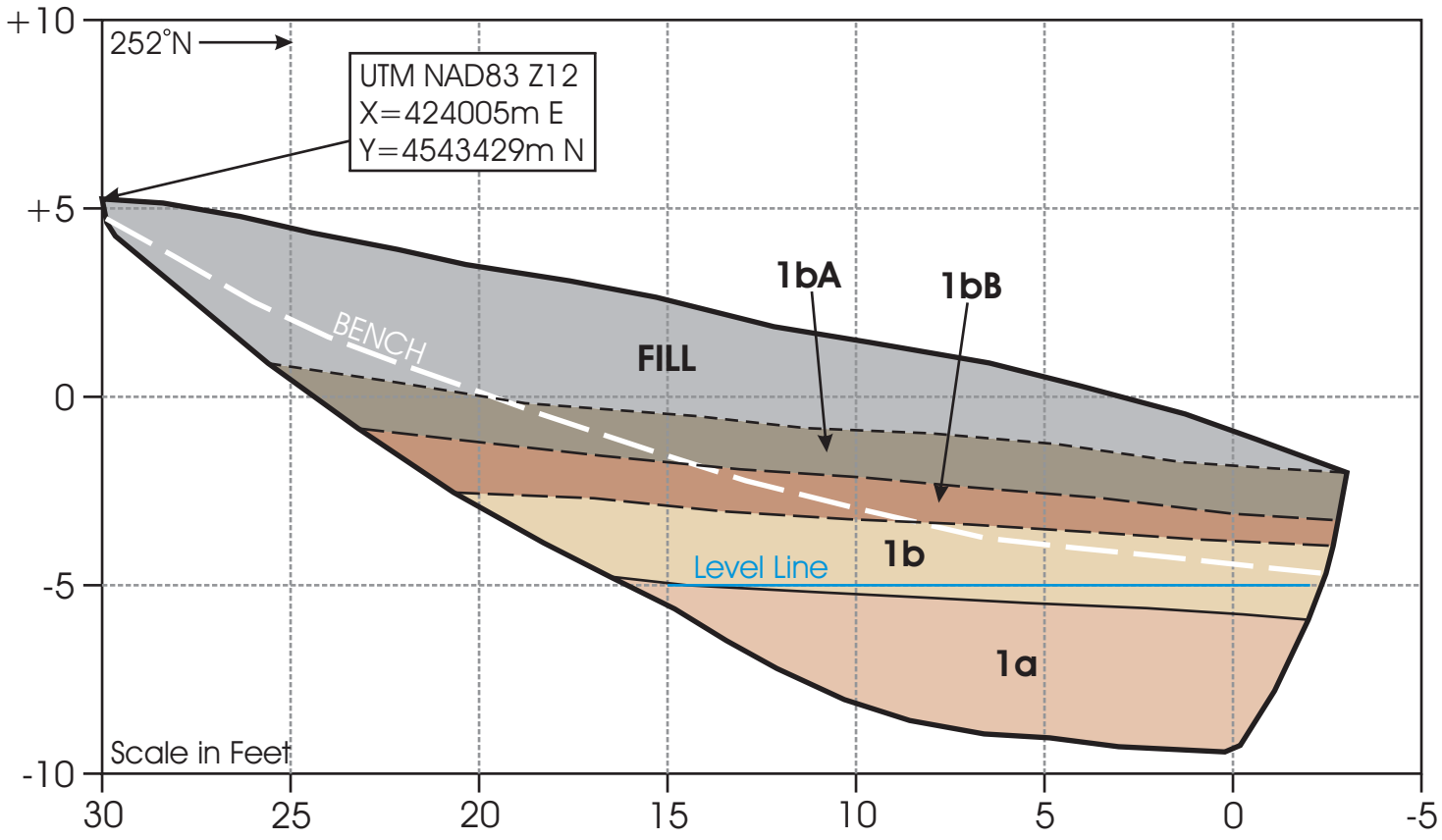
Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleistocene Lake Bonneville* - Sequence of well-bedded silt to sand comprised of a lower (**1a**), Pale-golden-brown, moderate density, interbedded silt (ML) and fine sand (SM); a middle (**1b**), pale-pinkish-brown, fine sand with ostracods and thin silt lamina cemented with carbonate; and an upper (**1c**) orange-brown, massive, silty sand (SM) with trace pea gravel and small rounded cobbles, basal silt lense along contact.

1cA. Modern A-horizon soil formed in unit 1c

1cB. B soil horizon formed in unit 1c.

Logged by Bill D. Black, P.G. on December 17, 2018.
South wall logged, west to east. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

Rock Loft Ridge Subdivision About 250 North 1450 East, Fruit Heights, Utah	CMT ENGINEERING LABORATORIES **Test Pit 14 Log**	Date: 24-Oct-2020 Job # 12188	Figure 5N



UNIT DESCRIPTIONS

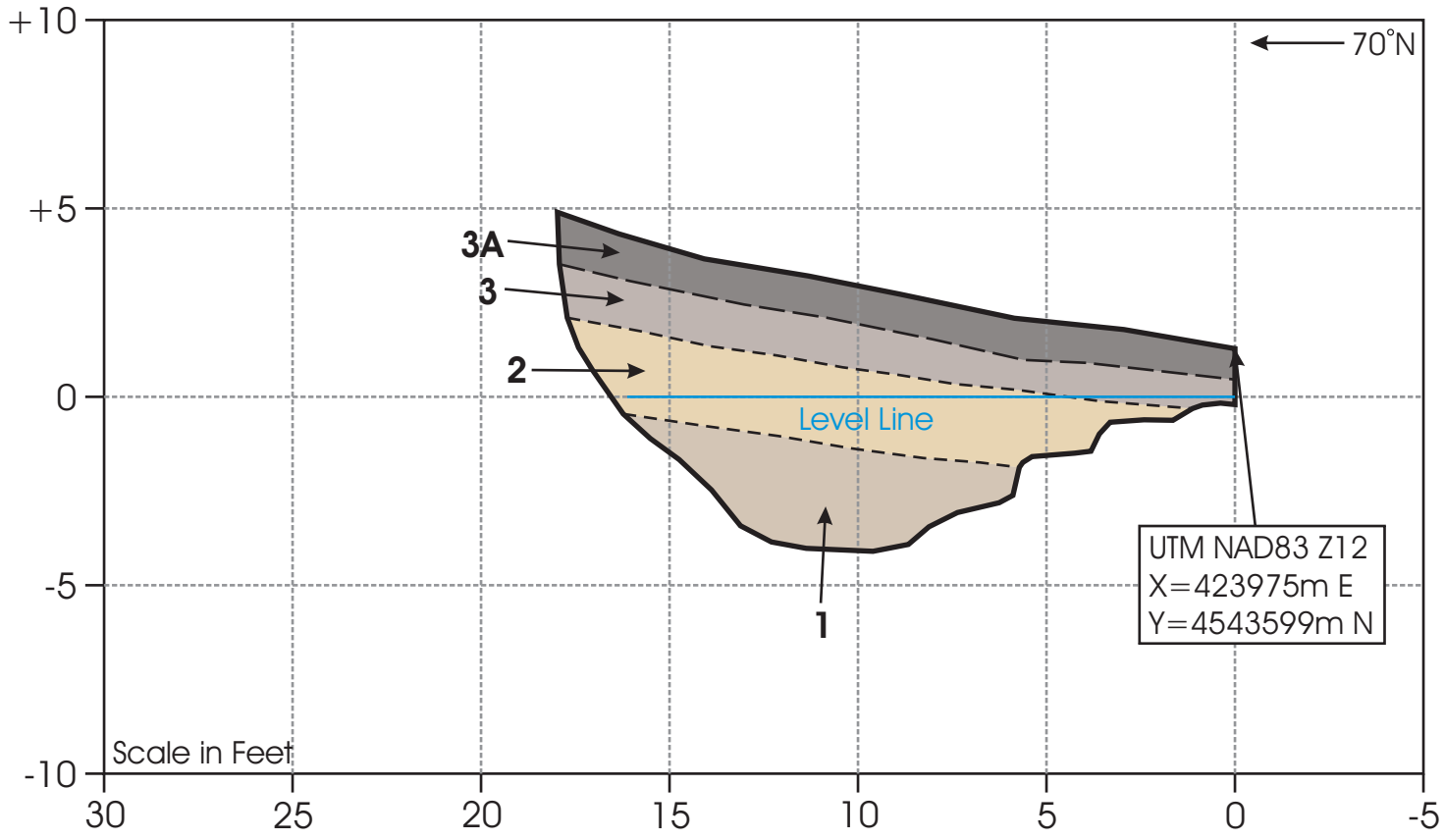
Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleistocene Lake Bonneville* - Sequence of well-bedded, pale-pinkish- to pale-golden-brown, silt to sand comprised of a lower (**1a**), moderate to high density, silt (ML) with fine sand lamina; and an upper (**1b**), moderate density, interbedded silt (ML) and fine sand sand (SM) with carbonate.

1bA. Modern A-horizon soil formed in unit 1a.

1aB. B soil horizon formed in unit 1b.

Logged by Bill D. Black, P.G. on December 17, 2018.
South wall logged, west to east. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

Rock Loft Ridge Subdivision About 250 North 1450 East, Fruit Heights, Utah	CMT ENGINEERING LABORATORIES **Test Pit 15 Log**	Date: 24-Oct-2020 Job # 12188	Figure 50



UNIT DESCRIPTIONS

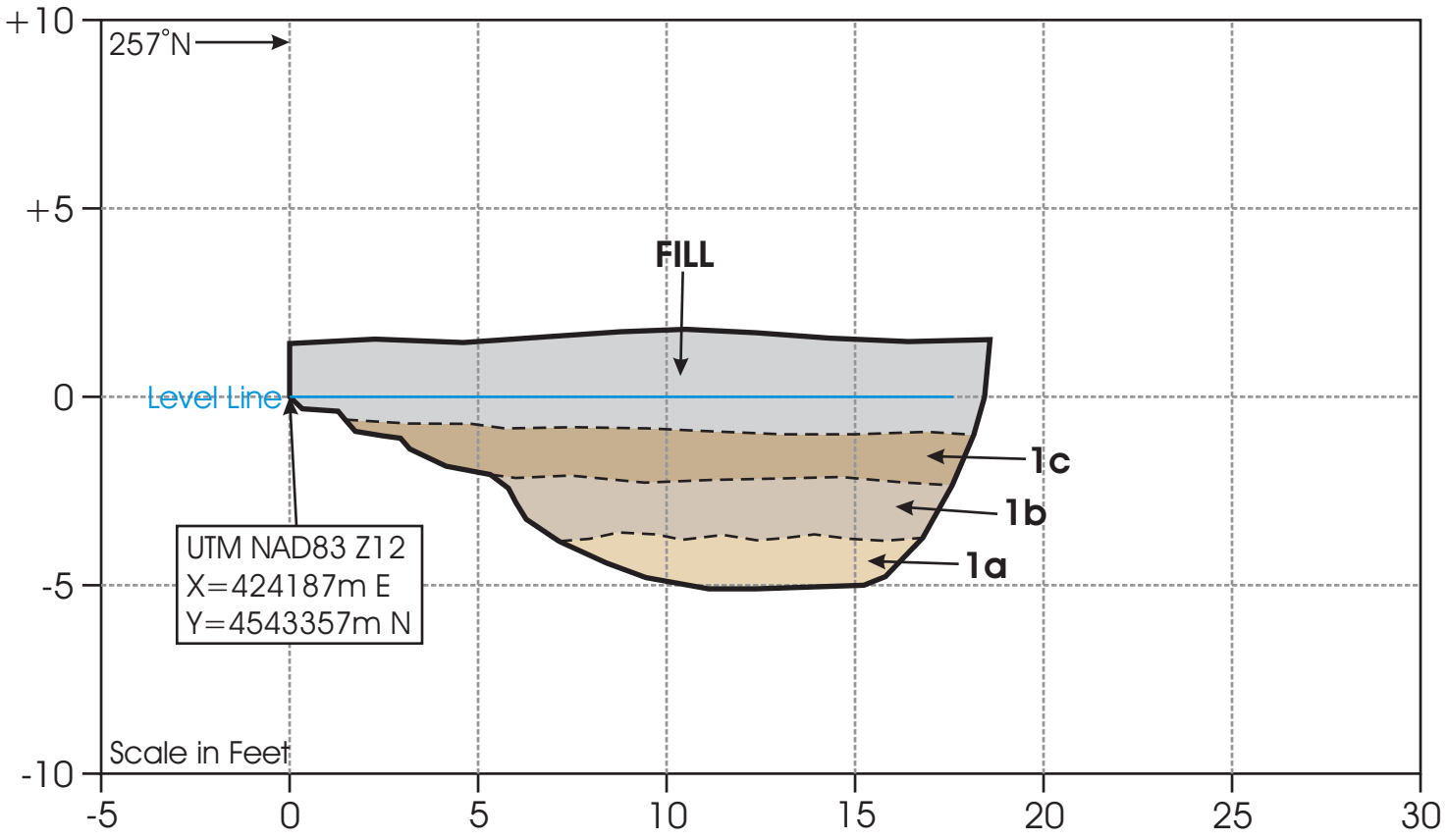
Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleitsocene Lake Bonneville* - Brown, poorly bedded, moderate density, gravelly sand to sandy gravel (SW/GW) with silty interbeds; clasts round to subangular with stage II carbonate.

Unit 2. *Post-Lake Bonneville debris flood deposit* - Light-yellowish-brown, massive, moderate density, sandy gravel to gravelly sand (GW/SW); mean thickness 2.1 feet.

Unit 3. *Post-Lake Bonneville debris flow deposit* - Dark-grayish-brown, massive, moderate density, gravelly sand (SW) with silt and cobbles; clasts subround to subangular with stage II carbonate; slight pinhole texture; largest clast 9" across; modern A horizon (**unit 3A**) formed in unit at surface; mean thickness 2.2 feet.

Logged by Bill D. Black, P.G. on September 2, 2020.
South wall logged, west to east. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

Rock Loft Ridge Subdivision About 250 North 1450 East, Fruit Heights, Utah	CMT ENGINEERING LABORATORIES **Test Pit 16 Log**	Date: 24-Oct-2020 Job # 12188	Figure 5P

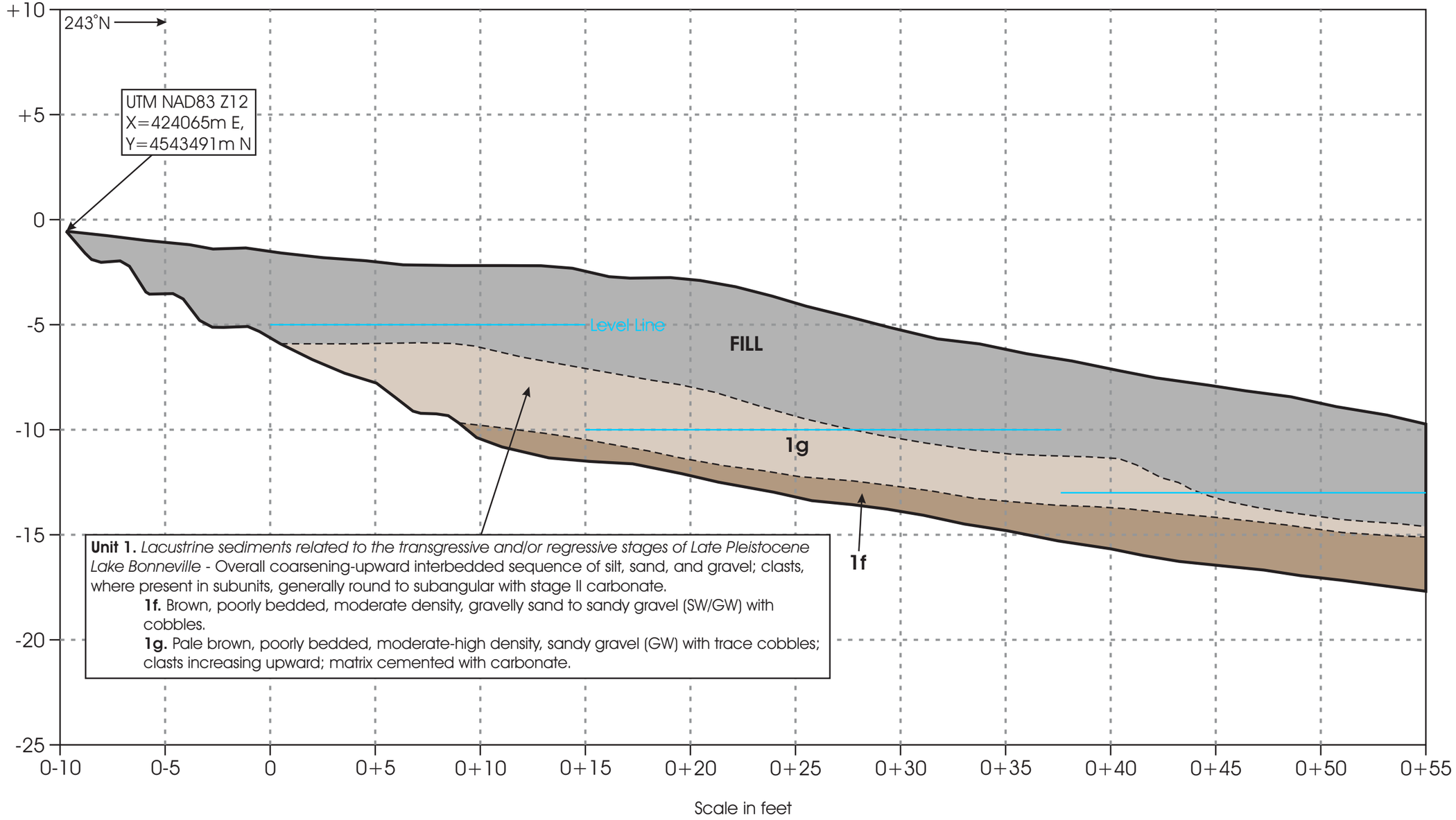


UNIT DESCRIPTIONS

Unit 1. *Post-Lake Bonneville debris flow and flood deposits* - Sequence comprised of a lower (**1a**) dark-yellowish-brown, low-moderate density, massive, gravelly sand (SW), boulders in trench floor; a middle (**1b**) moderate density, massive, sandy gravel to gravelly sand (GW/SW) with subround to subangular cobbles up to 6" across with stage II carbonate; and an upper (**1c**) dark-yellowish-brown, dense, silty sand (SM) with gravel; overlain by fill from gravel mining. Mean thicknesses: (**1a**) >1.2 feet, (**1b**) 1.6 feet, and (**1c**) 1.3 feet.

Logged by Bill D. Black, P.G. on September 2, 2020.
South wall logged, west to east. Scale 1 inch equals
5 feet (1:60) with no vertical exaggeration.

Rock Loft Ridge Subdivision About 250 North 1450 East, Fruit Heights, Utah	CMT ENGINEERING LABORATORIES **Test Pit 17 Log**	Date: 24-Oct-2020 Job # 12188	Figure 5Q



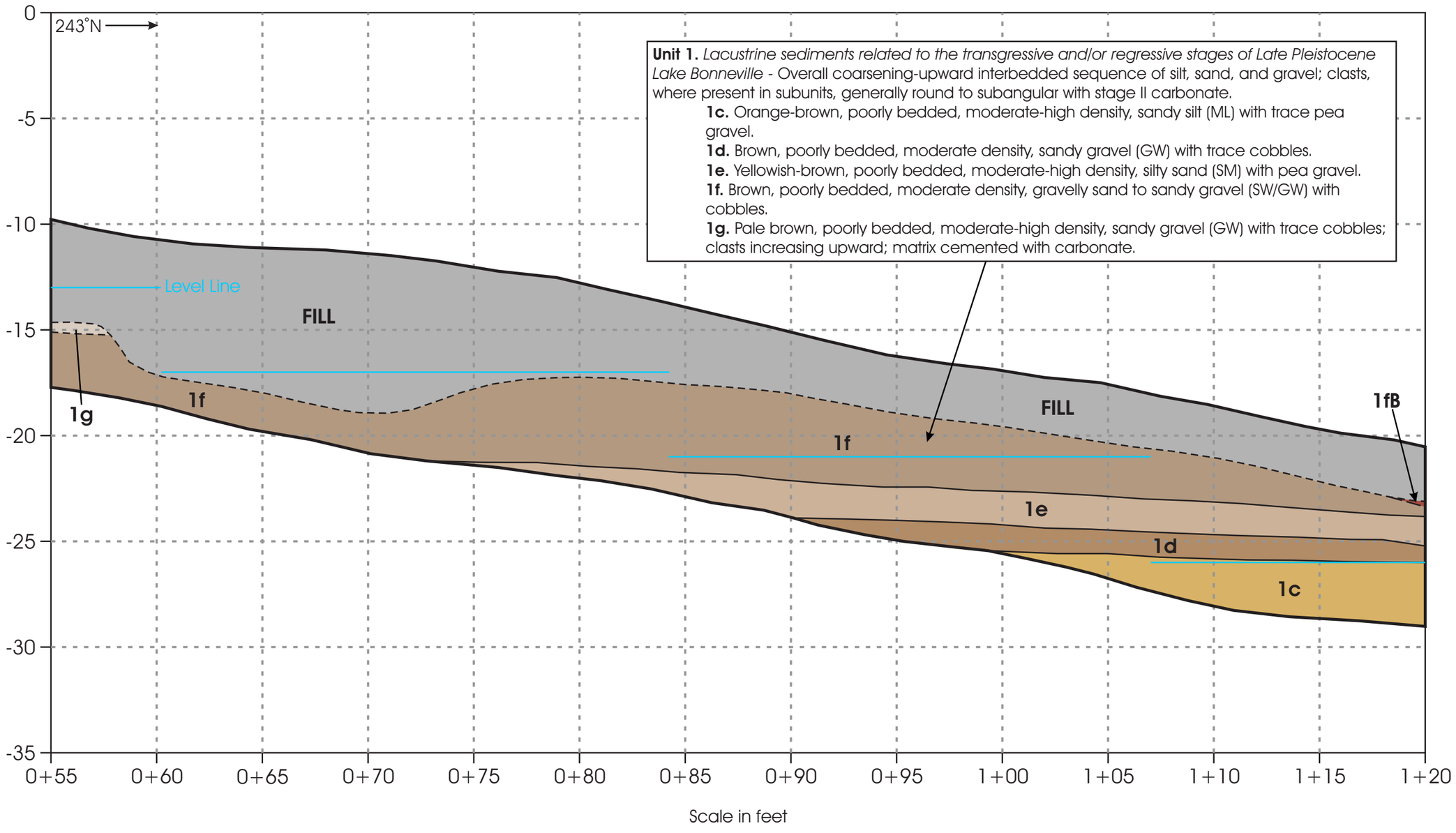
Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleistocene Lake Bonneville* - Overall coarsening-upward interbedded sequence of silt, sand, and gravel; clasts, where present in subunits, generally round to subangular with stage II carbonate.

1f. Brown, poorly bedded, moderate density, gravelly sand to sandy gravel (SW/GW) with cobbles.

1g. Pale brown, poorly bedded, moderate-high density, sandy gravel (GW) with trace cobbles; clasts increasing upward; matrix cemented with carbonate.

Logged by Bill D. Black, P.G. on December 20, 2018.
South wall logged, east to west. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

CMT ENGINEERING LABORATORIES	**Rock Loft Ridge Subdivision** About 250 North 1450 East, Fruit Heights, Utah	**Trench 1 Log, Sheet 1**	Date: 24-Oct-2020 CMT No.: 12188	Figure 6A



Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleistocene Lake Bonneville* - Overall coarsening-upward interbedded sequence of silt, sand, and gravel; clasts, where present in subunits, generally round to subangular with stage II carbonate.

1c. Orange-brown, poorly bedded, moderate-high density, sandy silt (ML) with trace pea gravel.

1d. Brown, poorly bedded, moderate density, sandy gravel (GW) with trace cobbles.

1e. Yellowish-brown, poorly bedded, moderate-high density, silty sand (SM) with pea gravel.

1f. Brown, poorly bedded, moderate density, gravelly sand to sandy gravel (SW/GW) with cobbles.

1g. Pale brown, poorly bedded, moderate-high density, sandy gravel (GW) with trace cobbles; clasts increasing upward; matrix cemented with carbonate.

Logged by Bill D. Black, P.G. on December 20, 2018.
South wall logged, east to west. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

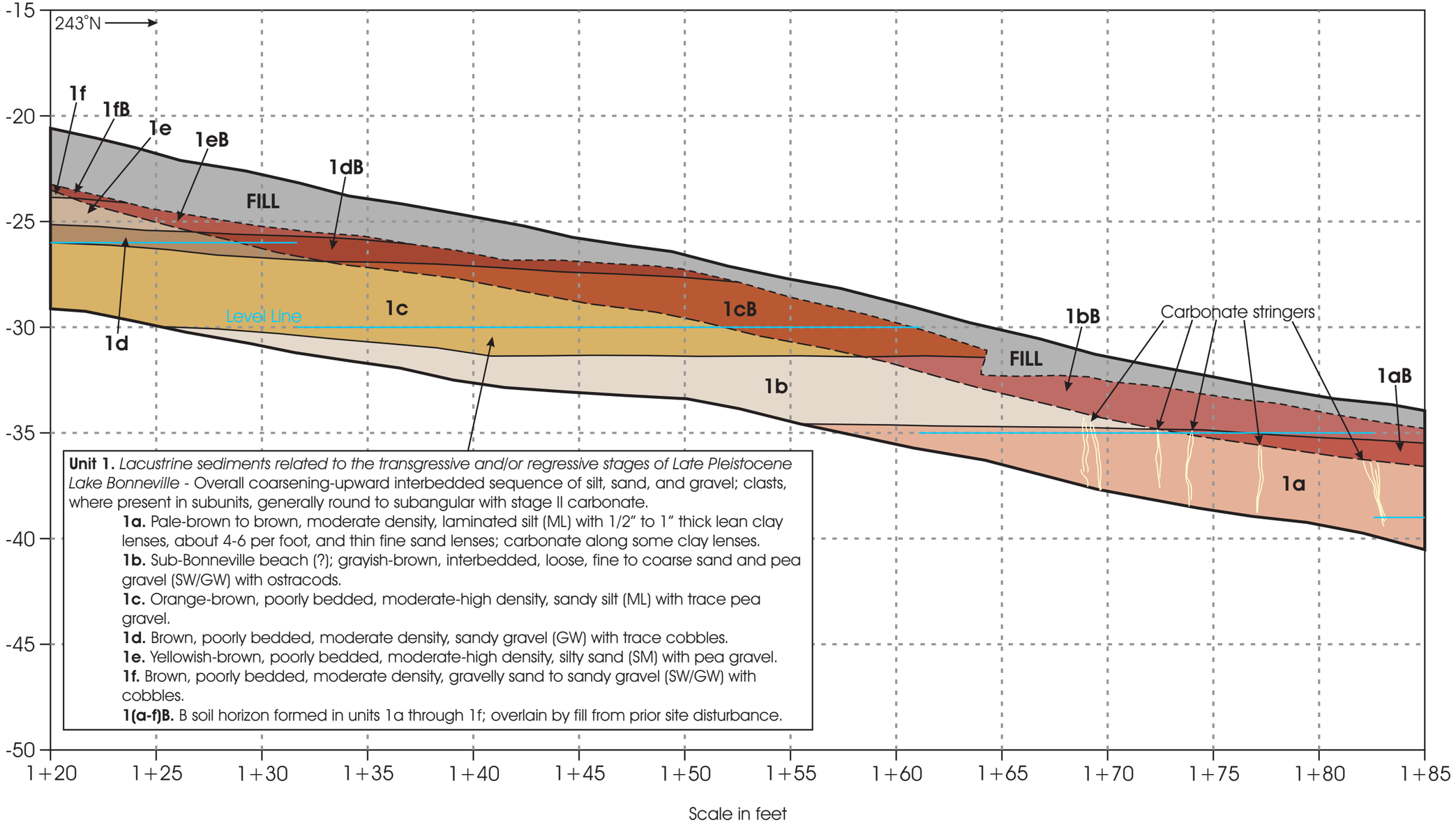
CMT ENGINEERING LABORATORIES

Rock Loft Ridge Subdivision
About 250 North 1450 East, Fruit Heights, Utah

Trench 1 Log, Sheet 2

Date:	24-Oct-2020
CMT No.:	12188

Figure **6B**



Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleistocene Lake Bonneville* - Overall coarsening-upward interbedded sequence of silt, sand, and gravel; clasts, where present in subunits, generally round to subangular with stage II carbonate.

1a. Pale-brown to brown, moderate density, laminated silt (ML) with 1/2" to 1" thick lean clay lenses, about 4-6 per foot, and thin fine sand lenses; carbonate along some clay lenses.

1b. Sub-Bonneville beach (?); grayish-brown, interbedded, loose, fine to coarse sand and pea gravel (SW/GW) with ostracods.

1c. Orange-brown, poorly bedded, moderate-high density, sandy silt (ML) with trace pea gravel.

1d. Brown, poorly bedded, moderate density, sandy gravel (GW) with trace cobbles.

1e. Yellowish-brown, poorly bedded, moderate-high density, silty sand (SM) with pea gravel.

1f. Brown, poorly bedded, moderate density, gravelly sand to sandy gravel (SW/GW) with cobbles.

1(a-f)B. B soil horizon formed in units 1a through 1f; overlain by fill from prior site disturbance.

Logged by Bill D. Black, P.G. on December 20, 2018.
South wall logged, east to west. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

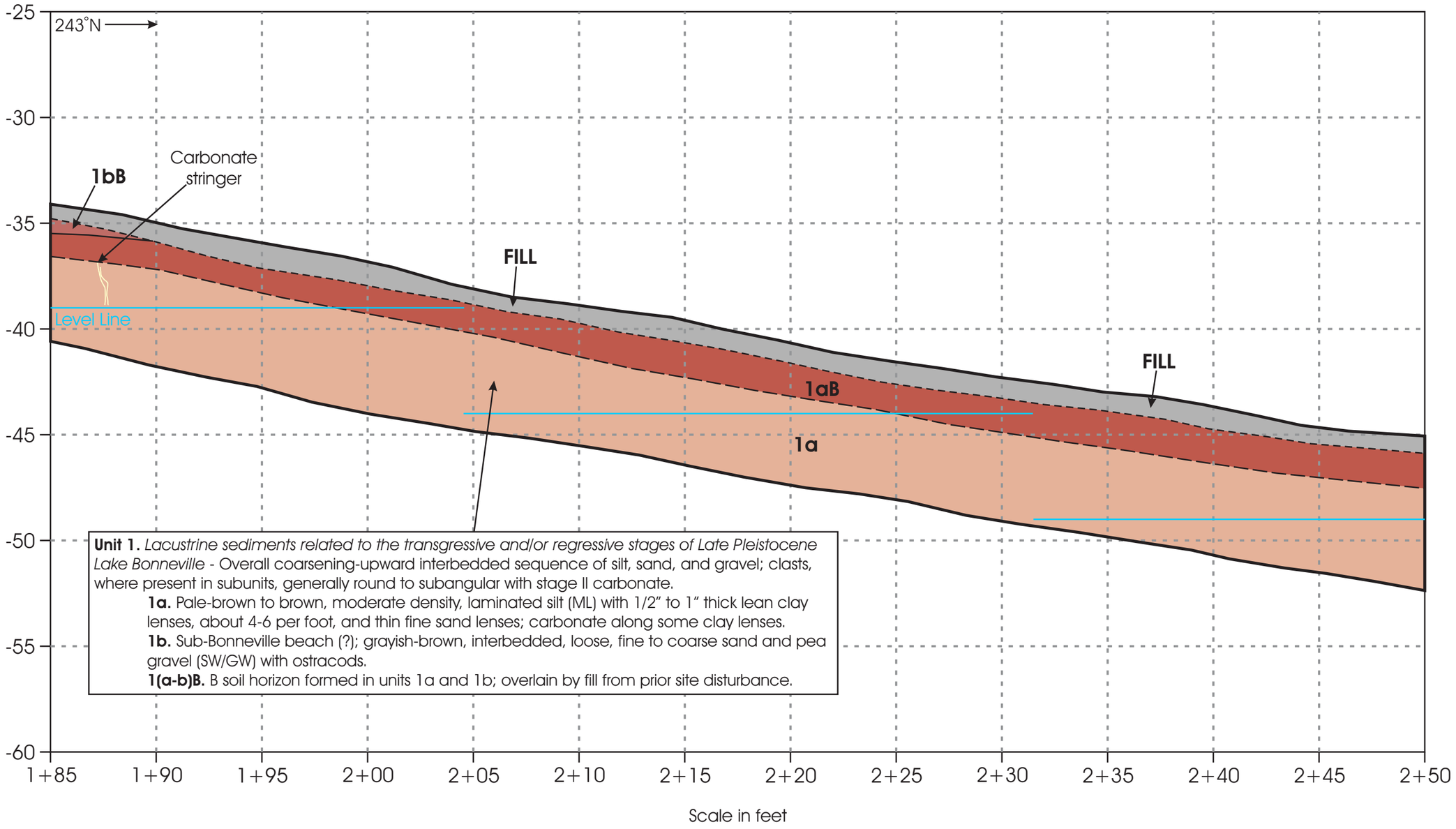
CMT ENGINEERING LABORATORIES

Rock Loft Ridge Subdivision
About 250 North 1450 East, Fruit Heights, Utah

Trench 1 Log, Sheet 3

Date:	24-Oct-2020
CMT No.:	12188

Figure 6C



Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleistocene Lake Bonneville* - Overall coarsening-upward interbedded sequence of silt, sand, and gravel; clasts, where present in subunits, generally round to subangular with stage II carbonate.

1a. Pale-brown to brown, moderate density, laminated silt (ML) with 1/2" to 1" thick lean clay lenses, about 4-6 per foot, and thin fine sand lenses; carbonate along some clay lenses.

1b. Sub-Bonneville beach (?); grayish-brown, interbedded, loose, fine to coarse sand and pea gravel (SW/GW) with ostracods.

1(a-b)B. B soil horizon formed in units 1a and 1b; overlain by fill from prior site disturbance.

Logged by Bill D. Black, P.G. on December 20, 2018.
South wall logged, east to west. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

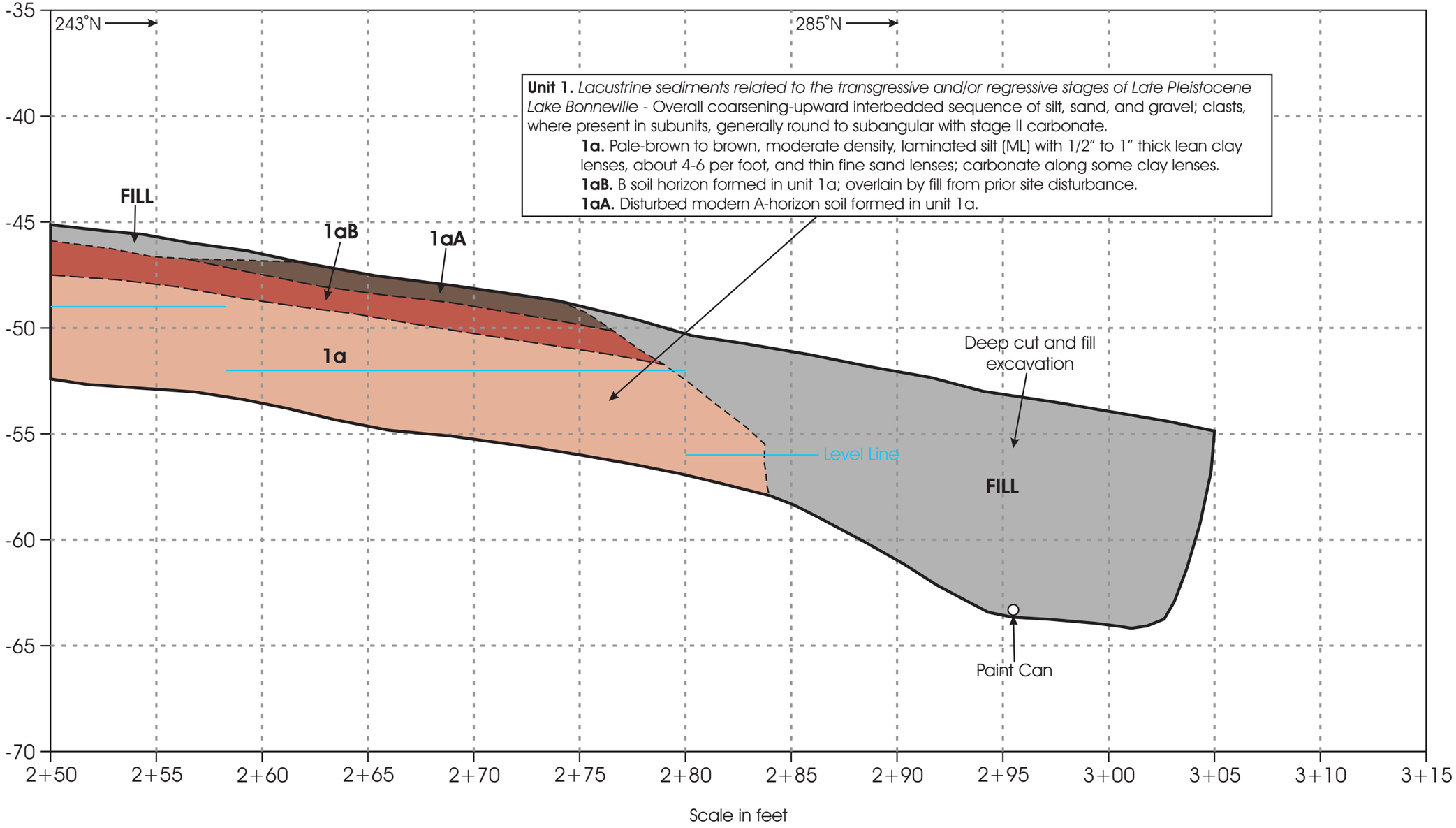
CMT ENGINEERING LABORATORIES

Rock Loft Ridge Subdivision
About 250 North 1450 East, Fruit Heights, Utah

Trench 1 Log, Sheet 4

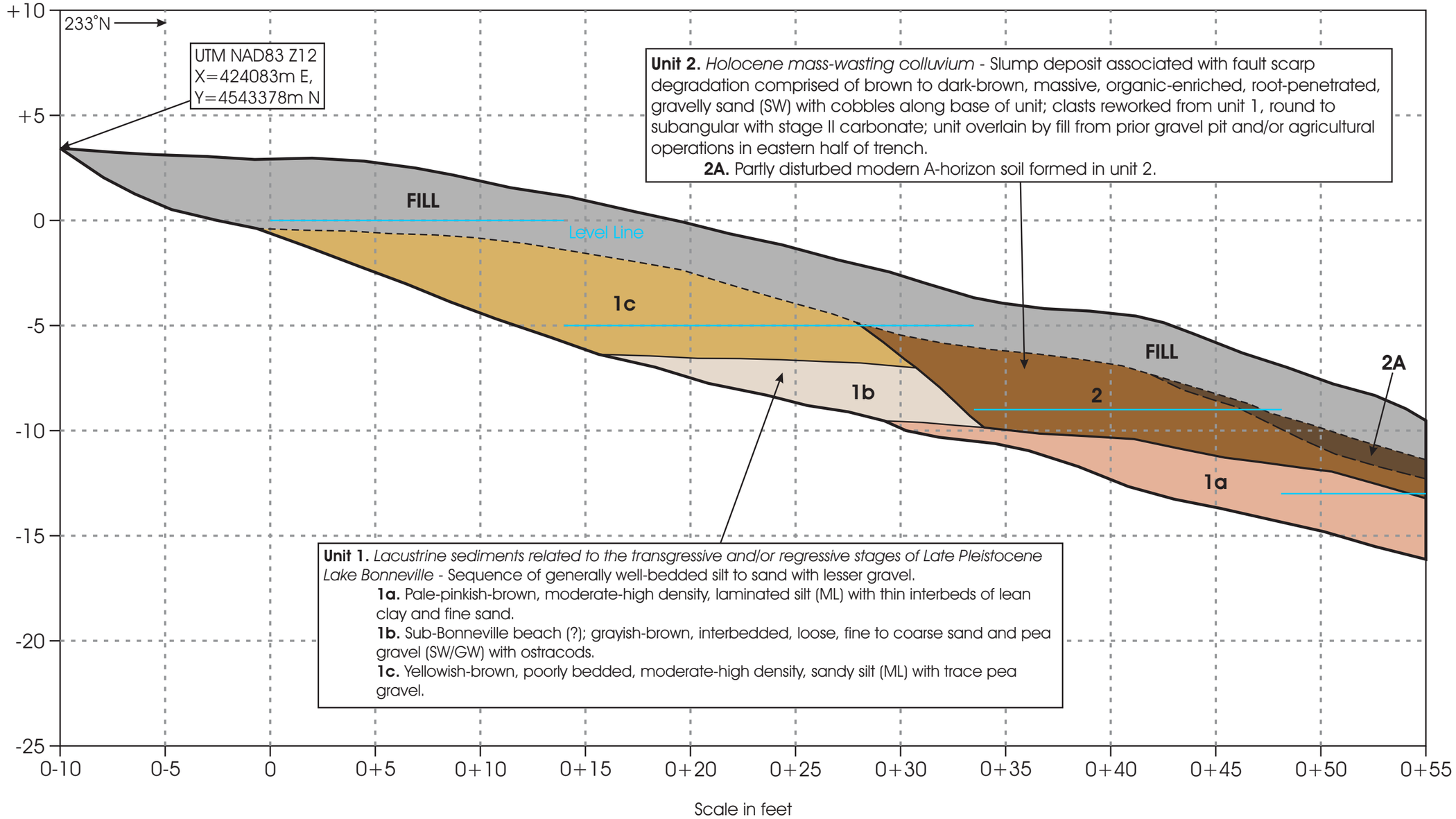
Date:	24-Oct-2020
CMT No.:	12188

Figure **6D**



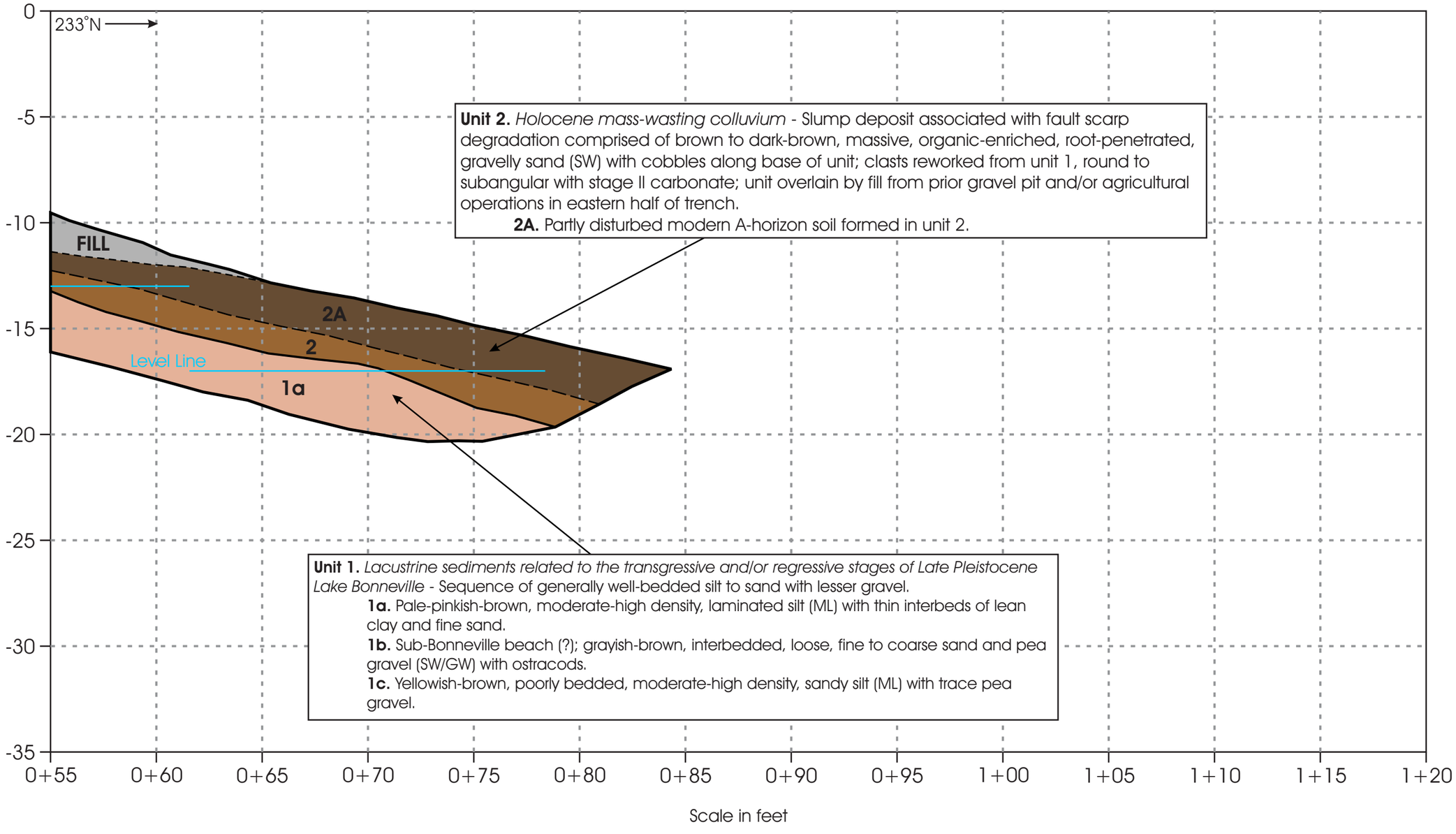
Logged by Bill D. Black, P.G. on December 20, 2018.
South wall logged, east to west. Scale 1 inch equals
5 feet (1:60) with no vertical exaggeration.

CMT ENGINEERING LABORATORIES	**Rock Loft Ridge Subdivision** About 250 North 1450 East, Fruit Heights, Utah	**Trench 1 Log, sheet 5**	Date: 24-Oct-2020 CMT No.: 12188	Figure **6E**



Logged by Bill D. Black, P.G. on December 19, 2018.
South wall logged, east to west. Scale 1 inch equals
5 feet (1:60) with no vertical exaggeration.

CMT ENGINEERING LABORATORIES	**Rock Loft Ridge Subdivision** About 250 North 1450 East, Fruit Heights, Utah	**Trench 2 Log, Sheet 1**	Date: 24-Oct-2020 CMT No.: 12188	Figure **7A**



Unit 2. *Holocene mass-wasting colluvium* - Slump deposit associated with fault scarp degradation comprised of brown to dark-brown, massive, organic-enriched, root-penetrated, gravelly sand (SW) with cobbles along base of unit; clasts reworked from unit 1, round to subangular with stage II carbonate; unit overlain by fill from prior gravel pit and/or agricultural operations in eastern half of trench.

2A. Partly disturbed modern A-horizon soil formed in unit 2.

Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleistocene Lake Bonneville* - Sequence of generally well-bedded silt to sand with lesser gravel.

1a. Pale-pinkish-brown, moderate-high density, laminated silt (ML) with thin interbeds of lean clay and fine sand.

1b. Sub-Bonneville beach (?); grayish-brown, interbedded, loose, fine to coarse sand and pea gravel (SW/GW) with ostracods.

1c. Yellowish-brown, poorly bedded, moderate-high density, sandy silt (ML) with trace pea gravel.

Logged by Bill D. Black, P.G. on December 19, 2018.
South wall logged, east to west. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

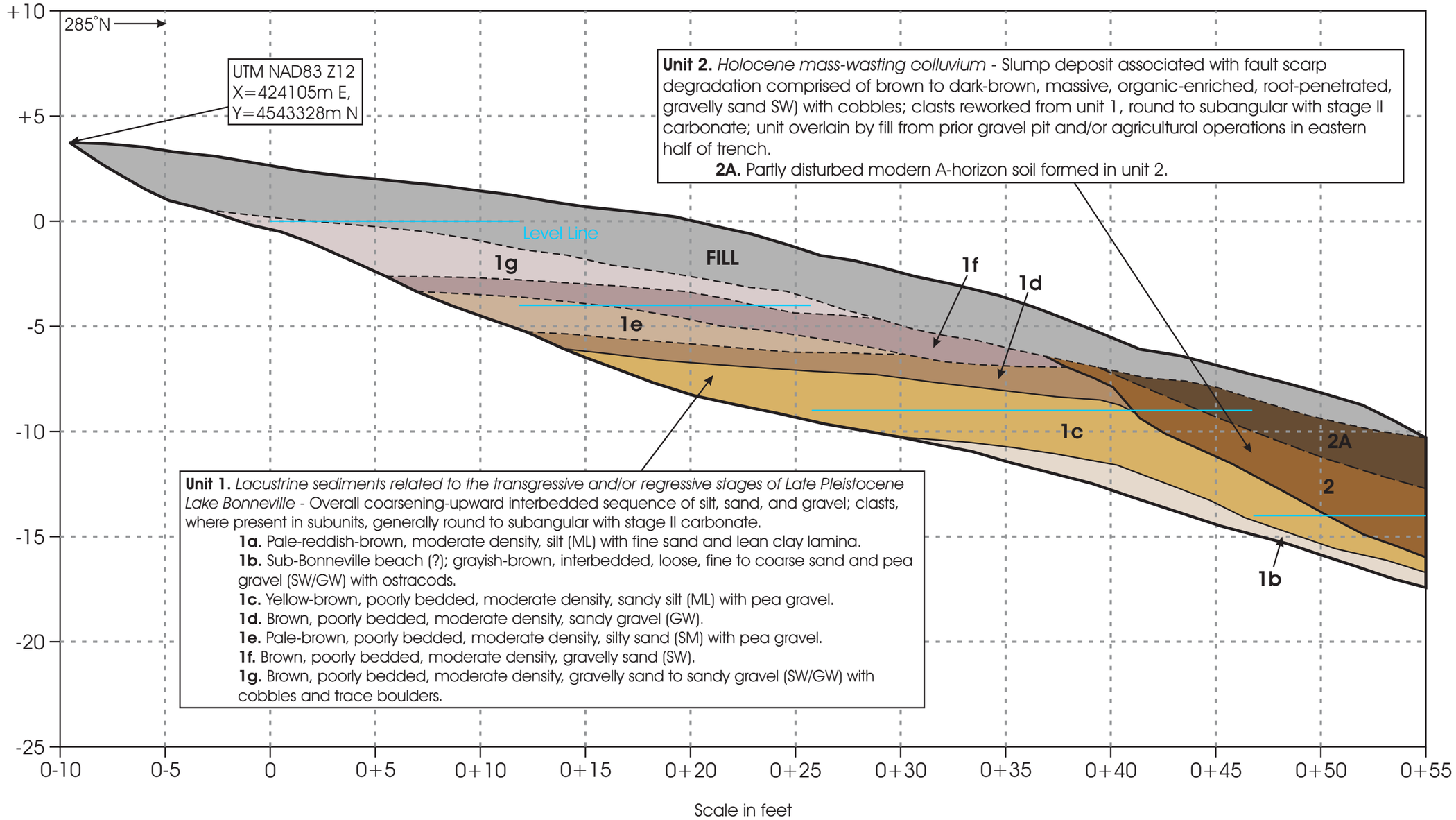
CMT ENGINEERING LABORATORIES

Rock Loft Ridge Subdivision
About 250 North 1450 East, Fruit Heights, Utah

Trench 2 Log, Sheet 2

Date:	24-Oct-2020
CMT No.:	12188

Figure **7B**



Unit 2. *Holocene mass-wasting colluvium* - Slump deposit associated with fault scarp degradation comprised of brown to dark-brown, massive, organic-enriched, root-penetrated, gravelly sand SW) with cobbles; clasts reworked from unit 1, round to subangular with stage II carbonate; unit overlain by fill from prior gravel pit and/or agricultural operations in eastern half of trench.

2A. Partly disturbed modern A-horizon soil formed in unit 2.

Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleistocene Lake Bonneville* - Overall coarsening-upward interbedded sequence of silt, sand, and gravel; clasts, where present in subunits, generally round to subangular with stage II carbonate.

1a. Pale-reddish-brown, moderate density, silt (ML) with fine sand and lean clay lamina.

1b. Sub-Bonneville beach (?); grayish-brown, interbedded, loose, fine to coarse sand and pea gravel (SW/GW) with ostracods.

1c. Yellow-brown, poorly bedded, moderate density, sandy silt (ML) with pea gravel.

1d. Brown, poorly bedded, moderate density, sandy gravel (GW).

1e. Pale-brown, poorly bedded, moderate density, silty sand (SM) with pea gravel.

1f. Brown, poorly bedded, moderate density, gravelly sand (SW).

1g. Brown, poorly bedded, moderate density, gravelly sand to sandy gravel (SW/GW) with cobbles and trace boulders.

Logged by Bill D. Black, P.G. on December 19, 2018.
South wall logged, east to west. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

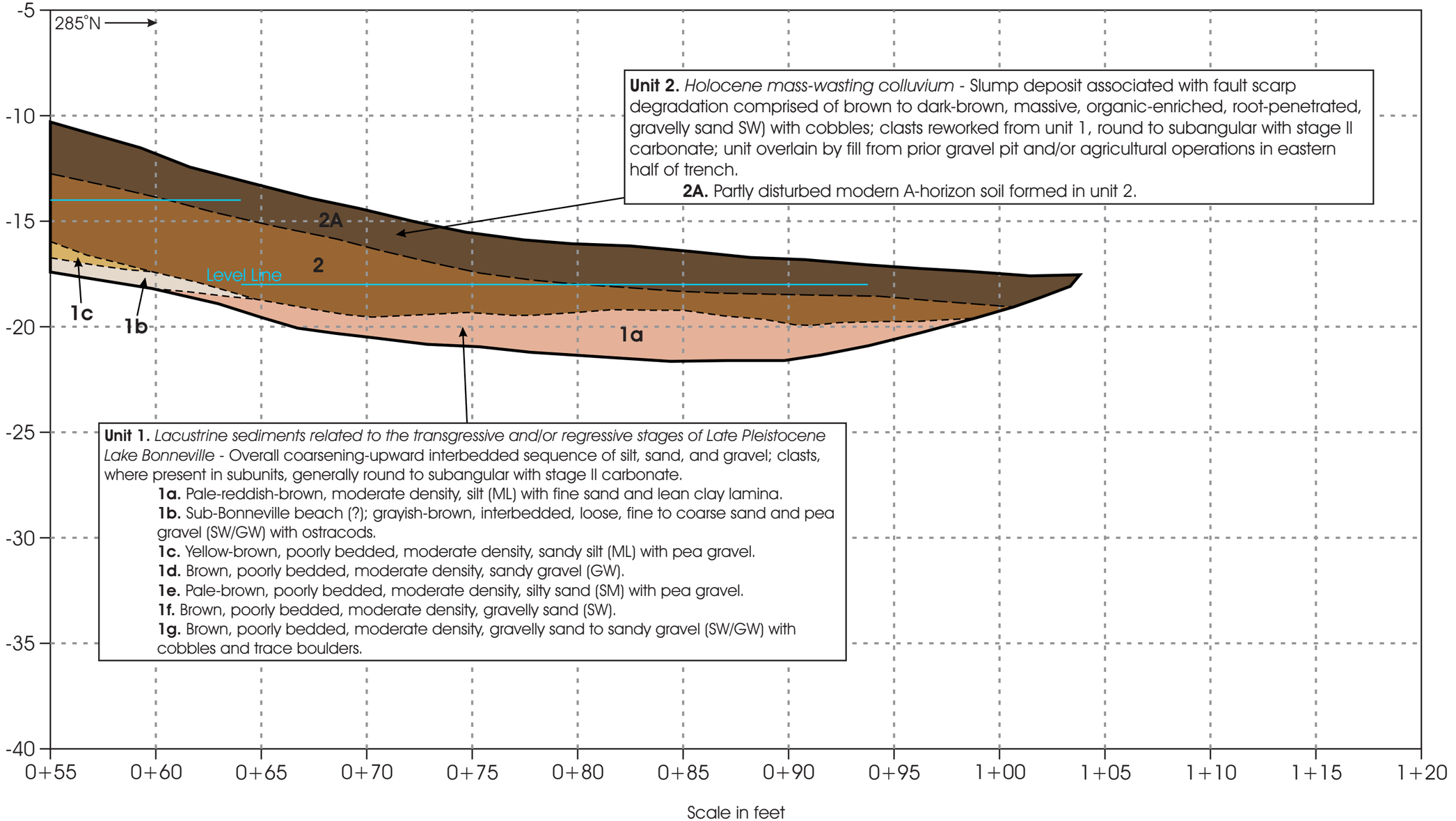
CMT ENGINEERING LABORATORIES

Rock Loft Ridge Subdivision
About 250 North 1450 East, Fruit Heights, Utah

Trench 3 Log, Sheet 1

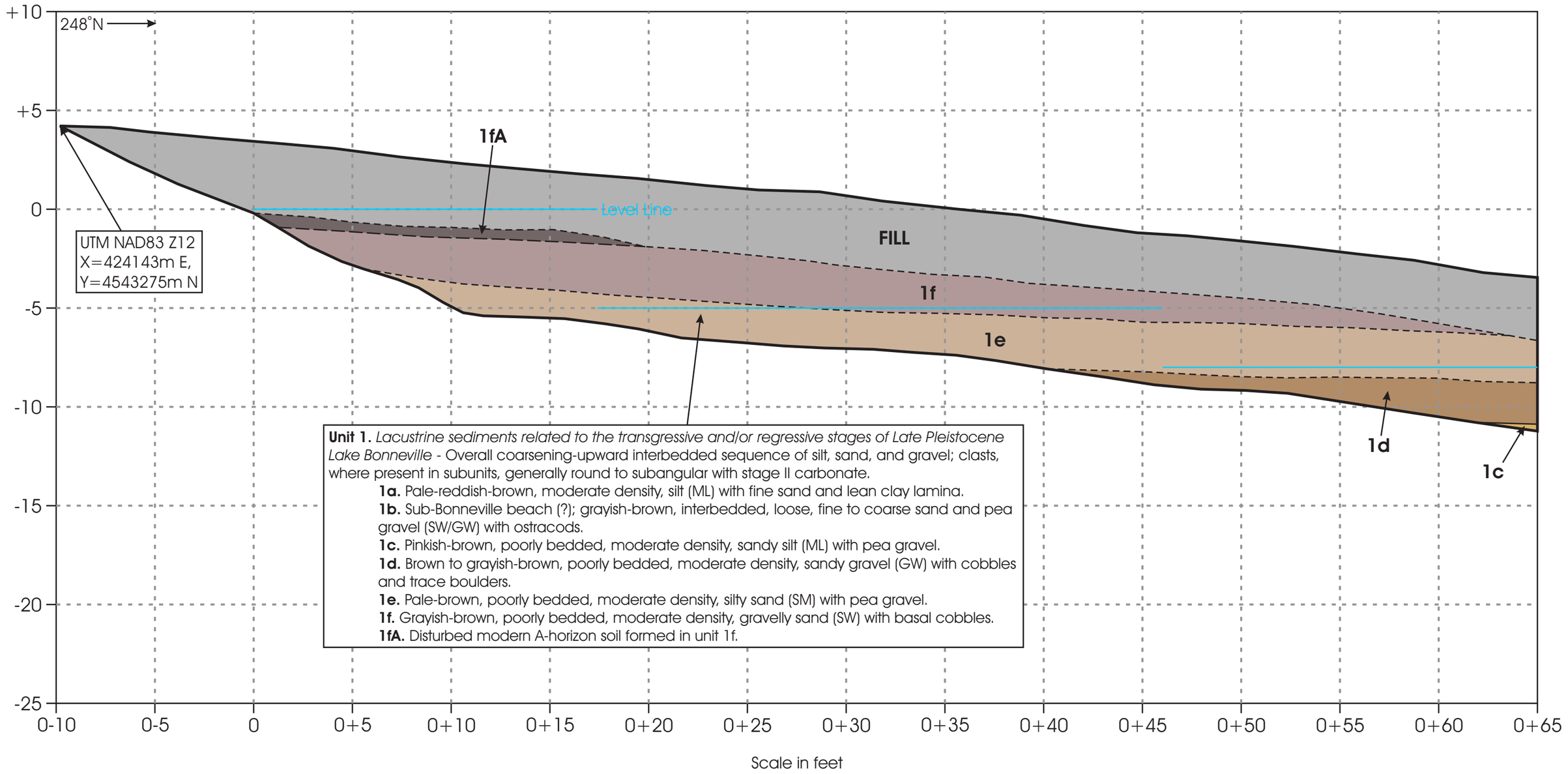
Date:	24-Oct-2020
CMT No.:	12188

Figure **8A**



Logged by Bill D. Black, P.G. on December 19, 2018.
South wall logged, east to west. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

CMT ENGINEERING LABORATORIES	**Rock Loft Ridge Subdivision** About 250 North 1450 East, Fruit Heights, Utah	**Trench 3 Log, Sheet 2**	Date: 24-Oct-2020 CMT No.: 12188	Figure **8B**



Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleistocene Lake Bonneville* - Overall coarsening-upward interbedded sequence of silt, sand, and gravel; clasts, where present in subunits, generally round to subangular with stage II carbonate.

1a. Pale-reddish-brown, moderate density, silt (ML) with fine sand and lean clay lamina.

1b. Sub-Bonneville beach (?); grayish-brown, interbedded, loose, fine to coarse sand and pea gravel (SW/GW) with ostracods.

1c. Pinkish-brown, poorly bedded, moderate density, sandy silt (ML) with pea gravel.

1d. Brown to grayish-brown, poorly bedded, moderate density, sandy gravel (GW) with cobbles and trace boulders.

1e. Pale-brown, poorly bedded, moderate density, silty sand (SM) with pea gravel.

1f. Grayish-brown, poorly bedded, moderate density, gravelly sand (SW) with basal cobbles.

1fA. Disturbed modern A-horizon soil formed in unit 1f.

Logged by Bill D. Black, P.G. on December 18, 2018.
South wall logged, east to west. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

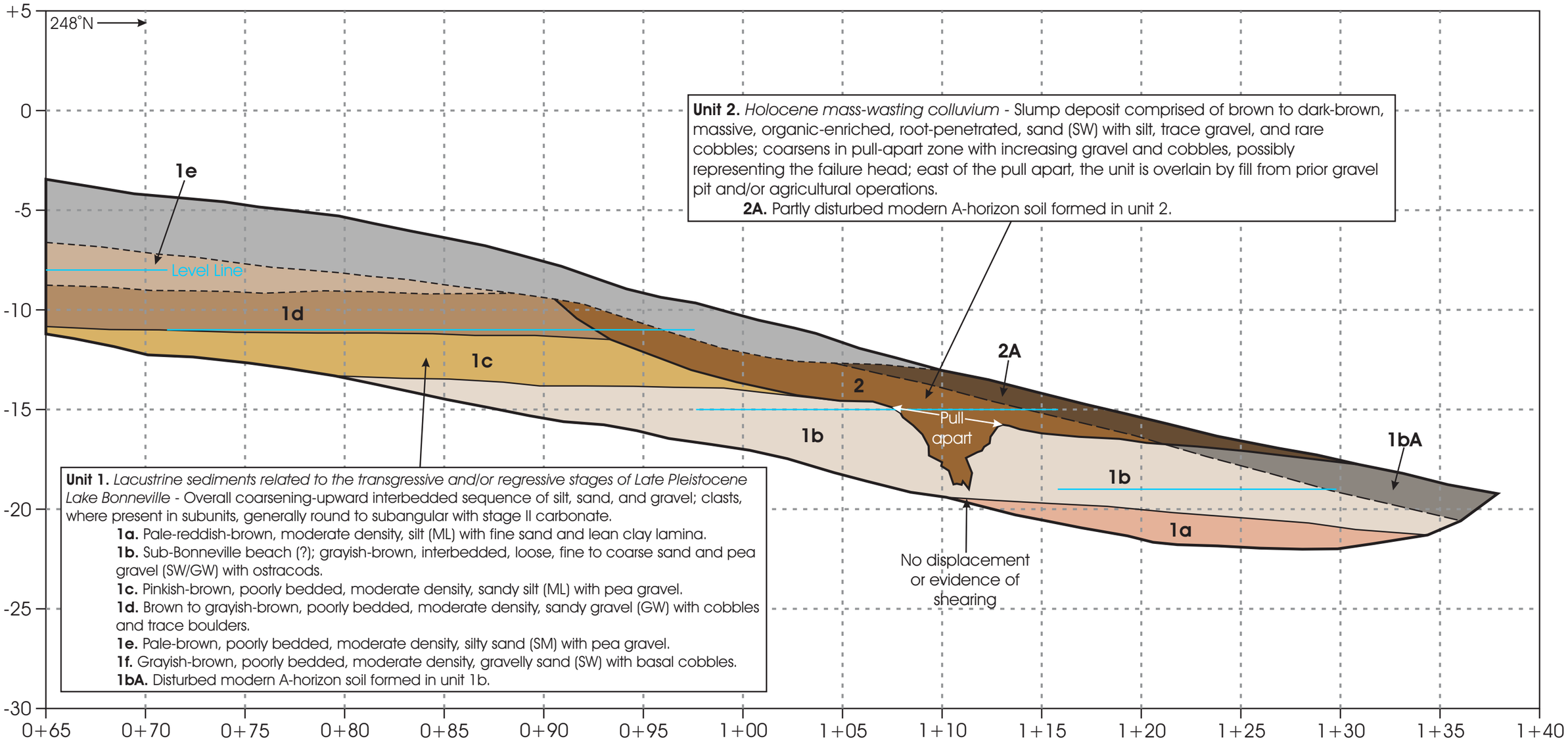
CMT ENGINEERING LABORATORIES

Rock Loft Ridge Subdivision
About 250 North 1450 East, Fruit Heights, Utah

Trench 4 Log, Sheet 1

Date:	24-Oct-2020
CMT No.:	12188

Figure **9A**



Unit 2. *Holocene mass-wasting colluvium* - Slump deposit comprised of brown to dark-brown, massive, organic-enriched, root-penetrated, sand (SW) with silt, trace gravel, and rare cobbles; coarsens in pull-apart zone with increasing gravel and cobbles, possibly representing the failure head; east of the pull apart, the unit is overlain by fill from prior gravel pit and/or agricultural operations.

2A. Partly disturbed modern A-horizon soil formed in unit 2.

Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleistocene Lake Bonneville* - Overall coarsening-upward interbedded sequence of silt, sand, and gravel; clasts, where present in subunits, generally round to subangular with stage II carbonate.

1a. Pale-reddish-brown, moderate density, silt (ML) with fine sand and lean clay lamina.

1b. Sub-Bonneville beach (?); grayish-brown, interbedded, loose, fine to coarse sand and pea gravel (SW/GW) with ostracods.

1c. Pinkish-brown, poorly bedded, moderate density, sandy silt (ML) with pea gravel.

1d. Brown to grayish-brown, poorly bedded, moderate density, sandy gravel (GW) with cobbles and trace boulders.

1e. Pale-brown, poorly bedded, moderate density, silty sand (SM) with pea gravel.

1f. Grayish-brown, poorly bedded, moderate density, gravelly sand (SW) with basal cobbles.

1bA. Disturbed modern A-horizon soil formed in unit 1b.

Logged by Bill D. Black, P.G. on December 18, 2018.
South wall logged, east to west. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

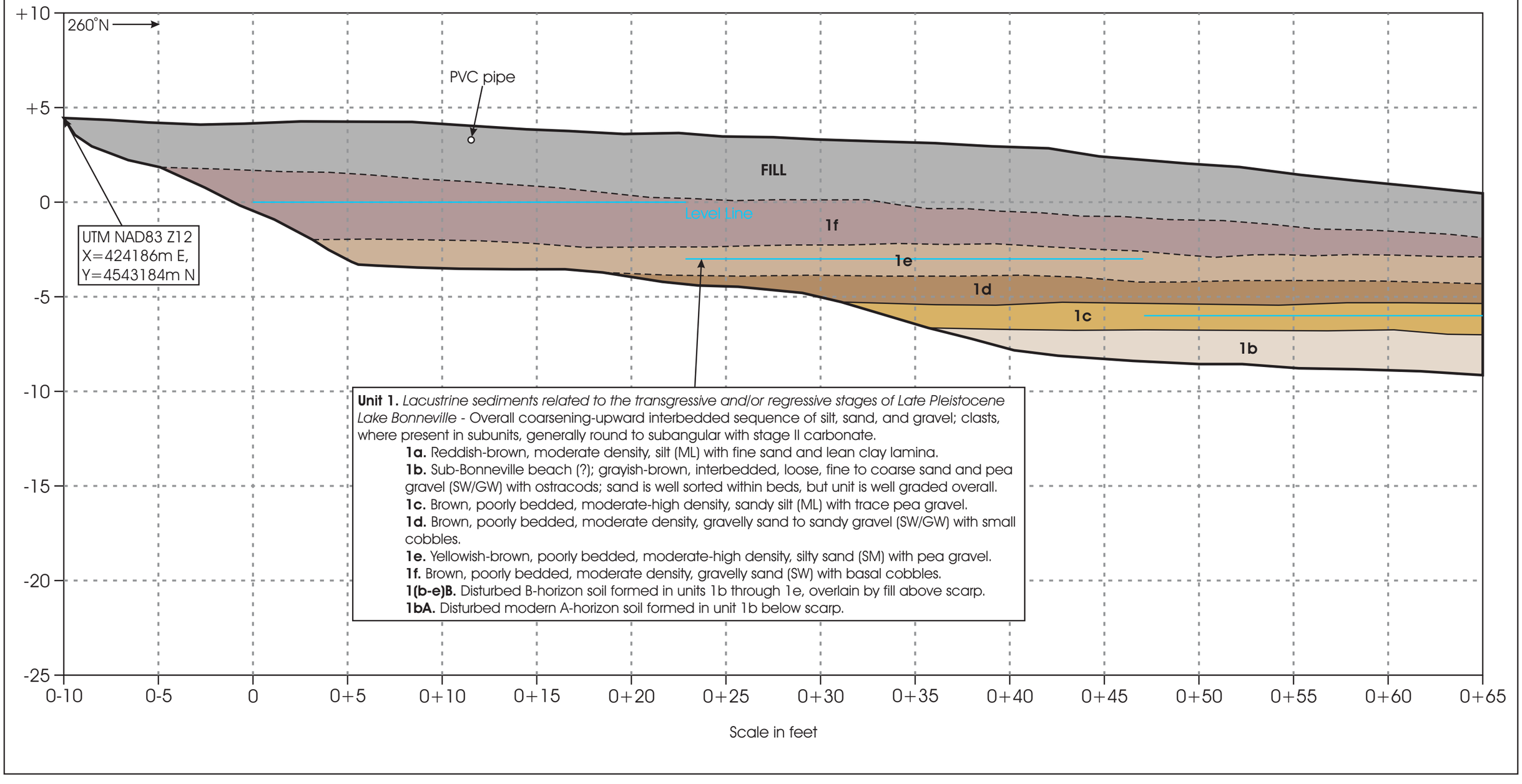
CMT ENGINEERING LABORATORIES

Rock Loft Ridge Subdivision
About 250 North 1450 East, Fruit Heights, Utah

Trench 4 Log, Sheet 2

Date:	24-Oct-2020
CMT No.:	12188

Figure 9B



Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleistocene Lake Bonneville* - Overall coarsening-upward interbedded sequence of silt, sand, and gravel; clasts, where present in subunits, generally round to subangular with stage II carbonate.

1a. Reddish-brown, moderate density, silt (ML) with fine sand and lean clay lamina.

1b. Sub-Bonneville beach (?); grayish-brown, interbedded, loose, fine to coarse sand and pea gravel (SW/GW) with ostracods; sand is well sorted within beds, but unit is well graded overall.

1c. Brown, poorly bedded, moderate-high density, sandy silt (ML) with trace pea gravel.

1d. Brown, poorly bedded, moderate density, gravelly sand to sandy gravel (SW/GW) with small cobbles.

1e. Yellowish-brown, poorly bedded, moderate-high density, silty sand (SM) with pea gravel.

1f. Brown, poorly bedded, moderate density, gravelly sand (SW) with basal cobbles.

1(b-e)B. Disturbed B-horizon soil formed in units 1b through 1e, overlain by fill above scarp.

1bA. Disturbed modern A-horizon soil formed in unit 1b below scarp.

Logged by Bill D. Black, P.G. on December 17, 2018.
South wall logged, east to west. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

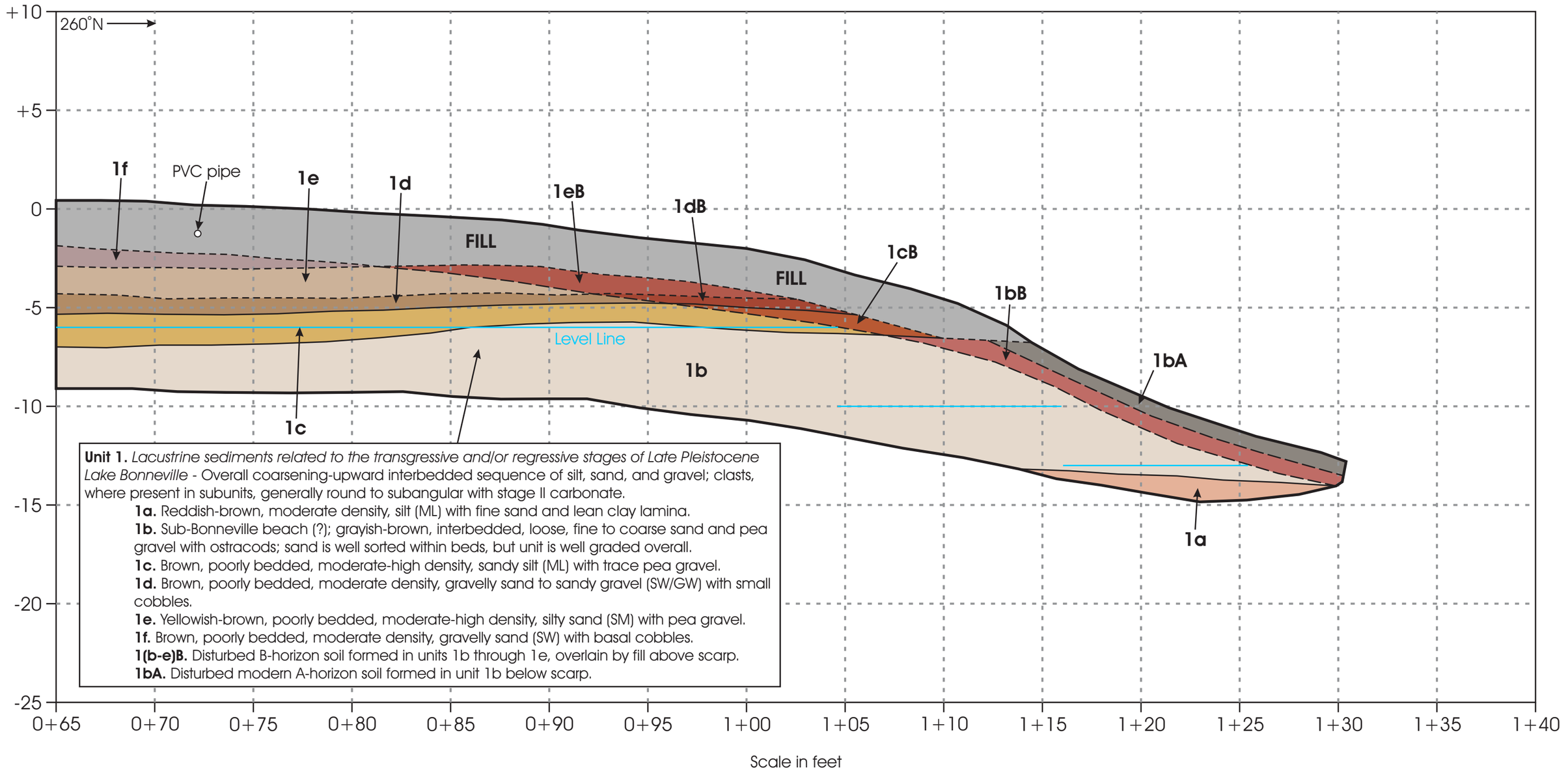
CMT ENGINEERING LABORATORIES

Rock Loft Ridge Subdivision
About 250 North 1450 East, Fruit Heights, Utah

Trench 5 Log, Sheet 1

Date:	24-Oct-2020
CMT No.:	12188

Figure 10A



Unit 1. *Lacustrine sediments related to the transgressive and/or regressive stages of Late Pleistocene Lake Bonneville* - Overall coarsening-upward interbedded sequence of silt, sand, and gravel; clasts, where present in subunits, generally round to subangular with stage II carbonate.

1a. Reddish-brown, moderate density, silt (ML) with fine sand and lean clay lamina.
1b. Sub-Bonneville beach (?); grayish-brown, interbedded, loose, fine to coarse sand and pea gravel with ostracods; sand is well sorted within beds, but unit is well graded overall.
1c. Brown, poorly bedded, moderate-high density, sandy silt (ML) with trace pea gravel.
1d. Brown, poorly bedded, moderate density, gravelly sand to sandy gravel (SW/GW) with small cobbles.
1e. Yellowish-brown, poorly bedded, moderate-high density, silty sand (SM) with pea gravel.
1f. Brown, poorly bedded, moderate density, gravelly sand (SW) with basal cobbles.
1(b-e)B. Disturbed B-horizon soil formed in units 1b through 1e, overlain by fill above scarp.
1bA. Disturbed modern A-horizon soil formed in unit 1b below scarp.

Logged by Bill D. Black, P.G. on December 17, 2018.
South wall logged, east to west. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

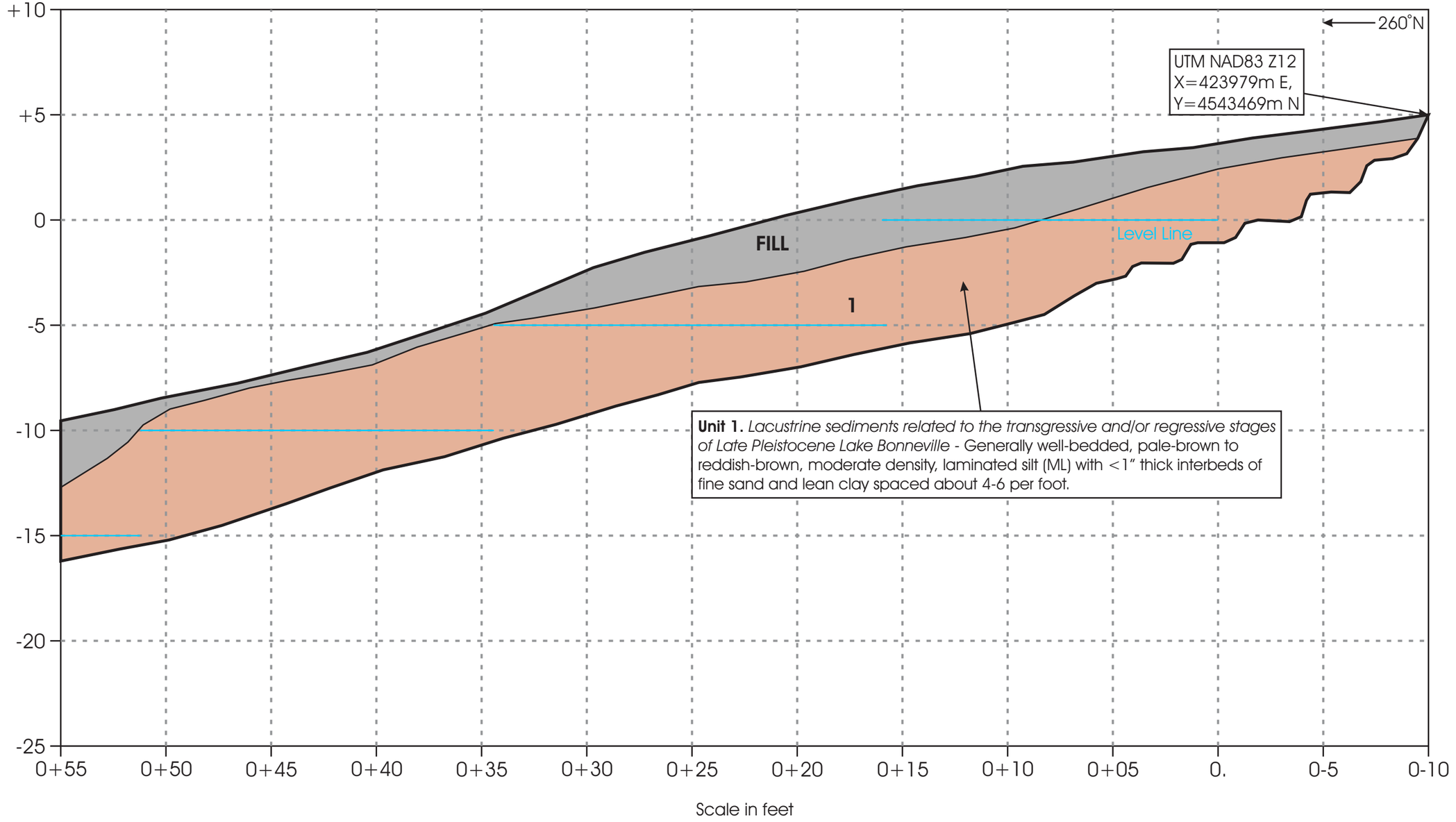
CMT ENGINEERING LABORATORIES

Rock Loft Ridge Subdivision
About 250 North 1450 East, Fruit Heights, Utah

Trench 5 Log, Sheet 2

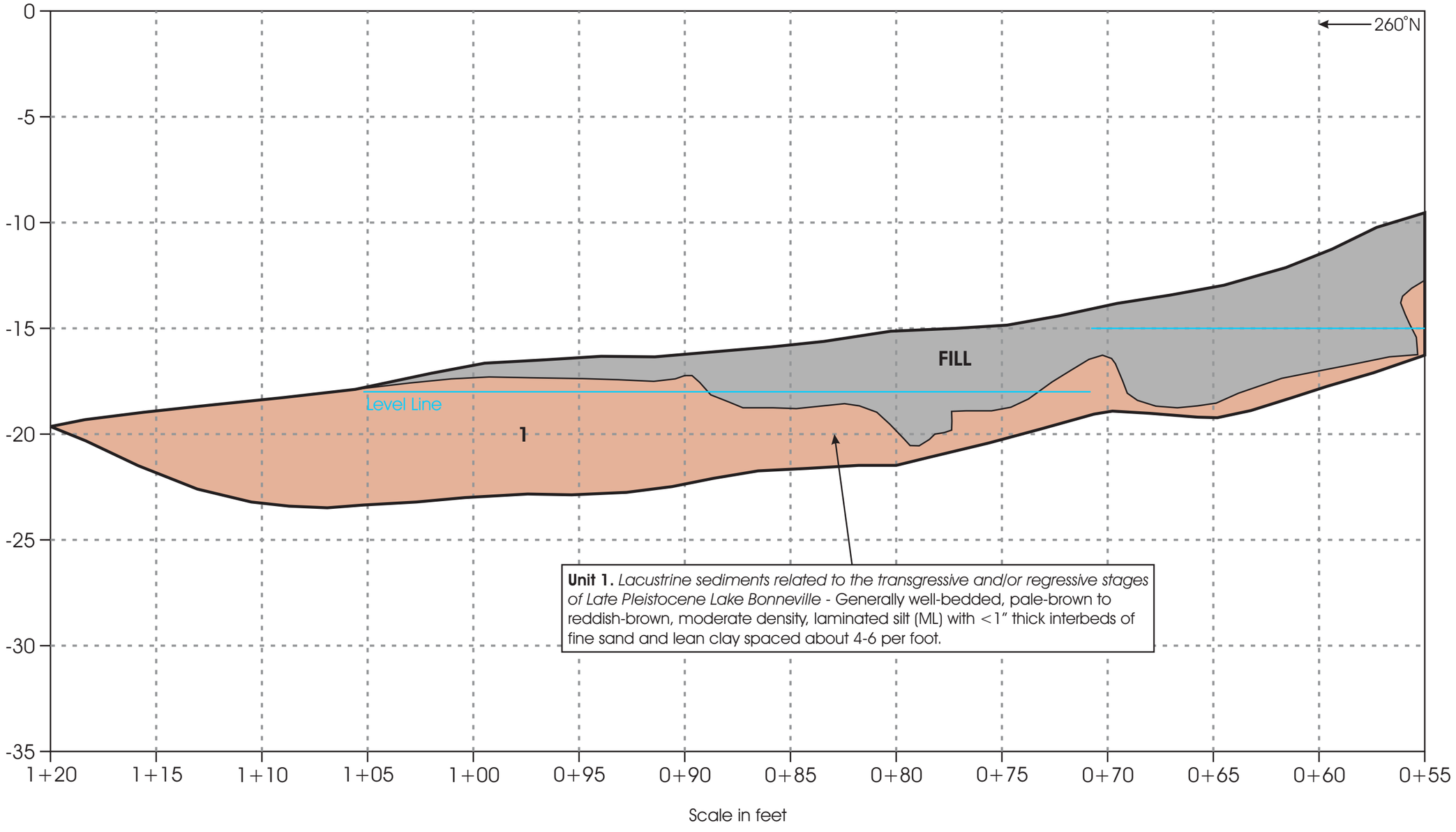
Date:	24-Oct-2020
CMT No.:	12188

Figure **10B**



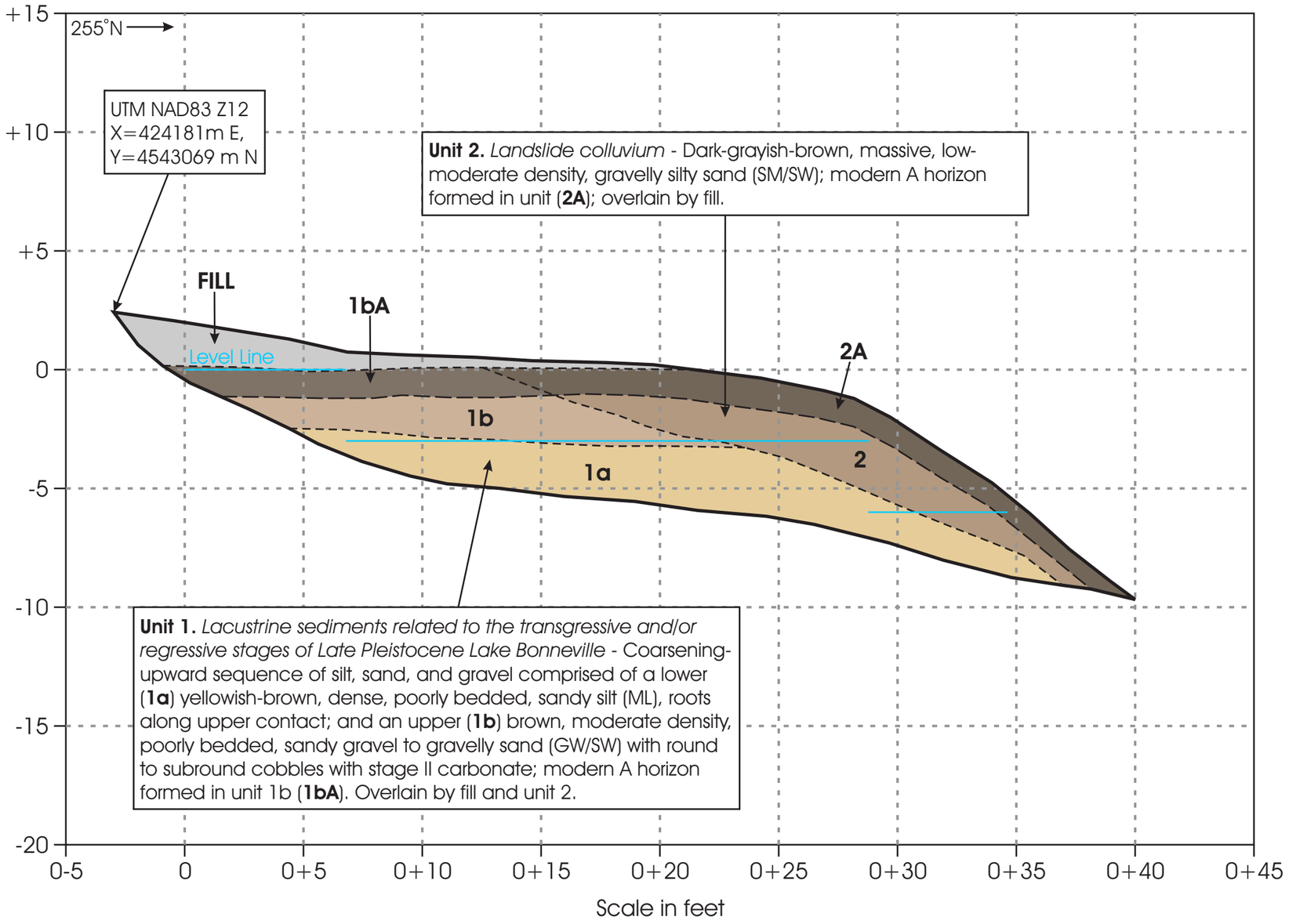
Logged by Bill D. Black, P.G. on December 20, 2018.
North wall logged, east to west. Scale 1 inch equals
5 feet (1:60) with no vertical exaggeration.

CMT ENGINEERING LABORATORIES	**Rock Loft Ridge Subdivision** About 250 North 1450 East, Fruit Heights, Utah	**Trench 6 Log, Sheet 1**	Date: 24-Oct-2020 CMT No.: 12188	Figure 11A



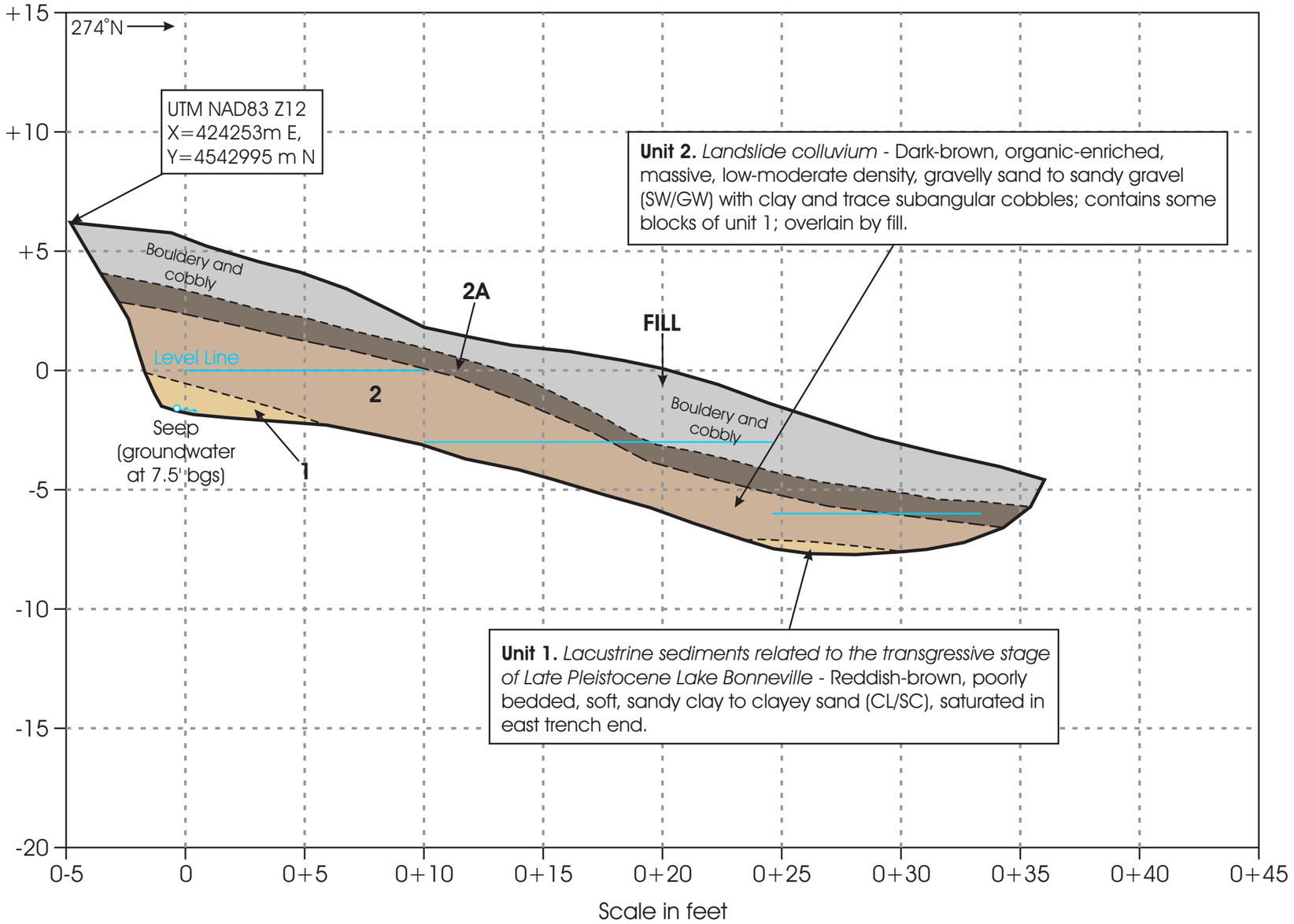
Logged by Bill D. Black, P.G. on December 20, 2018.
North wall logged, east to west. Scale 1 inch equals
5 feet (1:60) with no vertical exaggeration.

CMT ENGINEERING LABORATORIES	**Rock Loft Ridge Subdivision** About 250 North 1450 East, Fruit Heights, Utah	**Trench 6 Log, Sheet 2**	Date: 24-Oct-2020 CMT No.: 12188	Figure **11B**



Logged by Bill D. Black, P.G. on September 1, 2020.
North wall logged, east to west. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

CMT ENGINEERING LABORATORIES	**Rock Loft Ridge Subdivision** About 250 North 1450 East, Fruit Heights, Utah	**Trench 7 Log**	Date: 24-Oct-2020 CMT No.: 12188	Figure 12



Logged by Bill D. Black, P.G. on September 1, 2020.
North wall logged, east to west. Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

CMT ENGINEERING LABORATORIES

Rock Loft Ridge Subdivision
About 250 North 1450 East, Fruit Heights, Utah

Trench 8 Log

Date:	24-Oct-2020
CMT No.:	12188

Figure 13

Rock Loft Ridge Subdivision

Test Pit Log

TP-1

About 250 North 1450 East, Fruit Heights, Utah

Equipment: Rubber Tire Backhoe
Surface Elev. (approx):

Total Depth: 10.5'
Water Depth: (see Remarks)

Date: 12/13/18
Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Moisture (%)	Dry Density(pcf)	Gradation			Atterberg		
							Gravel %	Sand %	Fines %	LL	PL	PI
0		Brown Silty Fine to Coarse SAND (SP), with some fine and coarse gravel moist, medium dense (estimated_										
2				1								
4		Brown Sandy Fine and Coarse GRAVEL (GP), with some silt slightly moist, dense (estimated)										
6				2	1.9		38	44	18			
8												
10				3								
		END AT 10.5'										
12												
14												
16												
18												
20												
22												
24												
26												
28												

Remarks: Groundwater not encountered during excavation.

Figure: 14

CMT ENGINEERING LABORATORIES

Excavated By: Todd Nelson
Logged By: Hogan Wright

Rock Loft Ridge Subdivision

Test Pit Log

TP-2

About 250 North 1450 East, Fruit Heights, Utah

Equipment: Rubber Tire Backhoe
Surface Elev. (approx):

Total Depth: 11.5'
Water Depth: (see Remarks)

Date: 12/13/18
Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Moisture (%)	Dry Density(pcf)	Gradation Gravel %	Gradation Sand %	Gradation Fines %	Atterberg LL	Atterberg PL	Atterberg PI
0		Brown Silty Fine to Coarse SAND (SP), with some fine gravel slightly moist, medium dense (estimated)										
2				4								
4		grades with some coarse gravel										
				5	2.6		52	36	12			
6												
8												
10		Brown Sandy Fine and Coarse GRAVEL (GP), with occasional cobbles up to 8" in diameter dry, dense (estimated)		6								
12		END AT 11.5'										
14												
16												
18												
20												
22												
24												
26												
28												

Remarks: Groundwater not encountered during excavation.

Figure: 15

CMT ENGINEERING LABORATORIES

Excavated By: Todd Nelson
Logged By: Hogan Wright

Rock Loft Ridge Subdivision

Test Pit Log

TP-3

About 250 North 1450 East, Fruit Heights, Utah

Equipment: Rubber Tire Backhoe
Surface Elev. (approx):

Total Depth: 4'
Water Depth: (see Remarks)

Date: 12/13/18
Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Moisture (%)	Dry Density(pcf)	Gradation			Atterberg		
							Gravel %	Sand %	Fines %	LL	PL	PI
0		Brown Silty Fine to Coarse SAND (SP), with some fine and coarse gravel moist, medium dense (estimated)										
				7								
2		Gray BEDROCK (B) moderately hard (estimated)										
		very hard (estimated)										
4		REFUSAL AT 4'										
6												
8												
10												
12												
14												
16												
18												
20												
22												
24												
26												
28												

Remarks: Groundwater not encountered during excavation.

CMT ENGINEERING LABORATORIES

Excavated By: Todd Nelson
Logged By: Hogan Wright

Figure:

16

Rock Loft Ridge Subdivision

Test Pit Log

TP-4

About 250 North 1450 East, Fruit Heights, Utah

Equipment: Rubber Tire Backhoe | Total Depth: 13' | Date: 12/13/18

Surface Elev. (approx): | Water Depth: (see Remarks) | Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Moisture (%)	Dry Density(pcf)	Gradation Gravel %	Gradation Sand %	Gradation Fines %	Atterberg LL	Atterberg PL	Atterberg PI
0		Brown Silty Fine to Coarse SAND (SP) moist, medium dense (estimated)										
2				8								
4		grades with some fine and coarse gravel and occasional cobbles										
6				9	2.3		21	61	18			
8												
10		Brown Fine to Coarse Sandy Fine and Coarse GRAVEL (GP) and occasional cobbles up to 8" in diameter slightly moist, dense (estimated)										
12				10								
		END AT 13'										
14												
16												
18												
20												
22												
24												
26												
28												

Remarks: Groundwater not encountered during excavation.

Figure: 17

CMT ENGINEERING LABORATORIES

Excavated By: Todd Nelson

Logged By: Hogan Wright

Rock Loft Ridge Subdivision

Test Pit Log

TP-5

About 250 North 1450 East, Fruit Heights, Utah

Equipment: Rubber Tire Backhoe
Surface Elev. (approx):

Total Depth: 11'
Water Depth: (see Remarks)

Date: 12/13/18
Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Moisture (%)	Dry Density(pcf)	Gradation			Atterberg		
							Gravel %	Sand %	Fines %	LL	PL	PI
0		Brown Silty Fine to Coarse SAND (SP), with some fine and coarse gravel slightly moist, medium dense (estimated)										
2				11								
4		Brown Fine to Coarse Sandy Fine and Coarse GRAVEL (GP) slightly moist, dense (estimated)										
6				12								
8		grades with trace silt										
10				13	1.5		54	39	7			
		END AT 11'										
12												
14												
16												
18												
20												
22												
24												
26												
28												

Remarks: Groundwater not encountered during excavation.

Figure: 18

CMT ENGINEERING LABORATORIES

Excavated By: Todd Nelson
Logged By: Hogan Wright

Rock Loft Ridge Subdivision

Test Pit Log

TP-6

About 250 North 1450 East, Fruit Heights, Utah

Equipment: Rubber Tire Backhoe
Surface Elev. (approx):

Total Depth: 10.5'
Water Depth: (see Remarks)

Date: 12/13/18
Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Moisture (%)	Dry Density(pcf)	Gradation			Atterberg		
							Gravel %	Sand %	Fines %	LL	PL	PI
0		Brown Silty Fine to Coarse SAND (SM), with some fine gravel moist, medium dense (estimated)										
2												
				1								
4		grades with coarse sand										
6				2								
8												
10				3								
		END AT 10.5'										
12												
14												
16												
18												
20												
22												
24												
26												
28												

Remarks: Groundwater not encountered during excavation.

Figure: 19

CMT ENGINEERING LABORATORIES

Excavated By: Todd Nelson
Logged By: Hogan Wright

Rock Loft Ridge Subdivision

Test Pit Log

TP-7

About 250 North 1450 East, Fruit Heights, Utah

Equipment: Rubber Tire Backhoe
Surface Elev. (approx):

Total Depth: 9'
Water Depth: (see Remarks)

Date: 12/13/18
Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Moisture (%)	Dry Density(pcf)	Gradation			Atterberg		
							Gravel %	Sand %	Fines %	LL	PL	PI
0		Brown Silty Fine to Coarse SAND (SM), with some fine and coarse gravel and occasional cobbles up to 8" in diameter slightly moist, medium dense (estimated)										
6		Brown Silty Fine to Coarse Sandy GRAVEL (GM), with some cobbles up to 10" in diameter										
9		END AT 9'										
10												
12												
14												
16												
18												
20												
22												
24												
26												
28												

Remarks: Groundwater not encountered during excavation.

Figure: 20

CMT ENGINEERING LABORATORIES

Excavated By: Todd Nelson
Logged By: Hogan Wright

Rock Loft Ridge Subdivision

Test Pit Log

TP-8

About 250 North 1450 East, Fruit Heights, Utah

Equipment: Rubber Tire Backhoe
Surface Elev. (approx):

Total Depth: 11'
Water Depth: (see Remarks)

Date: 12/13/18
Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Moisture (%)	Dry Density(pcf)	Gradation Gravel %	Gradation Sand %	Gradation Fines %	Atterberg LL	Atterberg PL	Atterberg PI
0		Brown Silty Fine to Coarse SAND (SM), with some fine and coarse gravel moist, medium dense (estimated)										
2				4								
4		Brown Silty Fine to Coarse Sandy GRAVEL (GM), with some cobbles and occasional boulders up to 2" in diameter slightly moist, dense (estimated)										
6				5	3.3		36	51	13			
8												
10				6								
		END AT 11.0'										
12												
14												
16												
18												
20												
22												
24												
26												
28												

Remarks: Groundwater not encountered during excavation.

Figure: 21

CMT ENGINEERING LABORATORIES

Excavated By: Todd Nelson
Logged By: Hogan Wright

Rock Loft Ridge Subdivision

Test Pit Log

TP-9

About 250 North 1450 East, Fruit Heights, Utah

Equipment: Rubber Tire Backhoe
Surface Elev. (approx):

Total Depth: 10.5'
Water Depth: (see Remarks)

Date: 12/13/18
Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Moisture (%)	Dry Density(pcf)	Gradation			Atterberg		
							Gravel %	Sand %	Fines %	LL	PL	PI
0		TOPSOIL, upper 2-3", loose to 6"										
		Brown Silty Fine to Coarse SAND (SM), with some fine and coarse gravel with occasional cobbles and boulders up to 2' in diameter slightly moist, medium dense (estimated)										
2				7								
4												
6				8								
		Brown Silty Fine to Coarse Sandy Fine and Coarse GRAVEL (GM), with occasional cobbles and boulders up to 2' in diameter slightly moist, dense (estimated)										
8				9								
10		END AT 10.5'										
12												
14												
16												
18												
20												
22												
24												
26												
28												

Remarks: Groundwater not encountered during excavation.

Figure: 22

CMT ENGINEERING LABORATORIES

Excavated By: Todd Nelson
Logged By: Hogan Wright

Rock Loft Ridge Subdivision

Test Pit Log

TP-10

About 250 North 1450 East, Fruit Heights, Utah

Equipment: Rubber Tire Backhoe
Surface Elev. (approx):

Total Depth: 10.5'
Water Depth: (see Remarks)

Date: 12/13/18
Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Moisture (%)	Dry Density(pcf)	Gradation			Atterberg		
							Gravel %	Sand %	Fines %	LL	PL	PI
0		Brown Silty Fine to Coarse SAND (SM) with some fine and coarse gravel slightly moist, medium dense (estimated)										
2												
				10	2.64				8			
4												
6												
				11								
8												
10				12								
		END AT 10.5'										
12												
14												
16												
18												
20												
22												
24												
26												
28												

Remarks: Groundwater not encountered during excavation.

Figure: 23

CMT ENGINEERING LABORATORIES

Excavated By: Todd Nelson
Logged By: Hogan Wright

Rock Loft Ridge Subdivision

Test Pit Log

TP-11

About 250 North 1450 East, Fruit Heights, Utah

Equipment: Rubber Tire Backhoe
Surface Elev. (approx):

Total Depth: 10'
Water Depth: (see Remarks)

Date: 12/13/18
Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Moisture (%)	Dry Density(pcf)	Gradation			Atterberg		
							Gravel %	Sand %	Fines %	LL	PL	PI
0		TOPSOIL, upper 2-3", loose to 6"										
		Brown Silty Fine to Coarse SAND (SM), with trace gravel moist, medium dense (estimated)										
2												
		Brown Sandy Fine and Coarse GRAVEL (GM), with some silt slightly moist, medium dense (estimated)		1								
4												
				2								
6												
		Brown Silty Fine SAND (SM) slightly moist, medium dense (estimated) grades with thin interbedded fine sandy silt layers										
8												
				3	4.82				49			
10		END AT 10'										
12												
14												
16												
18												
20												
22												
24												
26												
28												

Remarks: Groundwater not encountered during excavation.

Figure: 24

CMT ENGINEERING LABORATORIES

Excavated By: Todd Nelson
Logged By: Hogan Wright

Rock Loft Ridge Subdivision

Test Pit Log

TP-12

About 250 North 1450 East, Fruit Heights, Utah

Equipment: Rubber Tire Backhoe
Surface Elev. (approx):

Total Depth: 10.5'
Water Depth: (see Remarks)

Date: 12/13/18
Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Moisture (%)	Dry Density(pcf)	Gradation			Atterberg		
							Gravel %	Sand %	Fines %	LL	PL	PI
0		TOPSOIL, upper 2-3", loose to 6"										
		Brown Slightly Cemented Silty Fine to Coarse SAND (SM), with some fine and coarse gravel slightly moist, medium dense to dense (estimated)										
2												
				4								
4												
6				5								
8		Brown Fine to Coarse Bedded SAND (SP) dry, medium dense (estimated)										
				6								
10		END AT 10.5'										
12												
14												
16												
18												
20												
22												
24												
26												
28												

Remarks: Groundwater not encountered during excavation.

Figure: 25

CMT ENGINEERING LABORATORIES

Excavated By: Todd Nelson
Logged By: Hogan Wright

Rock Loft Ridge Subdivision

Test Pit Log

TP-13

About 250 North 1450 East, Fruit Heights, Utah

Equipment: Rubber Tire Backhoe
Surface Elev. (approx):

Total Depth: 10.5'
Water Depth: (see Remarks)

Date: 12/13/18
Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Moisture (%)	Dry Density(pcf)	Gradation			Atterberg		
							Gravel %	Sand %	Fines %	LL	PL	PI
0		TOPSOIL, upper 2-3", loose to 6"										
		Brown Silty Fine to Coarse SAND (SM), with fine and coarse gravel and occasional cobbles up to 10" in diameter slightly moist, medium dense to dense (estimated)										
2												
				7								
4												
6		medium coarse sand grades out, trace gravel, no cobbles										
8				8	3.6		1	44	55			
		Brown Fine to Coarse SAND (SP), with some silt dry, medium dense (estimated)										
10				9								
		END AT 10.5'										
12												
14												
16												
18												
20												
22												
24												
26												
28												

Remarks: Groundwater not encountered during excavation.

Figure: 26

CMT ENGINEERING LABORATORIES

Excavated By: Todd Nelson
Logged By: Hogan Wright

Rock Loft Ridge Subdivision

Test Pit Log

TP-14

About 250 North 1450 East, Fruit Heights, Utah

Equipment: Rubber Tire Backhoe
Surface Elev. (approx):

Total Depth: 10'
Water Depth: (see Remarks)

Date: 12/13/18
Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Moisture (%)	Dry Density(pcf)	Gradation			Atterberg		
							Gravel %	Sand %	Fines %	LL	PL	PI
0		Brown Fine to Coarse SAND (SP), with some silt, trace gravel slightly moist, medium dense (estimated)										
2												
				10								
4		medium to coarse sand and gravel grade out										
6				11								
8		Brown Bedded Silty Fine SAND (SM) dry, medium dense (estimated)										
				12								
10		END AT 10'										
12												
14												
16												
18												
20												
22												
24												
26												
28												

Remarks: Groundwater not encountered during excavation.

CMT ENGINEERING LABORATORIES

Excavated By: Todd Nelson
Logged By: Hogan Wright

Figure: 27

Rock Loft Ridge Subdivision

Test Pit Log

TP-15

About 250 North 1450 East, Fruit Heights, Utah

Equipment: Rubber Tire Backhoe | Total Depth: 10' | Date: 12/13/18

Surface Elev. (approx): | Water Depth: (see Remarks) | Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Moisture (%)	Dry Density(pcf)	Gradation			Atterberg		
							Gravel %	Sand %	Fines %	LL	PL	PI
0		TOPSOIL, upper 2-3", loose to 6"										
		Brown Fine to Coarse SAND (SP), with some silt and trace gravel moist, medium dense (estimated)										
2				13								
		Brown Fine Sandy SILT (ML), with some clay and trace gravel slightly moist, stiff to very stiff to hard (estimated)										
4												
6		gravel grades out										
				14	21	86				30	21	9
8				15								
10				16								
		END AT 10'										
12												
14												
16												
18												
20												
22												
24												
26												
28												

Remarks: Groundwater not encountered during excavation.

Figure: 28

CMT ENGINEERING LABORATORIES

Excavated By: Todd Nelson

Logged By: Hogan Wright

Rock Loft Ridge Subdivision

Bore Hole Log

B-1

About 250 North 1450 East, Fruit Heights, Utah

Boring Type: Hollow-Stem Auger | Total Depth: 46.5' | Date: 12/11/18

Surface Elev. (approx): | Water Depth: (see Remarks) | Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Blows (N)	Total	Moisture (%)	Dry Density(pcf)	Gradation Gravel %	Gradation Sand %	Gradation Fines %	Atterberg LL	Atterberg PL	Atterberg PI
0		Brown Fine to Coarse Sandy Fine and Coarse GRAVEL (GP), with slightly moist, medium dense												
				1	13 16 18	34								
4		Brown Fine to Coarse SAND (SP-SM), with some silt slightly moist, medium dense												
				2	12 16 22	38	4.4	127	31	58	11			
				3	13 15 23	38								
				4	12 16 23	39								
12				5	10 24 45	69								
20				6	24 36 32	68								
24				7	15 25 20	45	4	120			12			
28														

Remarks: Groundwater not encountered during drilling.

Figure: 29

CMT ENGINEERING LABORATORIES

Drilled By: John Mower

Logged By: Hogan Wright

Rock Loft Ridge Subdivision

Bore Hole Log

B-1

About 250 North 1450 East, Fruit Heights, Utah

Boring Type: Hollow-Stem Auger | Total Depth: 46.5' | Date: 12/11/18
Surface Elev. (approx): | Water Depth: see Remark | Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Blows (N)	Total	Moisture (%)	Dry Density(pcf)	Gradation Gravel %	Gradation Sand %	Gradation Fines %	Atterberg LL	Atterberg PL	Atterberg PI
28		Brown Silty Fine SAND (SM) slightly moist												
				8	11 14 19	33	3.9	108			10			
32		Brown Fine to Coarse SAND (SP), with some silt, and trace gravel slightly moist, dense												
				9	10 20 55	75								
36		Brown Fine to Coarse Sandy Fine and Coarse GRAVEl (GP) dry, dense												
		Brown Highly Weathered BEDROCK (B) dry, very dense												
40				10										
44				11		50/3"								
		REFUSAL AT 46.5'												
48														
52														
56														

Remarks: Groundwater not encountered during drilling.

Figure: 29

CMT ENGINEERING LABORATORIES

Drilled By: John Mower
Logged By: Hogan Wright

Rock Loft Ridge Subdivision

Bore Hole Log

B-2

About 250 North 1450 East, Fruit Heights, Utah

Boring Type: Hollow-Stem Auger | Total Depth: 31.5' | Date: 12/11/18

Surface Elev. (approx): | Water Depth: (see Remarks) | Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Blows (N)	Total	Moisture (%)	Dry Density(pcf)	Gradation Gravel %	Gradation Sand %	Gradation Fines %	Atterberg LL	Atterberg PL	Atterberg PI
0		TOPSOIL												
		Brown Silty Fine to Coarse SAND (SM), with trace fine gravel moist, loose		12	5 6 8	14								
4				13	5 6 7	13								
		medium dense		14	7 12 14	26								
8				15	11 11 11	22	5	118			13			
		loose												
12				16	7 10 13	23								
		Brown Fine to Coarse SAND (SP), with some silt, and trace gravel slightly moist, medium dense												
16				17	9 11 16	27								
		grades with some fine gravel and trace coarse gravel slightly moist, medium dense												
20				18	9 11 17	28								
24														
		dense												
28														

Remarks: Groundwater not encountered during drilling.

Figure: 30

CMT ENGINEERING LABORATORIES

Drilled By: John Mower

Logged By: Hogan Wright

Rock Loft Ridge Subdivision

Bore Hole Log

B-2

About 250 North 1450 East, Fruit Heights, Utah

Boring Type: Hollow-Stem Auger | Total Depth: 31.5' | Date: 12/11/18

Surface Elev. (approx): | Water Depth: see Remark | Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Blows (N)		Moisture (%)	Dry Density(pcf)	Gradation			Atterberg		
						Total			Gravel %	Sand %	Fines %	LL	PL	PI
28		dry												
		Brown Highly Weathered BEDROCK (B)												
		dry, very dense			37									
				19	50	50/2"								
32		REFUSAL AT 31.5'												
36														
40														
44														
48														
52														
56														

Remarks: Groundwater not encountered during drilling.

Figure: 30

CMT ENGINEERING LABORATORIES

Drilled By: John Mower

Logged By: Hogan Wright

Rock Loft Ridge Subdivision

Bore Hole Log

B-3

About 250 North 1450 East, Fruit Heights, Utah

Boring Type: Hollow-Stem Auger | Total Depth: 36.5' | Date: 12/11/18

Surface Elev. (approx): | Water Depth: (see Remarks) | Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Blows (N)	Total	Moisture (%)	Dry Density(pcf)	Gradation Gravel %	Gradation Sand %	Gradation Fines %	Atterberg LL	Atterberg PL	Atterberg PI
0		TOPSOIL, upper 2-4", loose to 6"												
		Brown Silty Fine to Coarse SAND (SP), with trace fine gravel moist, medium dense												
				20	6 9 17	26								
4														
		Brown Fine Sandy SILT (ML), with some clay slightly moist, very stiff												
				21	9 14 38	52	6.5	96			82	26	23	3
8		hard		22	27 53 50	103								
				23	14 50 50	100								
12														
		very stiff		24	7 24 16	40	7.5				79			
16														
		Brown Silty Fine SAND (SP), with some thin interbedded silty clay layers moist, dense												
20				25	6 24 55	79								
		Brown Fine Sandy SILT (ML), with some clay slightly moist, hard												
24				26	17 32 66	98								
28														

Remarks: Groundwater not encountered during drilling.

Figure: 31

CMT ENGINEERING LABORATORIES

Drilled By: John Mower

Logged By: Hogan Wright

Rock Loft Ridge Subdivision

Bore Hole Log

B-3

About 250 North 1450 East, Fruit Heights, Utah

Boring Type: Hollow-Stem Auger | Total Depth: 36.5' | Date: 12/11/18

Surface Elev. (approx): | Water Depth: see Remark | Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Blows (N)	Total	Moisture (%)	Dry Density(pcf)	Gradation Gravel %	Gradation Sand %	Gradation Fines %	Atterberg LL	Atterberg PL	Atterberg PI
28														
				27	16 43 67	110								
32														
				28	10 45 50	95								
36		Auger refusal at 34 feet												
		END AT 36.5'												
40														
44														
48														
52														
56														

Remarks: Groundwater not encountered during drilling.

Figure: 31

CMT ENGINEERING LABORATORIES

Drilled By: John Mower

Logged By: Hogan Wright

Rock Loft Estates

Bore Hole Log

B-4A

About 250 North 1450 East, Fruit Heights, Utah

Boring Type:	Hollow-Stem Auger	Total Depth:	101.5'	Date:	9/5/19
Surface Elev. (approx):	N/A	Water Depth:	55'	Job #:	12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Blows (N)	Total	Moisture (%)	Dry Density(pcf)	Gradation Gravel %	Gradation Sand %	Gradation Fines %	Atterberg LL	Atterberg PL	Atterberg PI
0		Brown Sandy Clay (CL) with fine to coarse sand, trace coarse gravel — slightly moist, hard												
4				1	15 19	32								
8		trace coarse gravel grades out — stiff		2	13 7 5	9								
12				3	4 7 8	26								
16		Yellowish-Brown Well-graded Sand (SM) with trace fine gravel — slightly moist, medium dense			18									
20		Yellowish-Brown Poorly-graded Sand (SP) — slightly moist, medium dense		4	8 10 10	20								
24				5	8 13 9	22								
28														

Remarks: Groundwater encountered during drilling at depth of 55 feet.

Slotted PVC pipe installed to depth of 100 feet to facilitate water level measurements.

CMT ENGINEERING LABORATORIES

Drilled By: Earthcore

Logged By: Michelle/Sterling

Figure: 32

Rock Loft Estates

Bore Hole Log

B-4A

About 250 North 1450 East, Fruit Heights, Utah

Boring Type:	Hollow-Stem Auger	Total Depth:	101.5'	Date:	9/5/19
Surface Elev. (approx):	N/A	Water Depth:	55'	Job #:	12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Blows (N)	Total	Moisture (%)	Dry Density(pcf)	Gradation Gravel %	Gradation Sand %	Gradation Fines %	Atterberg LL	Atterberg PL	Atterberg PI
28														
		Light Brown Clayey Silt (CL-ML) with fine sand slightly moist, hard		6	13 14 9	23								
32														
		moist, very stiff		7	9 12 15	27								
36														
				8	8 6 4	10						25	21	4
40														
44		stiff		9	4 4 8	12						26	20	6
48		very stiff		10	6 12 16	28						23	19	4
52														
		wet		11	5 6	15	26.1				90.2	24	19	5
56														

Remarks: Groundwater encountered during drilling at depth of 55 feet.

Slotted PVC pipe installed to depth of 100 feet to facilitate water level measurements.

Figure: 32

CMT ENGINEERING LABORATORIES

Drilled By: Earthcore

Logged By: Michelle/Sterling

Rock Loft Estates

Bore Hole Log

B-4A

About 250 North 1450 East, Fruit Heights, Utah

Boring Type:	Hollow-Stem Auger	Total Depth:	101.5'	Date:	9/5/19
Surface Elev. (approx):	N/A	Water Depth:	55'	Job #:	12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Blows (N)	Total	Moisture (%)	Dry Density(pcf)	Gradation Gravel %	Gradation Sand %	Gradation Fines %	Atterberg LL	Atterberg PL	Atterberg PI
56					9									
60														
64														
68														
72		Dark Yellowish-Brown Sandy Silt (ML) with fine grained sand and some clay — wet, medium stiff		12	2 3 4	7	25.7				90.6	23	21	2
76				13	4 10 12	22								
80				14	6 14 18	32								
84														

Remarks: Groundwater encountered during drilling at depth of 55 feet.

Slotted PVC pipe installed to depth of 100 feet to facilitate water level measurements.

CMT ENGINEERING LABORATORIES

Drilled By: Earthcore

Logged By: Michelle/Sterling

Figure: 32

Rock Loft Estates

Bore Hole Log

B-4A

About 250 North 1450 East, Fruit Heights, Utah

Boring Type:	Hollow-Stem Auger	Total Depth:	101.5'	Date:	9/5/19
Surface Elev. (approx):	N/A	Water Depth:	55'	Job #:	12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Blows (N)	Moisture (%)	Dry Density(pcf)	Gradation Gravel %	Gradation Sand %	Gradation Fines %	Atterberg LL	Atterberg PL	Atterberg PI
84													
		Brown to Gray Silty Clay (CL-ML) with sand layers wet, stiff to medium stiff		15	6 7 8 15						23	22	1
88													
				16	6 12 13 25								
92													
				17	5 8 12 20								
96													
				18	7 8 11 19	21.8				85.6	24	23	1
100		END AT 101.5'											
				19									
104													
				20									
108													
112													

Remarks: Groundwater encountered during drilling at depth of 55 feet.

Slotted PVC pipe installed to depth of 100 feet to facilitate water level measurements.

CMT ENGINEERING LABORATORIES

Drilled By: Earthcore

Logged By: Michelle/Sterling

Figure: 32

Rock Loft Ridge Subdivision

Bore Hole Log

B-5

About 250 North 1450 East, Fruit Heights, Utah

Boring Type: Hollow-Stem Auger | Total Depth: 30' | Date: 12/11/18

Surface Elev. (approx): | Water Depth: (see Remarks) | Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Blows (N)	Total	Moisture (%)	Dry Density(pcf)	Gradation Gravel %	Gradation Sand %	Gradation Fines %	Atterberg LL	Atterberg PL	Atterberg PI
0		FILL, Brown SILT (ML), with some fine sand and trace gravel												
				41	7 14 24	38	4.32					21	20	1
4		FILL, Brown Silty Fine to Coarse SAND (SM), with some fine gravel												
				42	10 15 16	31								
		Brown Silty Fine SAND (SM), with some medium to coarse sand, and trace gravel slightly moist												
8				43	15 21 19	40	25	82			27			
		gravel layer approximately 1' thick		44	50									
12		medium to coarse sand and gravel grade out												
16				45	15 23 48	71								
		Brown Fine Cemented Sandy SILT/CLAY (ML-CL) slightly moist, hard												
20				46	10 40 75	115	7.9	107			85	27	20	7
24		Brown Clayey SILT (ML), with some fine sand slightly moist, hard												
				47	40 70 50	120	6.7	105			87	25	22	3
28														

Remarks: Groundwater not encountered during drilling.

Figure: 33

CMT ENGINEERING LABORATORIES

Drilled By: John Mower

Logged By: Hogan Wright

Rock Loft Ridge Subdivision

Bore Hole Log

B-5

About 250 North 1450 East, Fruit Heights, Utah

Boring Type: Hollow-Stem Auger | Total Depth: 30' | Date: 12/11/18

Surface Elev. (approx): | Water Depth: see Remark | Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Blows (N)	Total	Moisture (%)	Dry Density(pcf)	Gradation Gravel %	Gradation Sand %	Gradation Fines %	Atterberg LL	Atterberg PL	Atterberg PI
28														
				48	32 62 50	112								
		REFUSAL AT 30'												
32														
36														
40														
44														
48														
52														
56														

Remarks: Groundwater not encountered during drilling.

Figure: 33

CMT ENGINEERING LABORATORIES

Drilled By: John Mower

Logged By: Hogan Wright

Rock Loft Estates

About 250 North 1450 East, Fruit Heights, Utah

Key to Symbols

Date: 9/5/19
Job #: 12188

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Blows(N)	Total	Moisture (%)	Dry Density(pcf)	Gradation ⑩ Gravel %	Sand %	Fines %	Atterberg ⑪ LL	PL	PI
①	②	③	④	⑤	⑥	⑦	⑧	⑨						

COLUMN DESCRIPTIONS

① **Depth (ft.):** Depth (feet) below the ground surface (including groundwater depth - see water symbol below).

② **Graphic Log:** Graphic depicting type of soil encountered (see ② below).

③ **Soil Description:** Description of soils encountered, including Unified Soil Classification Symbol (see below).

④ **Sample Type:** Type of soil sample collected at depth interval shown; sampler symbols are explained below-right.

⑤ **Sample #:** Consecutive numbering of soil samples collected during field exploration.

⑥ **Blows:** Number of blows to advance sampler in 6" increments, using a 140-lb hammer with 30" drop.

⑦ **Total Blows:** Number of blows to advance sampler the 2nd and 3rd 6" increments.

⑧ **Moisture (%):** Water content of soil sample measured in laboratory (percentage of dry weight of sample).

⑨ **Dry Density (pcf):** The dry density of a soil measured in laboratory (pounds per cubic foot).

⑩ **Gradation:** Percentages of Gravel, Sand and Fines (Silt/Clay), obtained from lab test results of soil passing the No. 4 and No. 200 sieves.

⑪ **Atterberg:** Individual descriptions of Atterberg Tests are as follows:

LL = Liquid Limit (%): Water content at which a soil changes from plastic to liquid behavior.

PL = Plastic Limit (%): Water content at which a soil changes from liquid to plastic behavior.

PI = Plasticity Index (%): Range of water content at which a soil exhibits plastic properties (= Liquid Limit - Plastic Limit).

STRATIFICATION	
Description	**Thickness**
Seam	Up to ½ inch
Lense	Up to 12 inches
Layer	Greater than 12 in.
Occasional	1 or less per foot
Frequent	More than 1 per foot

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

MOISTURE CONTENT
Dry: Absence of moisture, dusty, dry to the touch.
Moist: Damp / moist to the touch, but no visible water.
Saturated: Visible water, usually soil below groundwater.

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

MAJOR DIVISIONS			USCS SYMBOLS	②	TYPICAL DESCRIPTIONS
COARSE-GRAINED SOILS More than 50% of material is larger than No. 200 sieve size.	GRAVELS The coarse fraction retained on No. 4 sieve.	CLEAN GRAVELS (< 5% fines)	GW		Well-Graded Gravels, Gravel-Sand Mixtures, Little or No Fines
			GP		Poorly-Graded Gravels, Gravel-Sand Mixtures, Little or No Fines
		GRAVELS WITH FINES (≥ 12% fines)	GM		Silty Gravels, Gravel-Sand-Silt Mixtures
			GC		Clayey Gravels, Gravel-Sand-Clay Mixtures
	SANDS The coarse fraction passing through No. 4 sieve.	CLEAN SANDS (< 5% fines)	SW		Well-Graded Sands, Gravelly Sands, Little or No Fines
			SP		Poorly-Graded Sands, Gravelly Sands, Little or No Fines
		SANDS WITH FINES (≥ 12% fines)	SM		Silty Sands, Sand-Silt Mixtures
			SC		Clayey Sands, Sand-Clay Mixtures
FINE-GRAINED SOILS More than 50% of material is smaller than No. 200 sieve size.	SILTS AND CLAYS Liquid Limit less than 50%		ML		Inorganic Silts and Very Fine Sands, Rock Flour, Silty or Clayey Fine Sands or Clayey Silts with Slight Plasticity
			CL		Inorganic Clays of Low to Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean Clays
			OL		Organic Silts and Organic Silty Clays o f Low Plasticity
	SILTS AND CLAYS Liquid Limit greater than 50%		MH		Inorganic Silts, Micacious or Diatomacious Fine Sand or Silty Soils
			CH		Inorganic Clays of High Plasticity, Fat Clays
			OH		Organic Silts and Organic Clays of Medium to High Plasticity
HIGHLY ORGANIC SOILS			PT		Peat, Humus, Swamp Soils with High Organic Contents

Note: Dual Symbols are used to indicate borderline soil classifications (i.e. GP-GM, SC-SM, etc.).

SAMPLER SYMBOLS

- Block Sample
- Bulk/Bag Sample
- Modified California Sampler 3.5" OD, 2.42" ID D&M Sampler
- Rock Core
- Standard Penetration Split Spoon Sampler
- Thin Wall (Shelby Tube)

WATER SYMBOL

- Encountered Water Level
- Measured Water Level

(see Remarks on Logs)

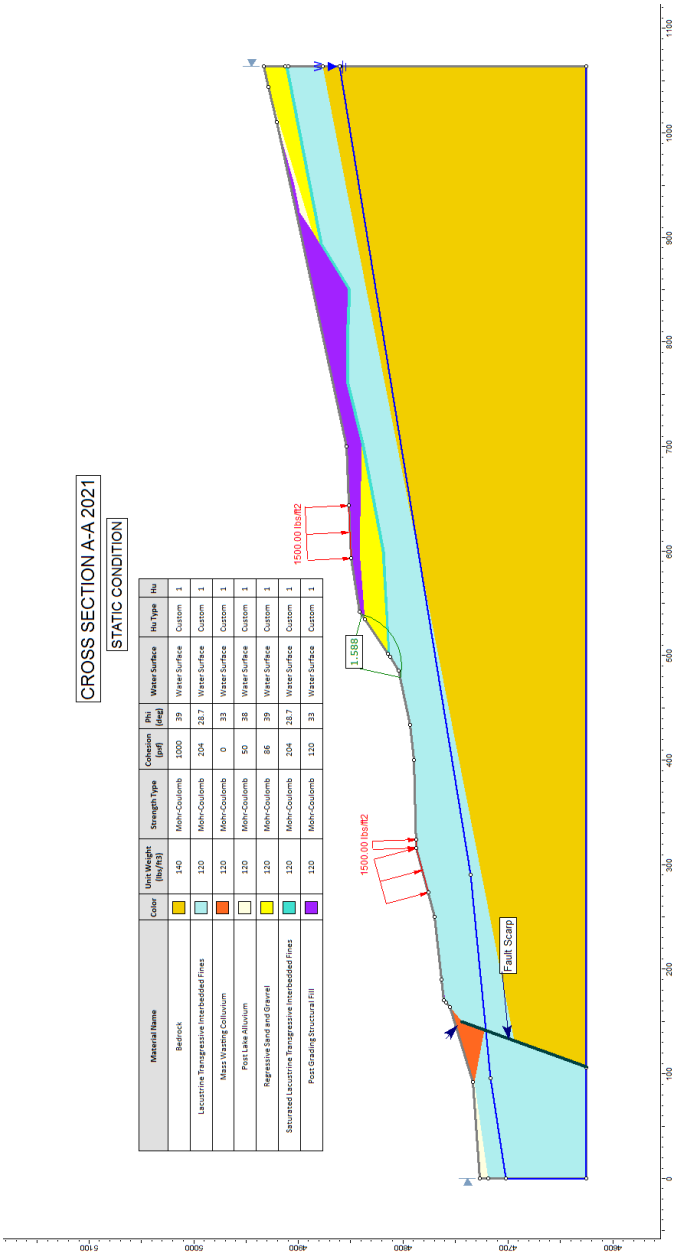
1. The results of laboratory tests on the samples collected are shown on the logs at the respective sample depths.
2. The subsurface conditions represented on the logs are for the locations specified. Caution should be exercised if interpolating between or extrapolating beyond the exploration locations.
3. The information presented on each log is subject to the limitations, conclusions, and recommendations presented in this report.

CMT ENGINEERING LABORATORIES

Figure: 34

STABILITY RESULTS

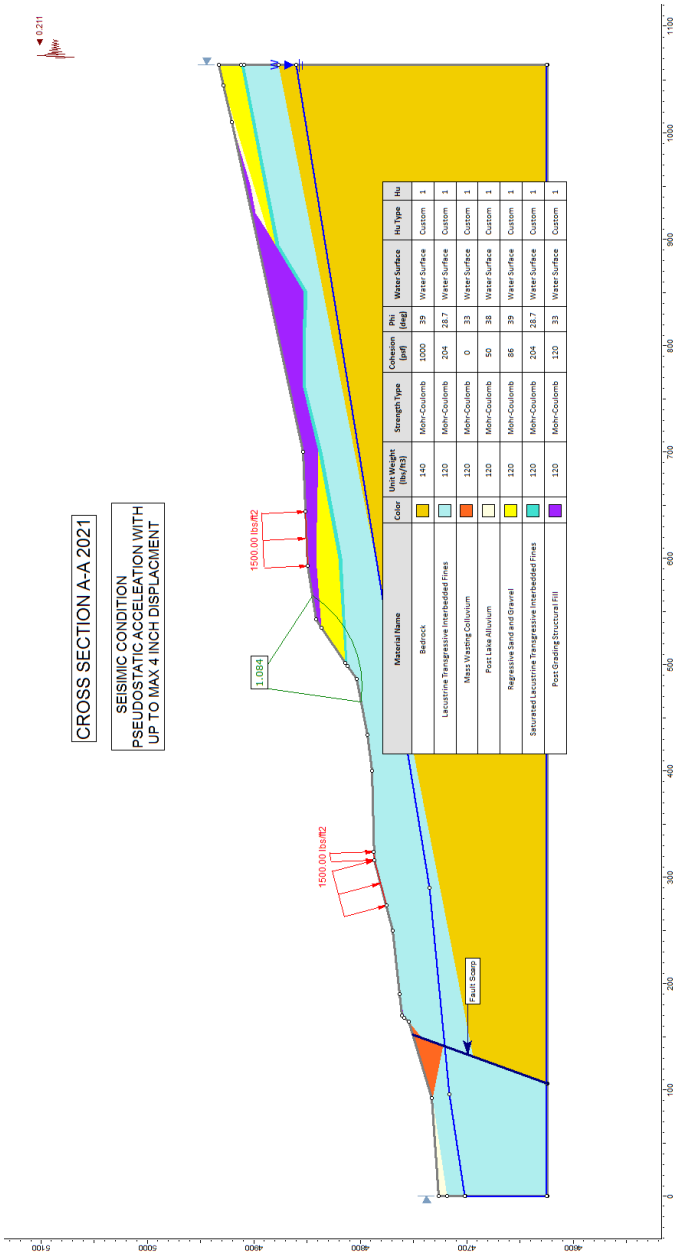
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: A-A
Static Condition



PROJECT NO.: 12188		**FIGURE NO.:** 35-1

STABILITY RESULTS

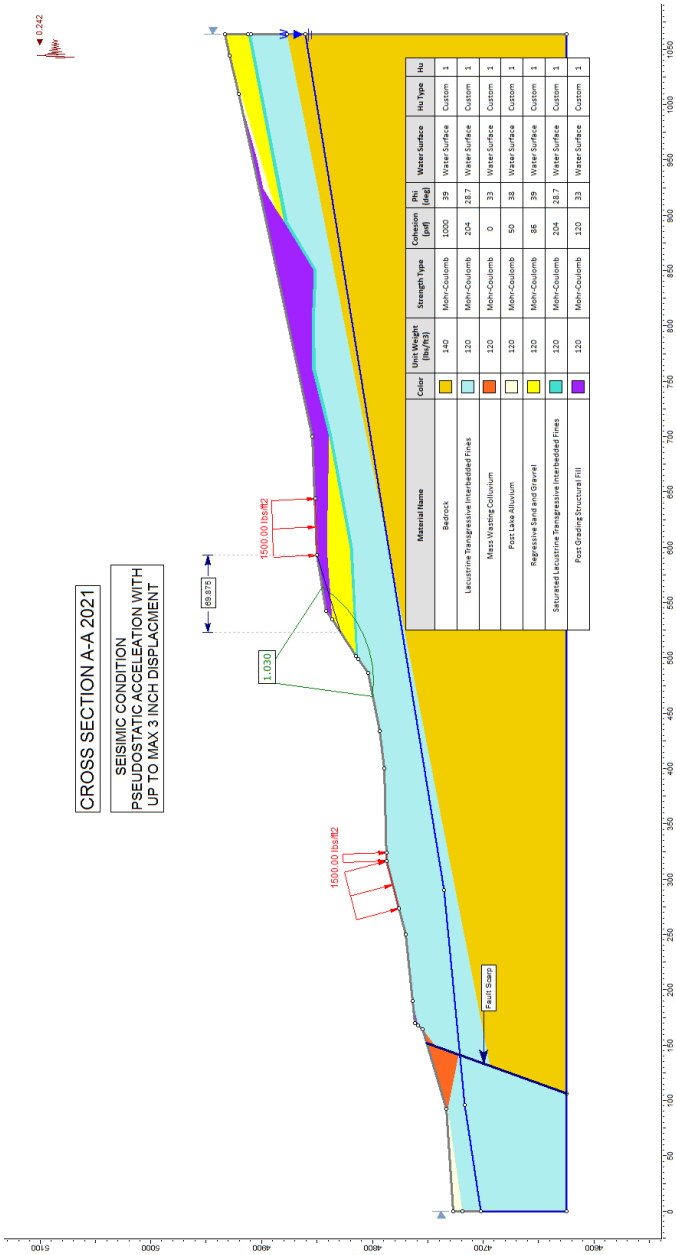
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: A-A
Seismic Condition
Pseudostatic Acceleration with Up to 4-Inch Displacement



PROJECT NO.: 12188	**FIGURE NO.:** 35-2

STABILITY RESULTS

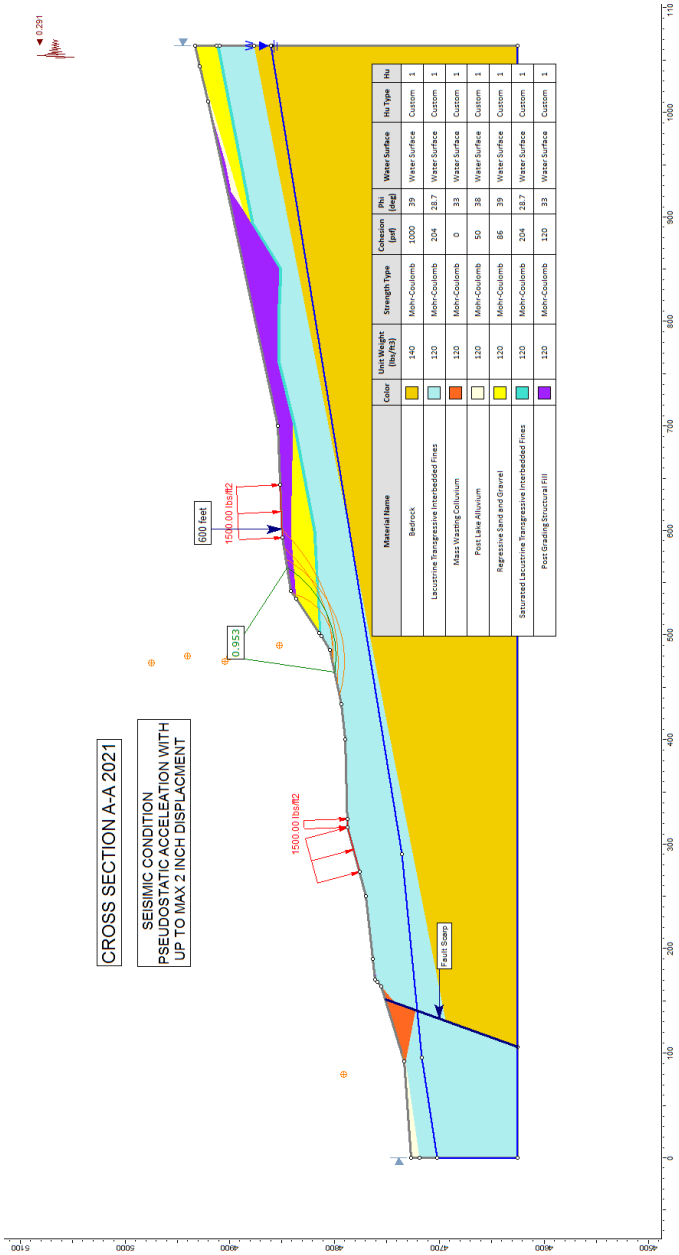
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: A-A
Seismic Condition
Pseudostatic Acceleration with Up to 3-Inch Displacement



PROJECT NO.: 12188	**FIGURE NO.:** 35-3

STABILITY RESULTS

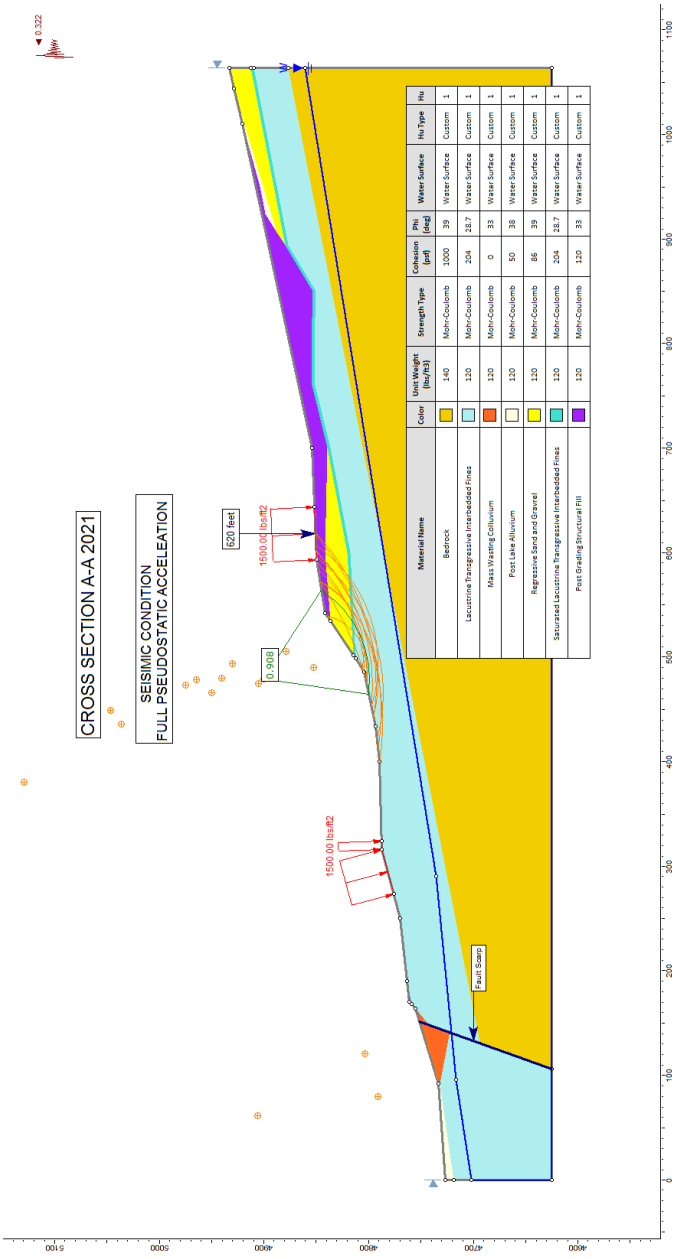
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: A-A
Seismic Condition
Pseudostatic Acceleration with Up to 2-Inch Displacement



PROJECT NO.: 12188		**FIGURE NO.:** 35-4

STABILITY RESULTS

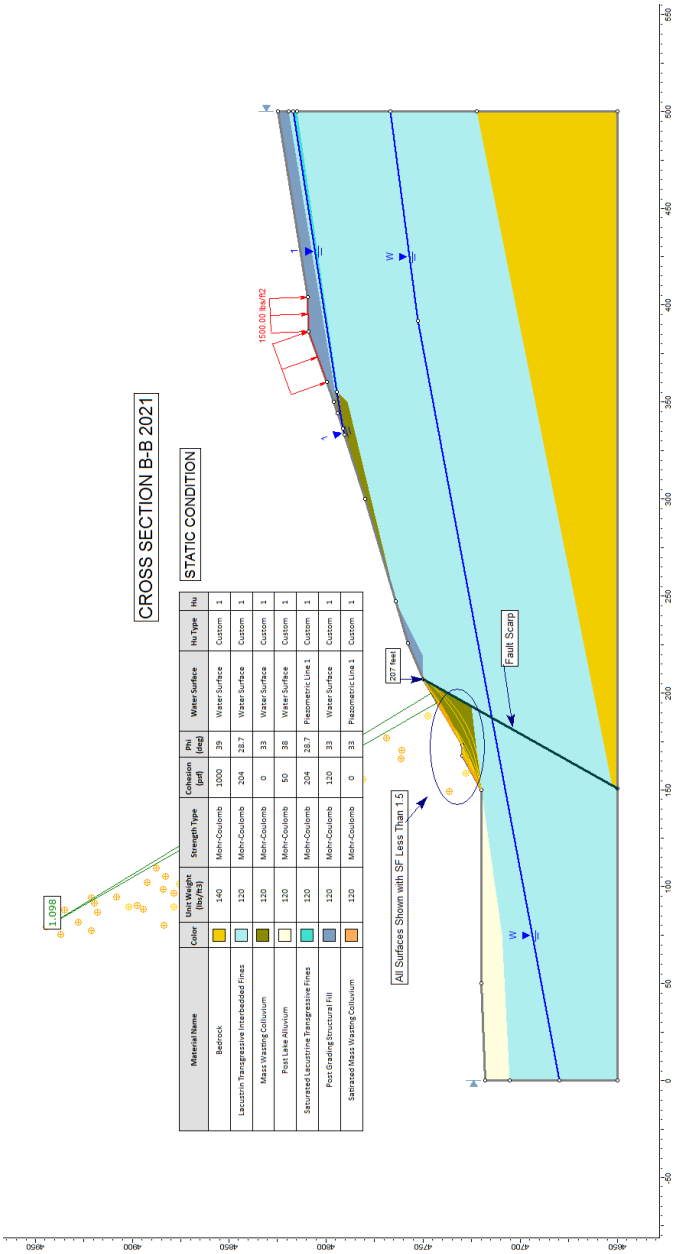
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: A-A
Seismic Condition
Full Pseudostatic Acceleration



PROJECT NO.: 12188		**FIGURE NO.:** 35-5

STABILITY RESULTS

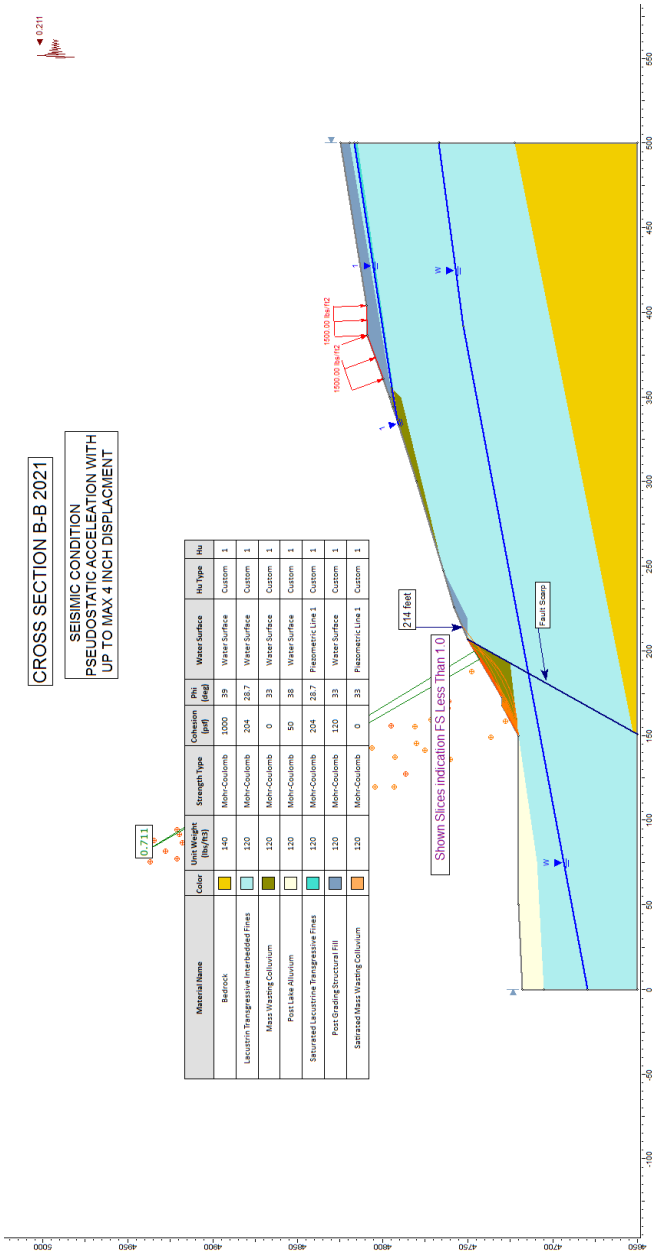
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: B-B
Static Condition



PROJECT NO.: 12188		**FIGURE NO.:** 36-1

STABILITY RESULTS

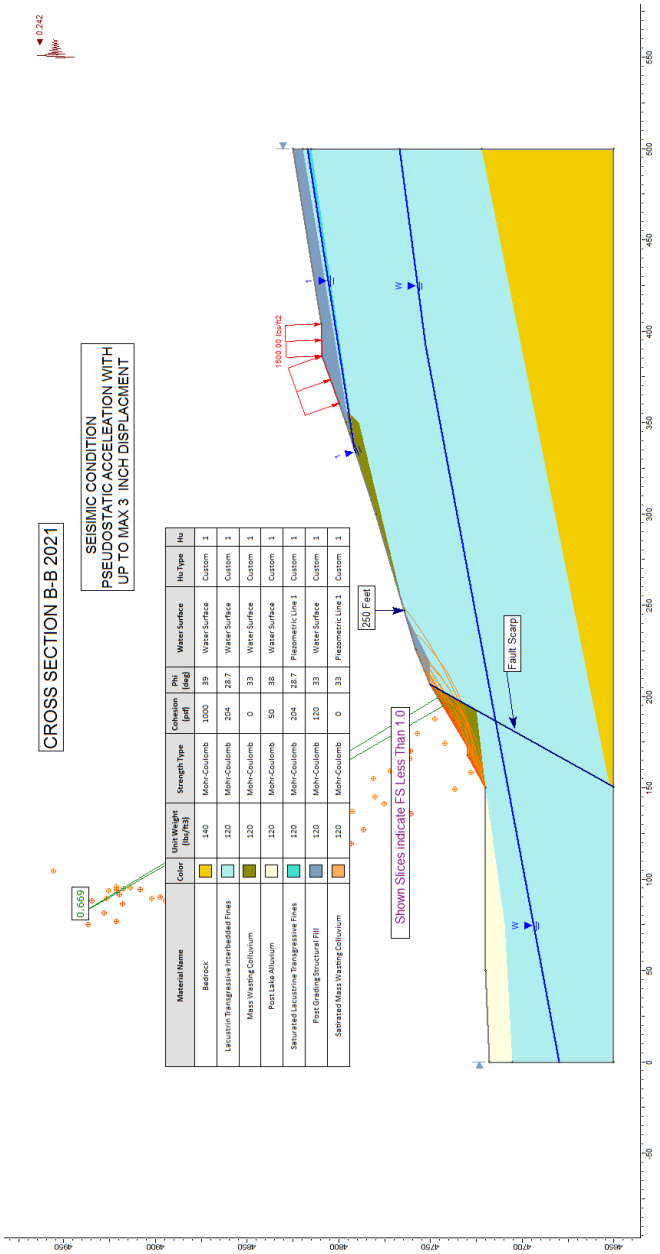
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: B-B
Seismic Condition
Pseudostatic Acceleration with Up to 4-Inch Displacement



PROJECT NO.: 12188		**FIGURE NO.:** 36-2

STABILITY RESULTS

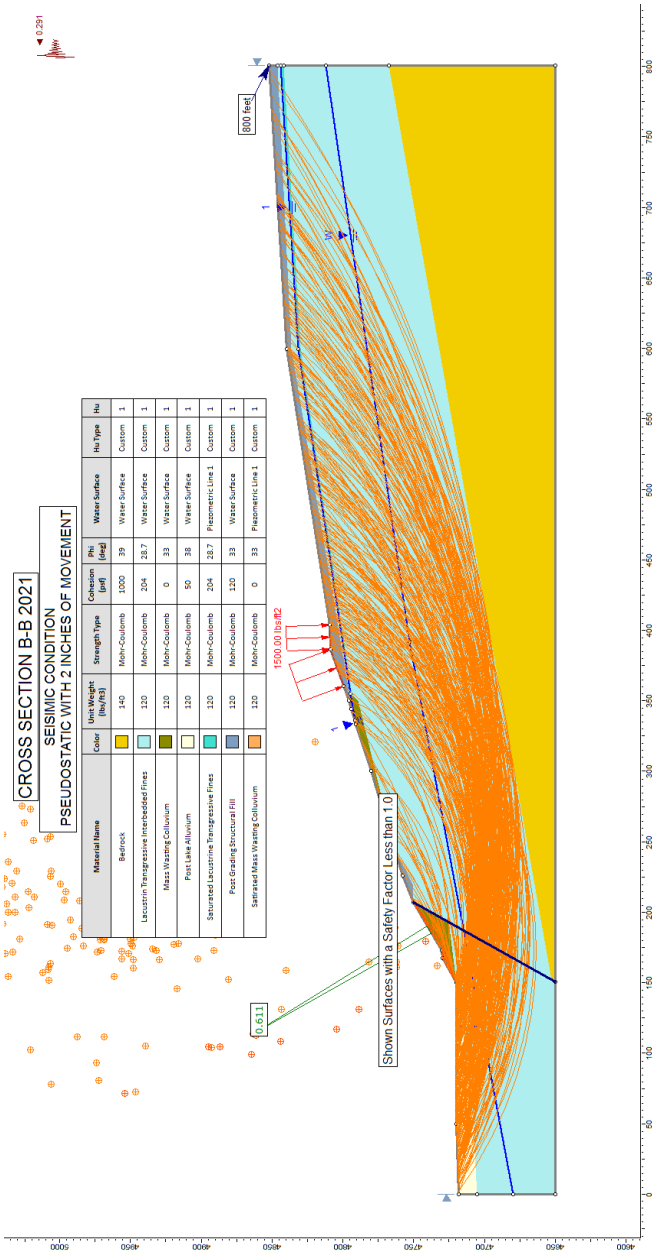
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: B-B
Seismic Condition
Pseudostatic Acceleration with Up to 3-Inch Displacement



PROJECT NO.: 12188		**FIGURE NO.:** 36-3

STABILITY RESULTS

Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: B-B
Seismic Condition
Pseudostatic Acceleration with Up to 2-Inch Displacement



PROJECT NO.: 12188		**FIGURE NO.:** 36-4

STABILITY RESULTS

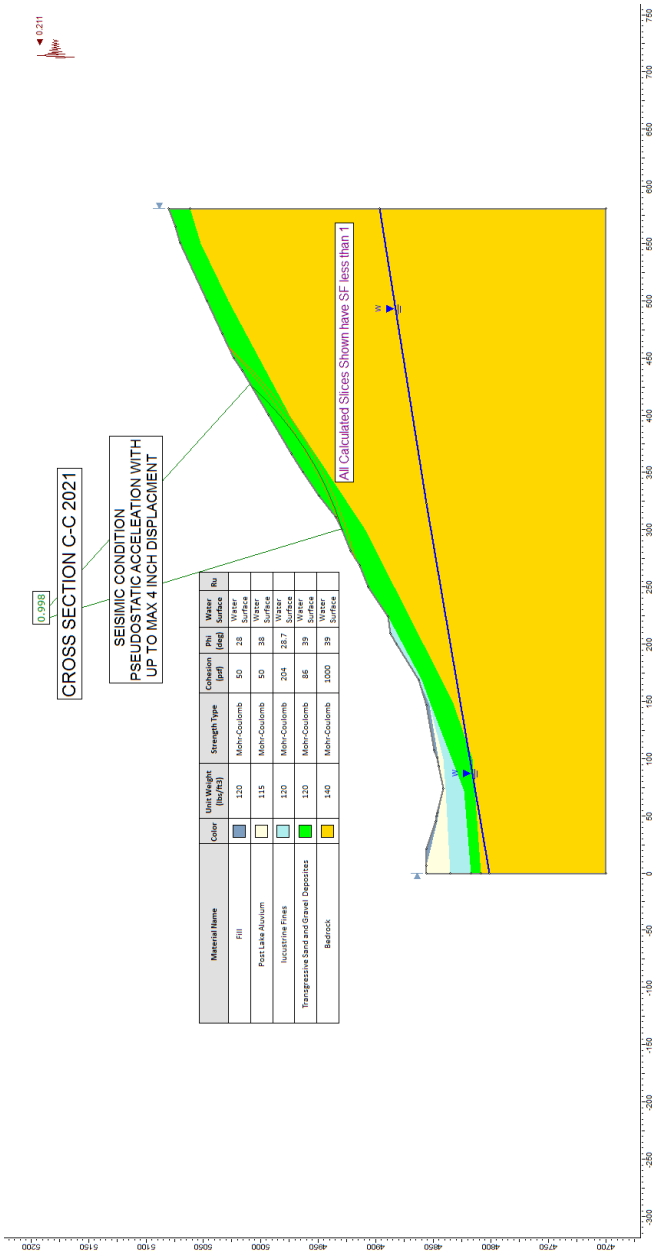
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: C-C
Static Condition



PROJECT NO.: 12188		**FIGURE NO.:** 37-1

STABILITY RESULTS

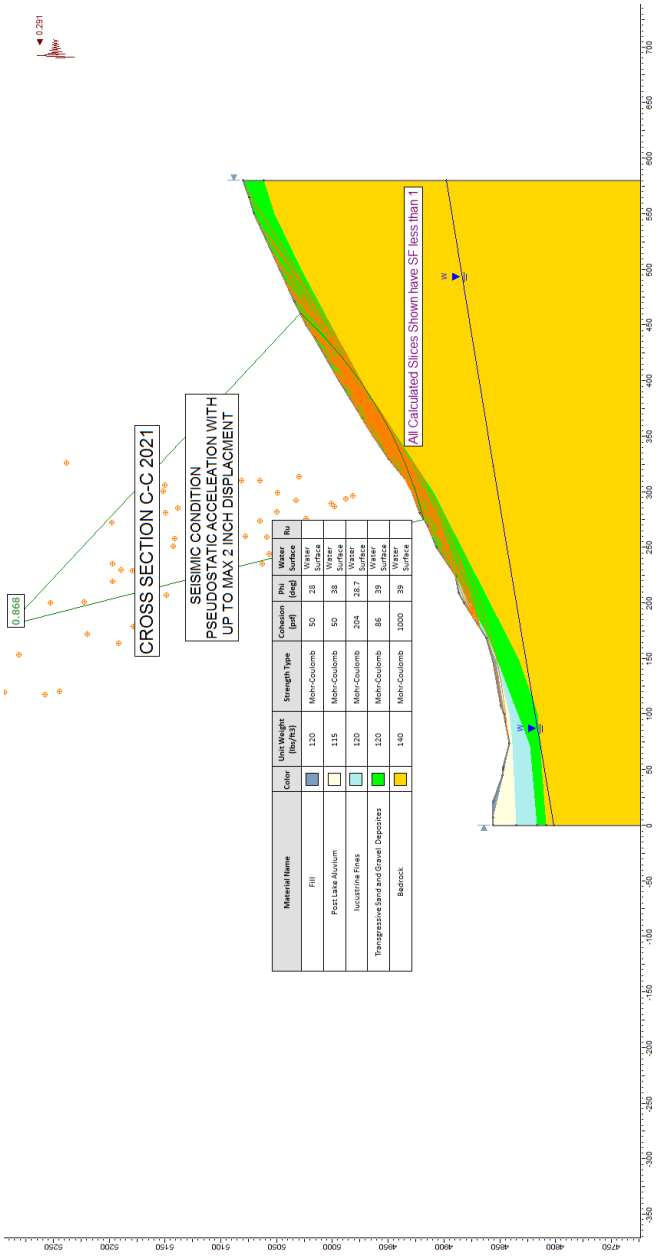
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: C-C
Seismic Condition
Pseudostatic Acceleration with Up to 4-Inch Displacement



PROJECT NO.: 12188		**FIGURE NO.:** 37-2

STABILITY RESULTS

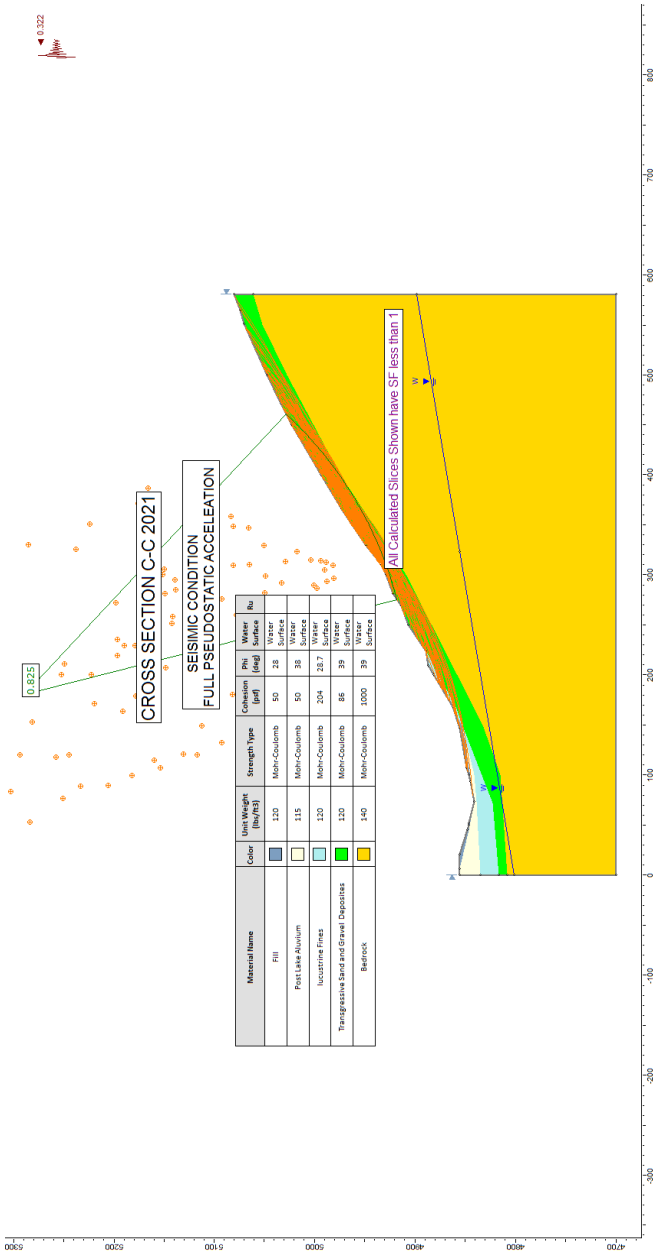
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: C-C
Seismic Condition
Pseudostatic Acceleration with Up to 2-Inch Displacement



PROJECT NO.: 12188		**FIGURE NO.:** 37-3

STABILITY RESULTS

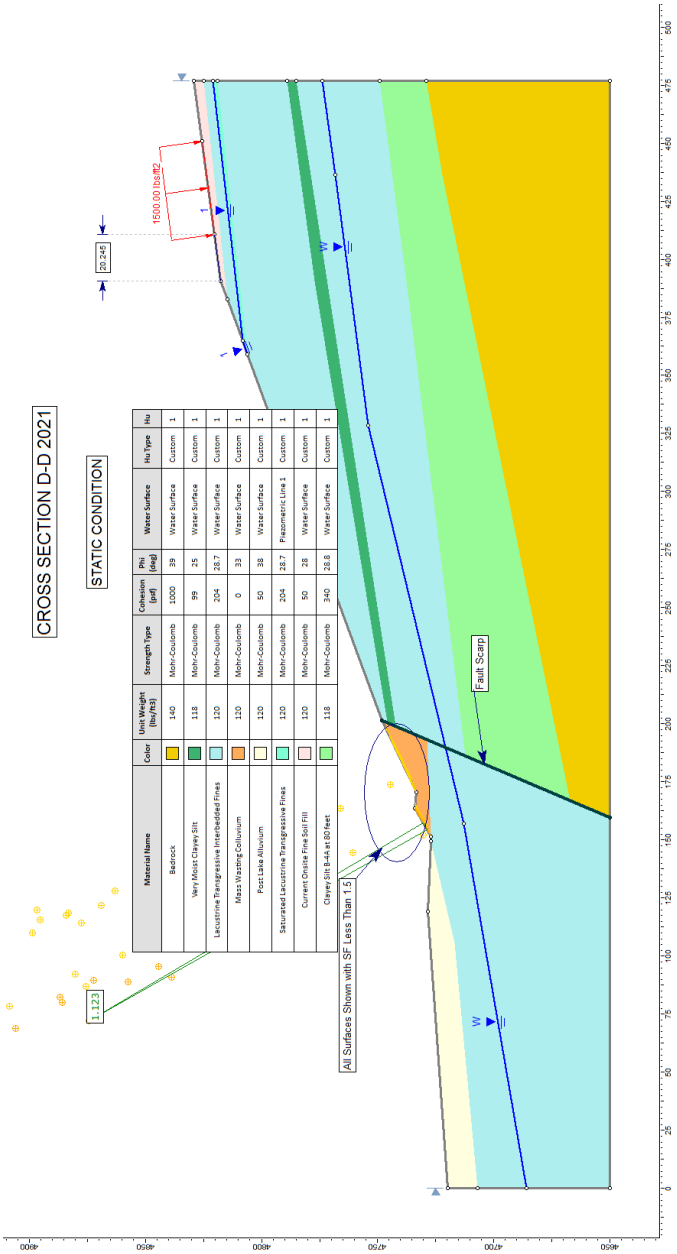
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: C-C
Seismic Condition
Full Pseudostatic Acceleration



PROJECT NO.: 12188		**FIGURE NO.:** 37-4

STABILITY RESULTS

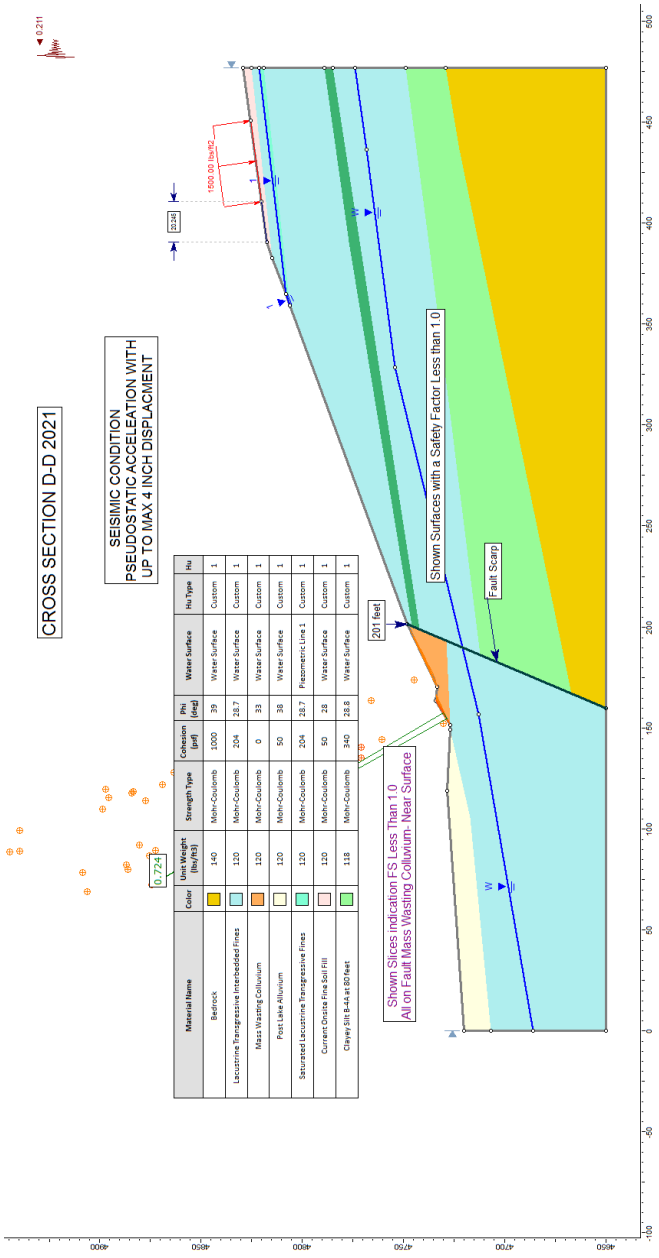
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: D-D
Static Condition



PROJECT NO.: 12188		**FIGURE NO.:** 38-1

STABILITY RESULTS

Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: D-D
Seismic Condition
Pseudostatic Acceleration with Up to 4-Inch Displacement

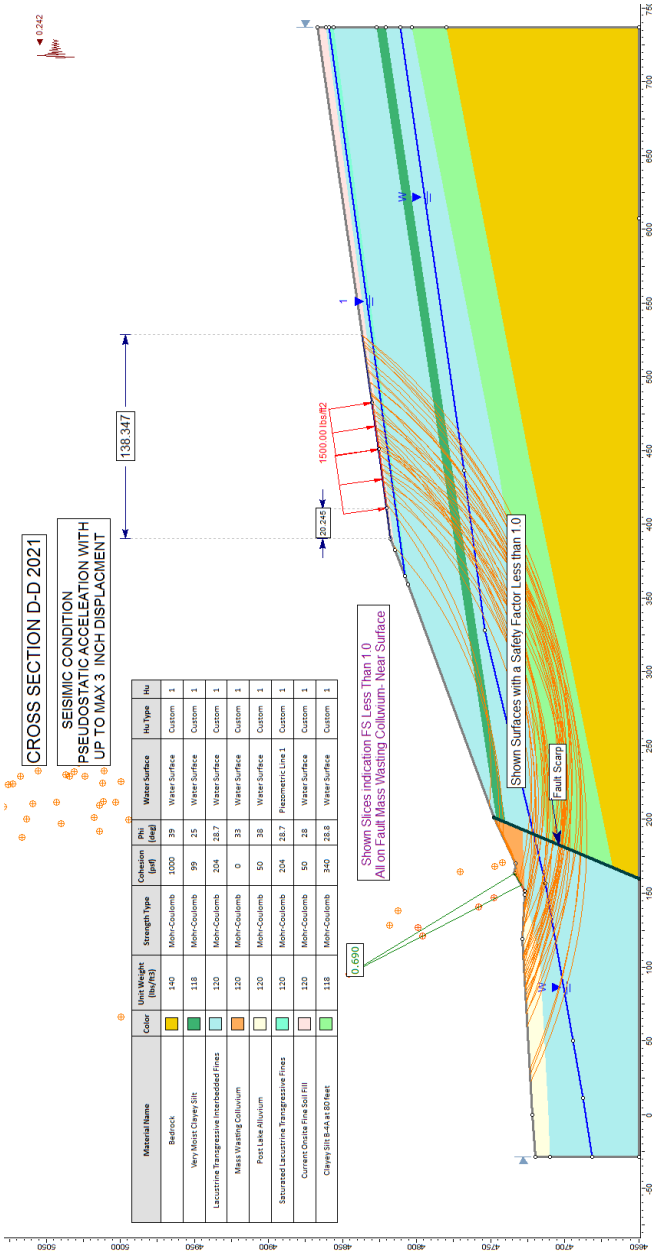


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Bedrock		140	Mohr-Coulomb	1000	39	Water Surface	Custom	1
Lacustrine Transgressive Interbedded Fines		120	Mohr-Coulomb	204	28.7	Water Surface	Custom	1
Mass Wasting Colluvium		120	Mohr-Coulomb	0	33	Water Surface	Custom	1
Post Lake Alluvium		120	Mohr-Coulomb	50	38	Water Surface	Custom	1
Saturated Lacustrine Transgressive Fines		120	Mohr-Coulomb	204	28.7	Piezometric Line 1	Custom	1
Current Onsite Fine Soil Fill		120	Mohr-Coulomb	50	28	Water Surface	Custom	1
Clayey Silt B-4A at 80 Feet		118	Mohr-Coulomb	340	28.8	Water Surface	Custom	1

PROJECT NO.: 12188		**FIGURE NO.:** 38-2

STABILITY RESULTS

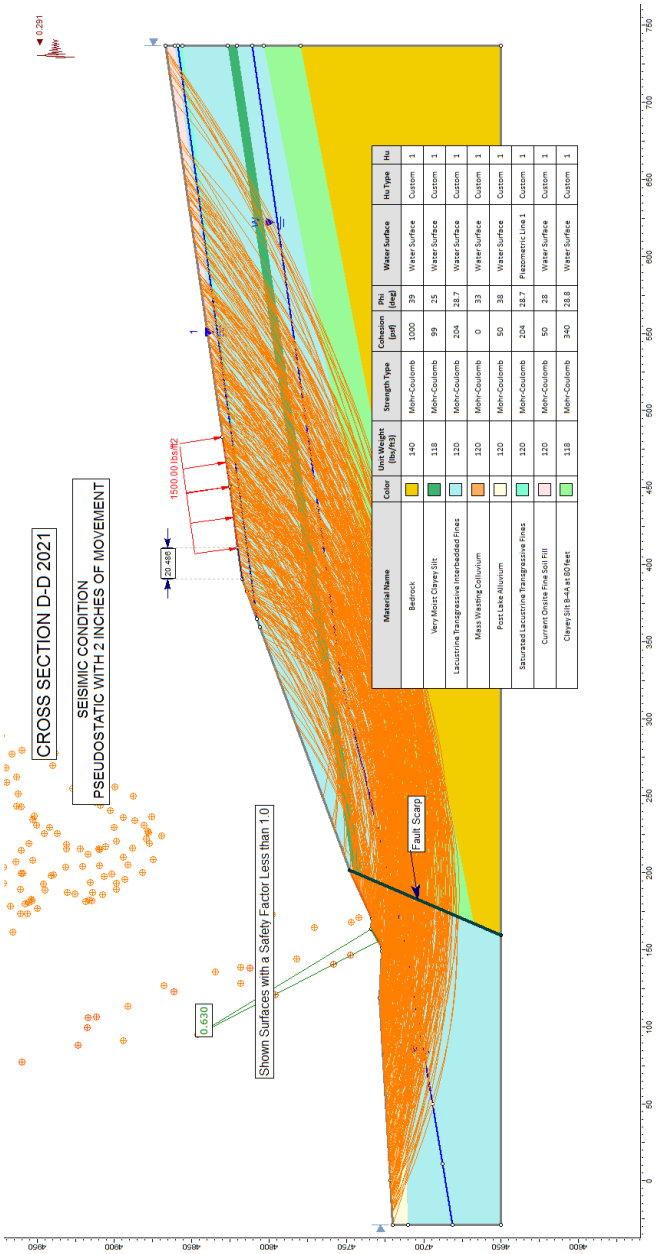
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: D-D
Seismic Condition
Pseudostatic Acceleration with Up to 3-Inch Displacement



PROJECT NO.: 12188		**FIGURE NO.:** 38-3

STABILITY RESULTS

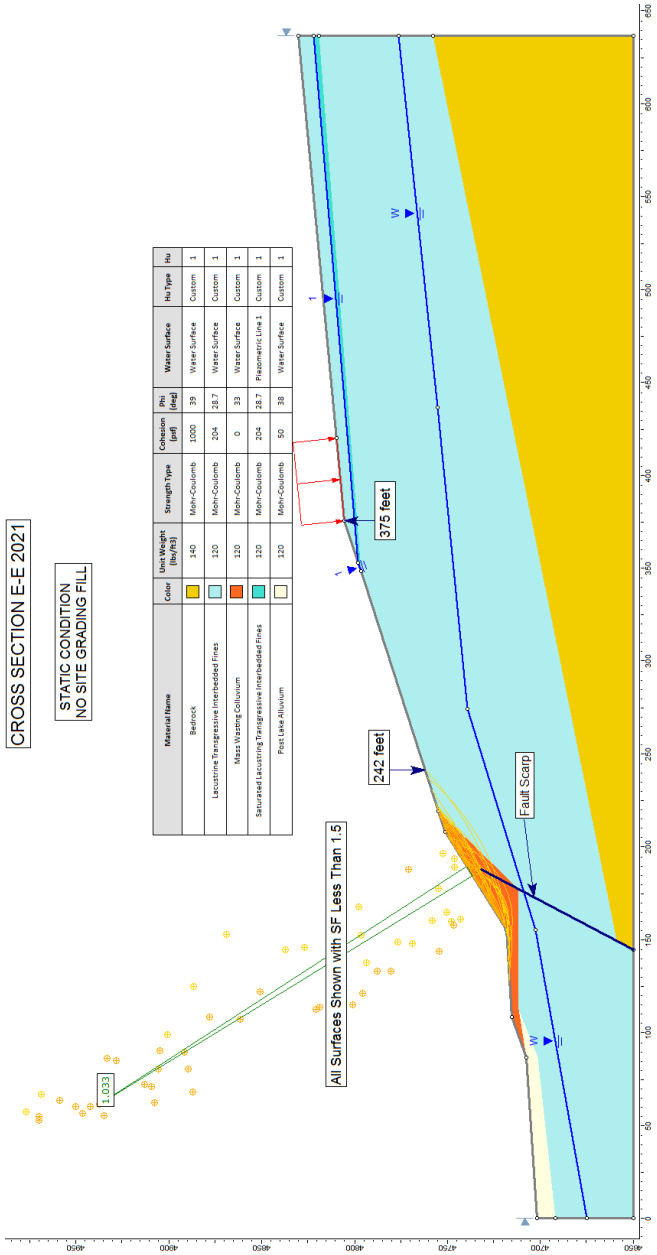
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: D-D
Seismic Condition
Pseudostatic Acceleration with Up to 2-Inch Displacement



PROJECT NO.: 12188		**FIGURE NO.:** 38-4

STABILITY RESULTS

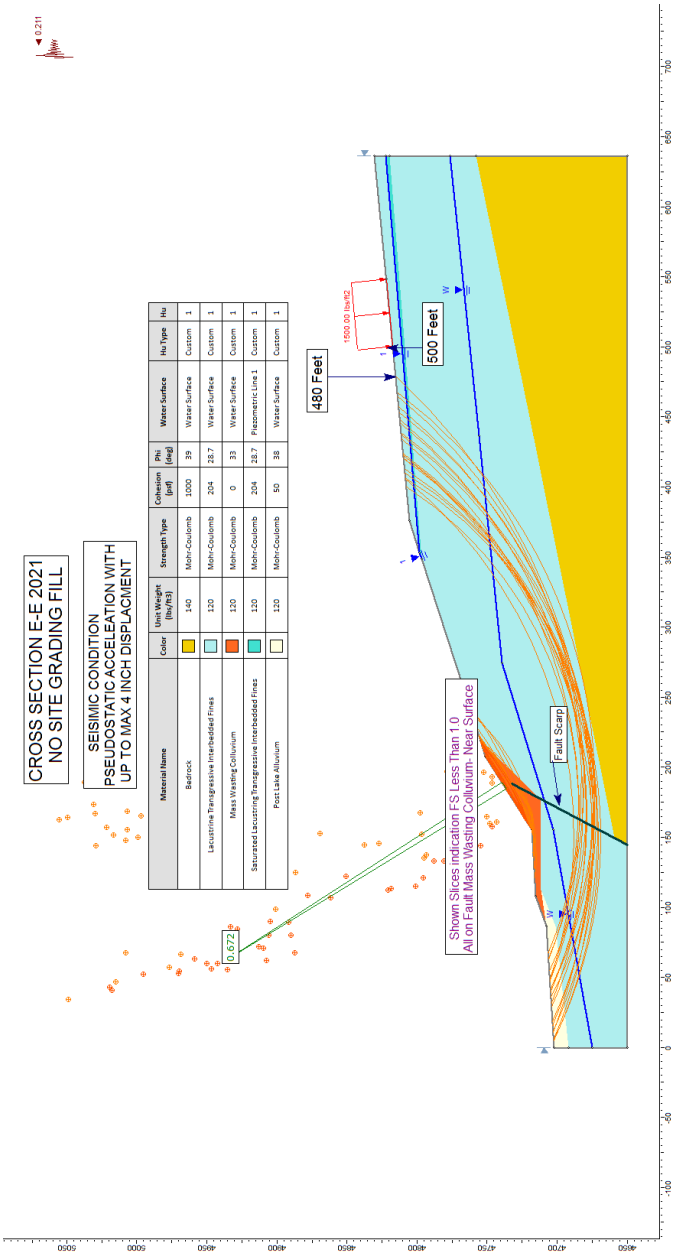
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: E-E
Static Condition



PROJECT NO.: 12188		**FIGURE NO.:** 39-1

STABILITY RESULTS

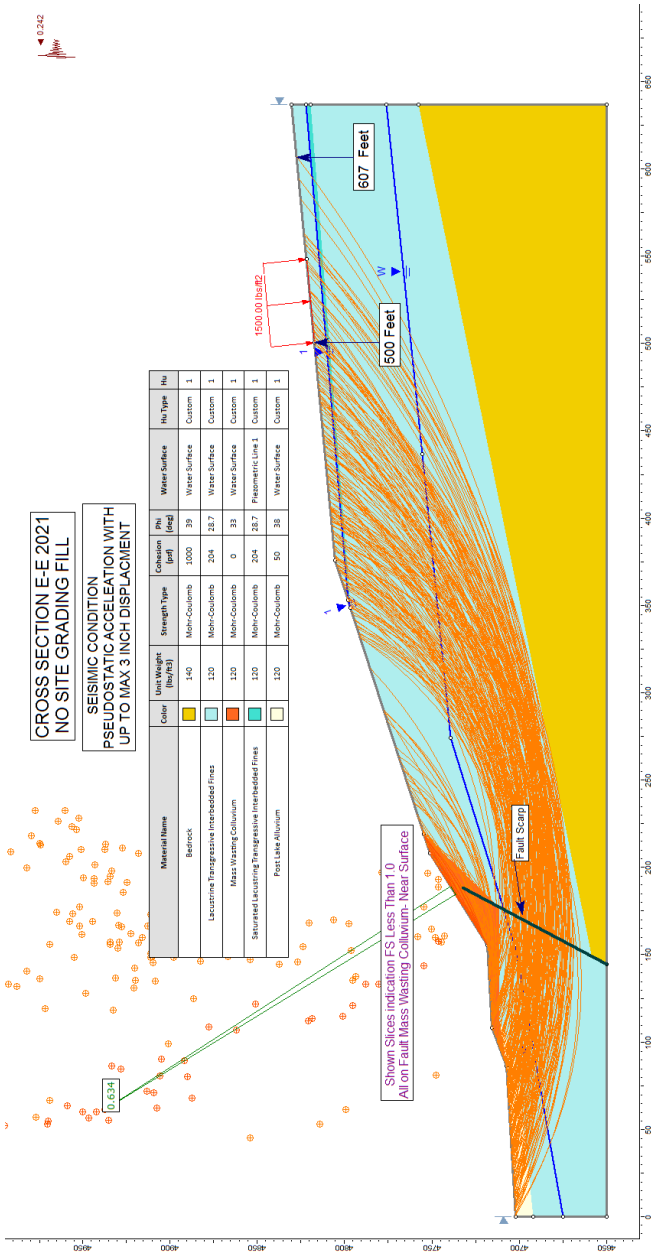
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: E-E
Seismic Condition
Pseudostatic Acceleration with Up to 4-Inch Displacement



PROJECT NO.: 12188		**FIGURE NO.:** 39-2

STABILITY RESULTS

Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: E-E
Seismic Condition
Pseudostatic Acceleration with Up to 3-Inch Displacement

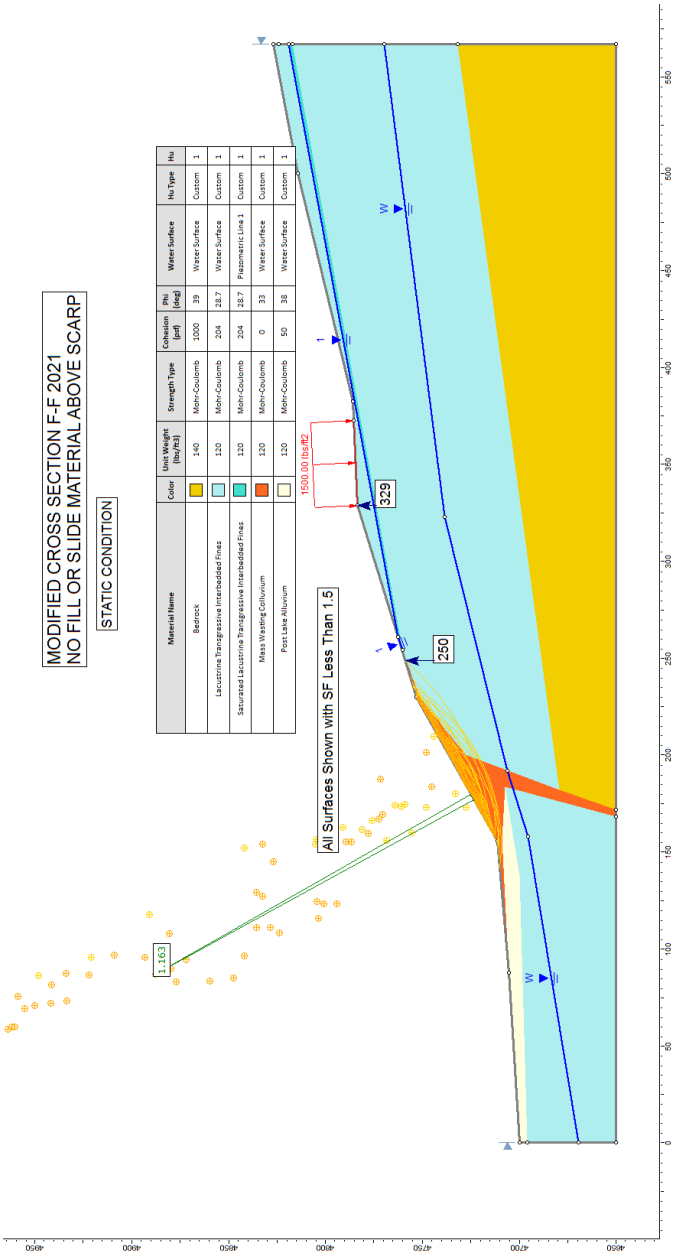


Material Name	Color	Unit Weight [lbs/ft3]	Strength Type	Cohesion [psf]	Phi [deg]	Water Surface	Hu Type	Hu
Bedrock		140	Mohr-Coulomb	1000	39	Water Surface	Custom	1
Lacustrine Transgressive Interbedded Fines		120	Mohr-Coulomb	204	28.7	Water Surface	Custom	1
Mass Wasting Colluvium		120	Mohr-Coulomb	0	33	Water Surface	Custom	1
Saturated Lacustrine Transgressive Interbedded Fines		120	Mohr-Coulomb	204	28.7	Piezometric Line 1	Custom	1
Post Lake Alluvium		120	Mohr-Coulomb	50	38	Water Surface	Custom	1

PROJECT NO.: 12188		**FIGURE NO.:** 39-3

STABILITY RESULTS

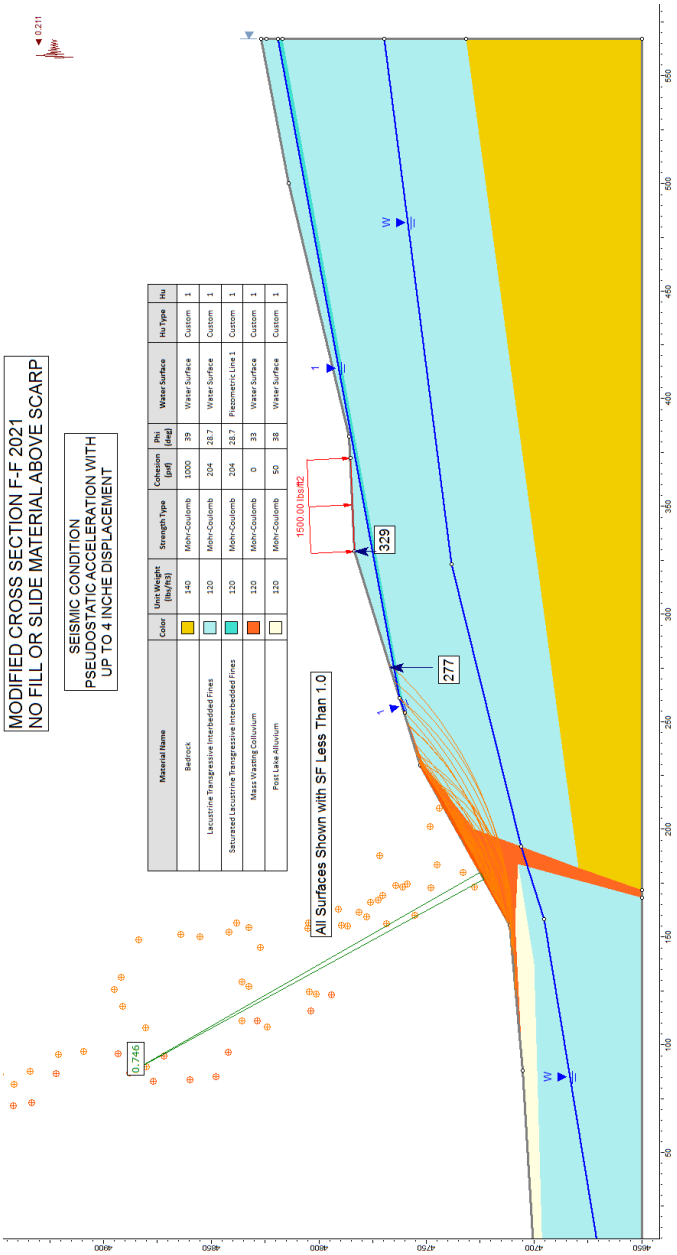
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: F-F
Static Condition



PROJECT NO.: 12188		**FIGURE NO.:** 40-1

STABILITY RESULTS

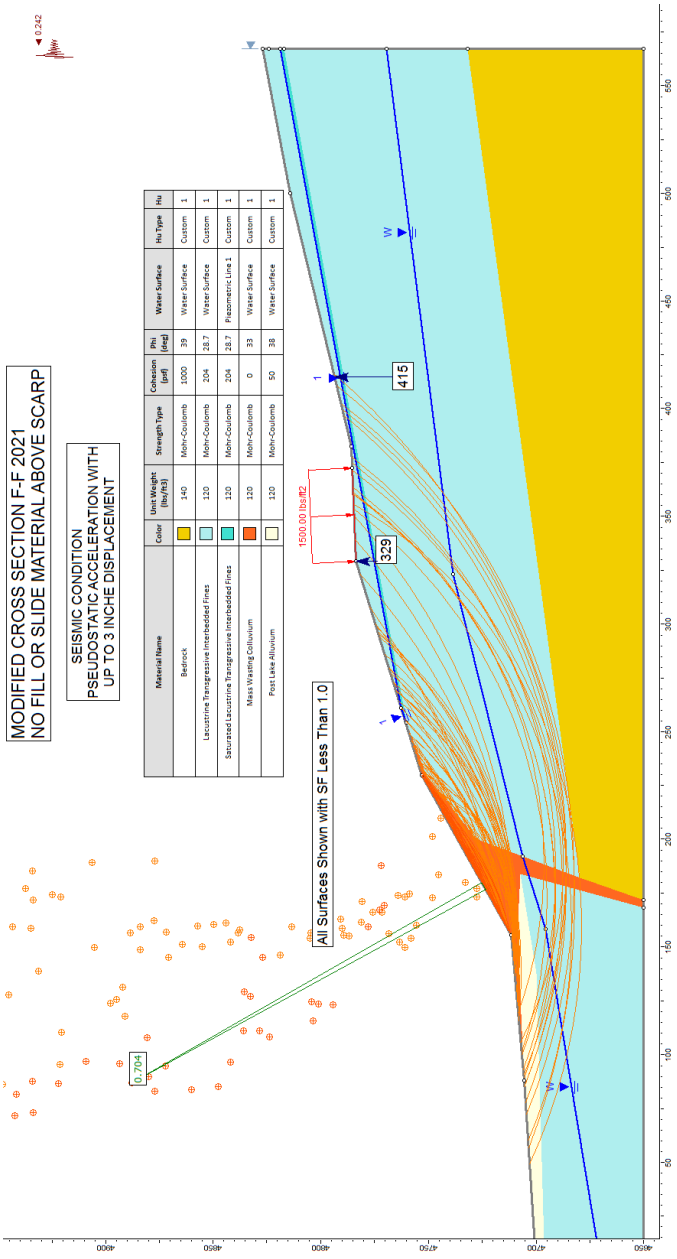
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: F-F
Seismic Condition
Pseudostatic Acceleration with Up to 4-Inch Displacement



PROJECT NO.: 12188		**FIGURE NO.:** 40-2

STABILITY RESULTS

Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: F-F
Seismic Condition
Pseudostatic Acceleration with Up to 3-Inch Displacement

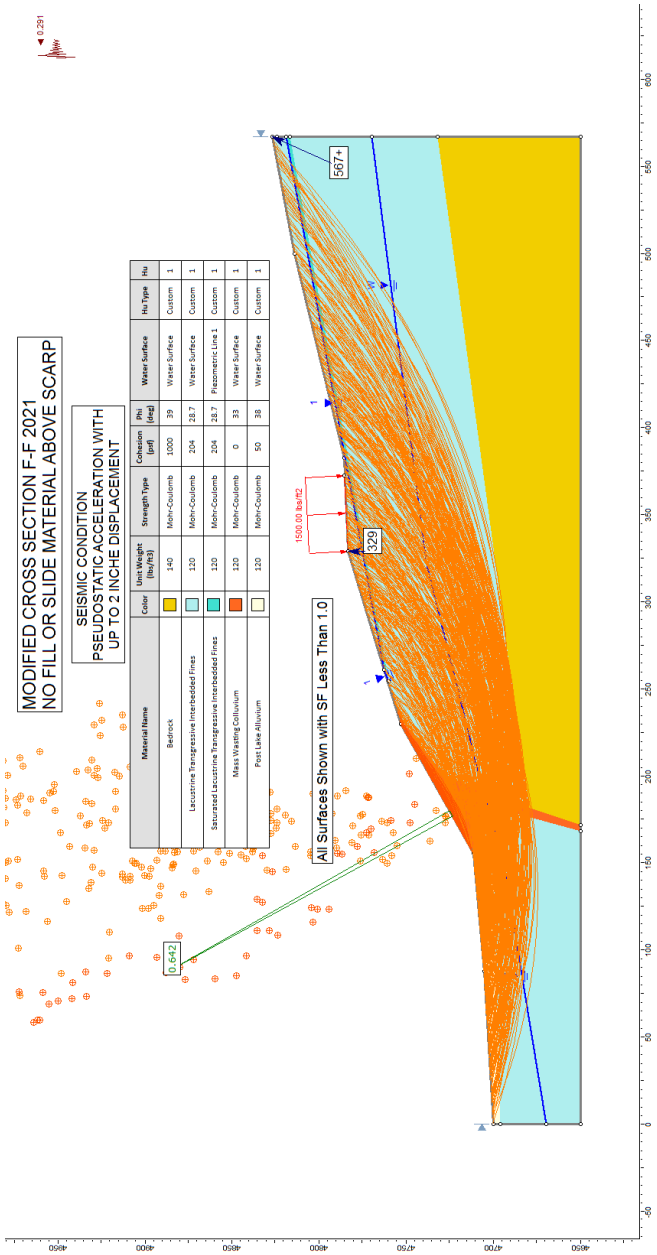


Material Name	Color	Unit Weight [lbs/ft3]	Strength Type	Cohesion [psf]	Phi [deg]	Water Surface	Hu Type	Hu
Bedrock		140	Mohr-Coulomb	1000	39	Water Surface	Custom	1
Lacustrine Transgressive Interbedded Fines		120	Mohr-Coulomb	204	28.7	Water Surface	Custom	1
Saturated Lacustrine Transgressive Interbedded Fines		120	Mohr-Coulomb	204	28.7	Piezometric Line 1	Custom	1
Mass Wasting Colluvium		120	Mohr-Coulomb	0	33	Water Surface	Custom	1
Post Lake Alluvium		120	Mohr-Coulomb	50	38	Water Surface	Custom	1

PROJECT NO.: 12188		**FIGURE NO.:** 40-3

STABILITY RESULTS

Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: F-F
Seismic Condition
Pseudostatic Acceleration with Up to 2-Inch Displacement

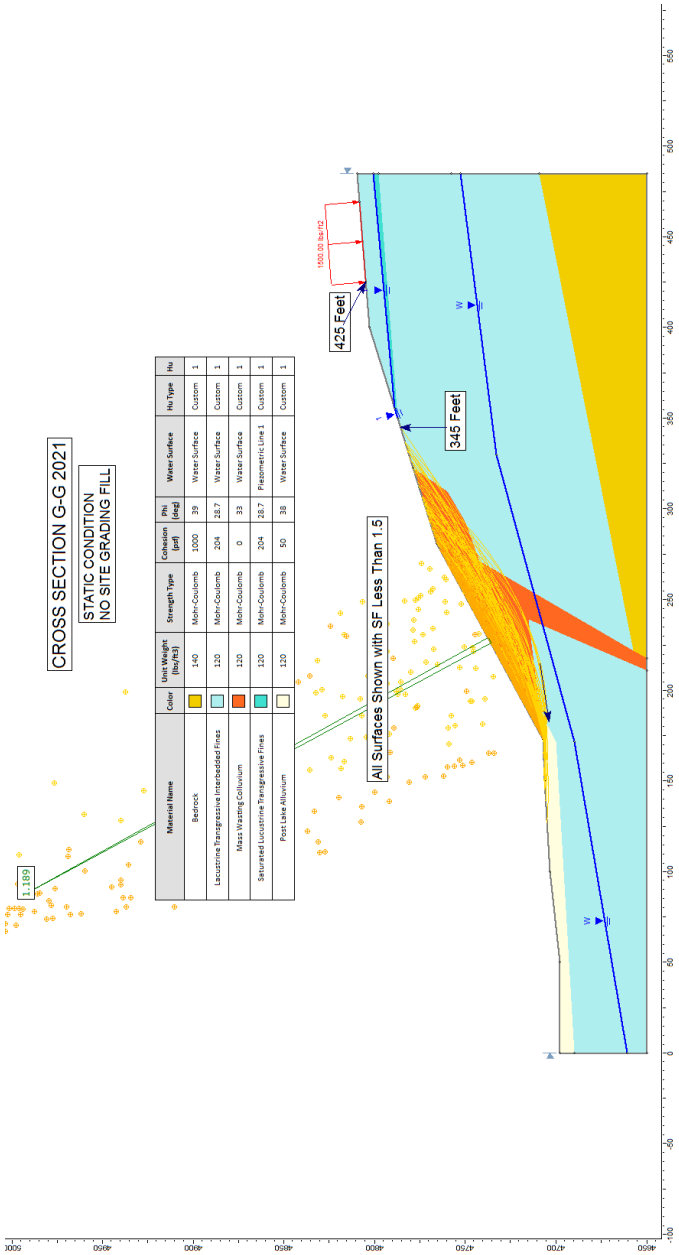


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Bedrock		140	Mohr-Coulomb	1000	39	Water Surface	Custom	1
Lacustrine Transgressive Interbedded Fines		120	Mohr-Coulomb	204	28.7	Water Surface	Custom	1
Saturated Lacustrine Transgressive Interbedded Fines		120	Mohr-Coulomb	204	28.7	Piezometric Line 1	Custom	1
Mass Wasting Colluvium		120	Mohr-Coulomb	0	33	Water Surface	Custom	1
Post Lake Alluvium		120	Mohr-Coulomb	50	38	Water Surface	Custom	1

PROJECT NO.: 12188		**FIGURE NO.:** 40-4

STABILITY RESULTS

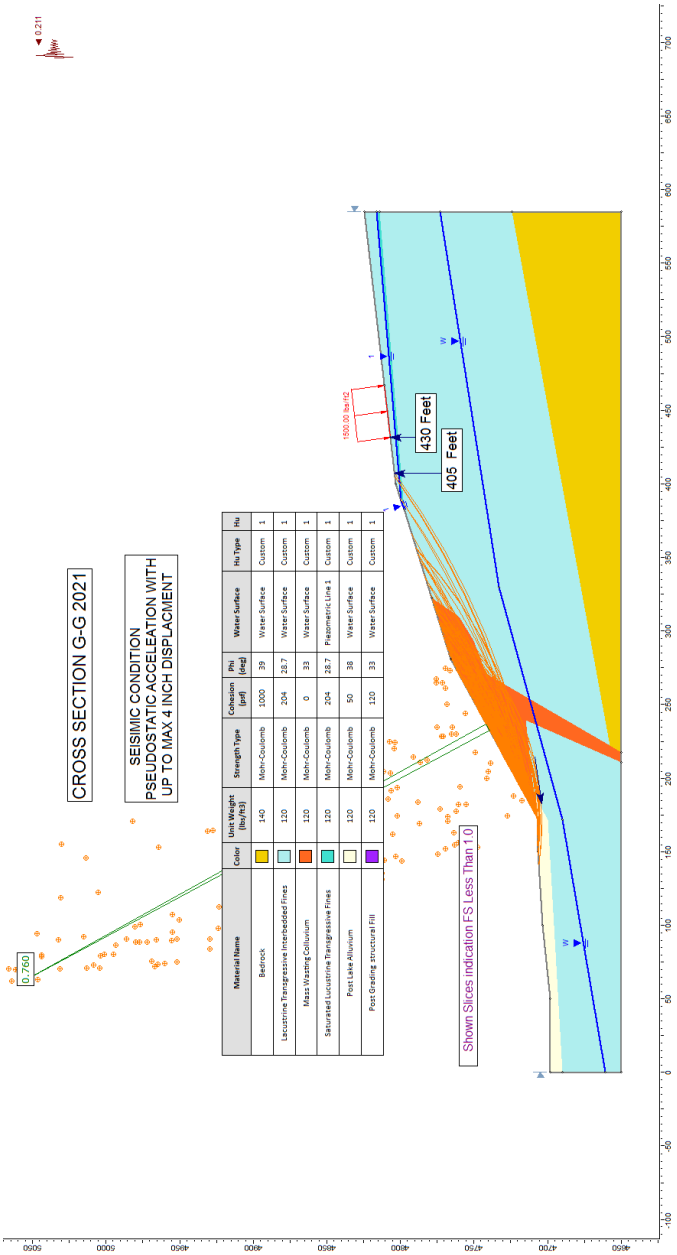
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: G-G
Static Condition



PROJECT NO.: 12188		**FIGURE NO.:** 41-1

STABILITY RESULTS

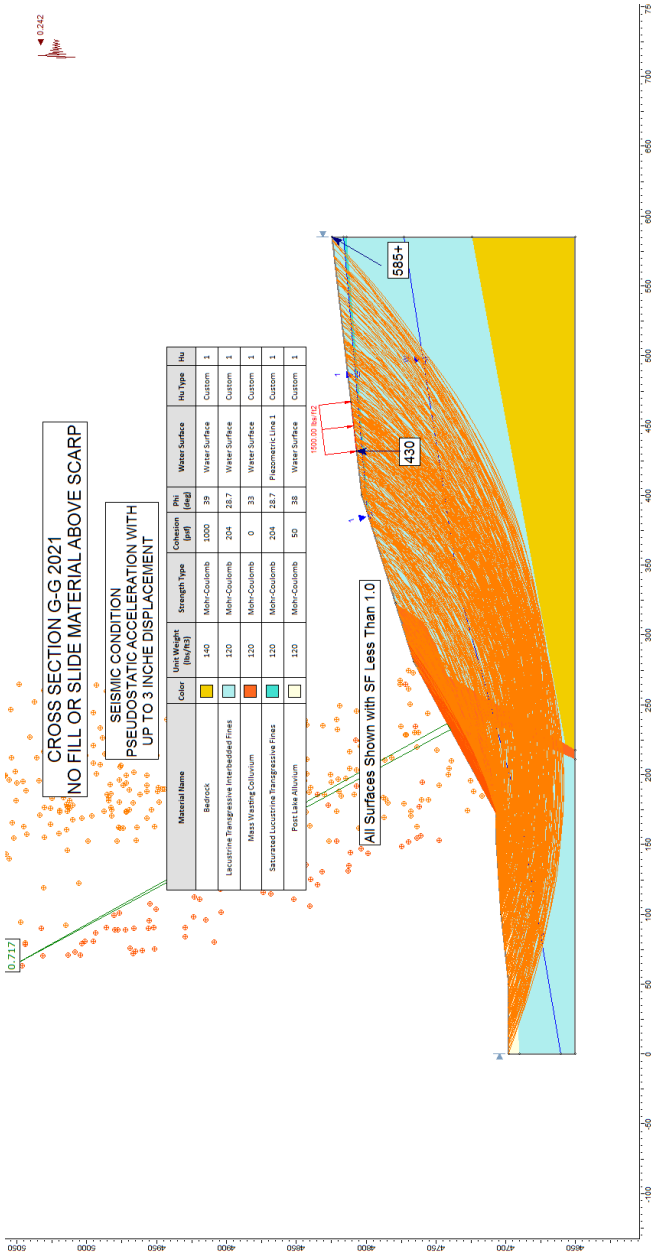
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: G-G
Seismic Condition
Pseudostatic Acceleration with Up to 4-Inch Displacement



PROJECT NO.: 12188		**FIGURE NO.:** 41-2

STABILITY RESULTS

Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: G-G
Seismic Condition
Pseudostatic Acceleration with Up to 3-Inch Displacement

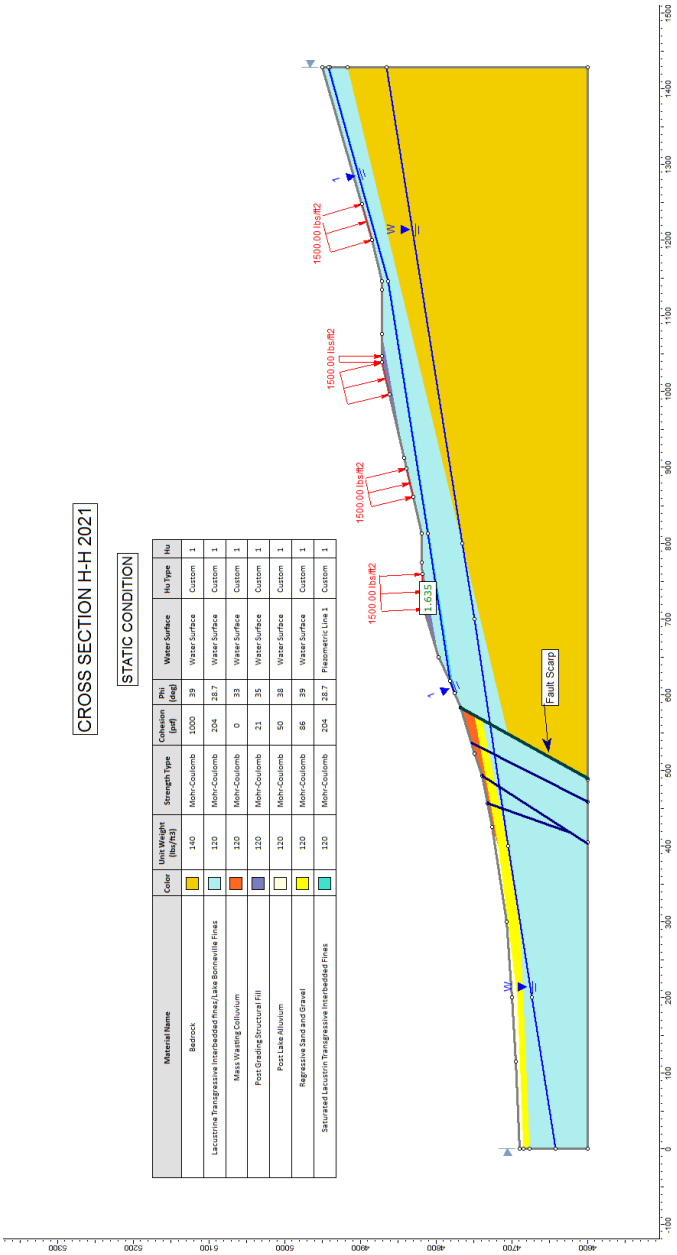


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Bedrock		140	Mohr-Coulomb	1000	39	Water Surface	Custom	1
Lacustrine Transgressive Interbedded Fines		120	Mohr-Coulomb	204	28.7	Water Surface	Custom	1
Mass Wasting Colluvium		120	Mohr-Coulomb	0	33	Water Surface	Custom	1
Saturated Lacustrine Transgressive Fines		120	Mohr-Coulomb	204	28.7	Piezometric Line 1	Custom	1
Post Lake Alluvium		120	Mohr-Coulomb	50	38	Water Surface	Custom	1

PROJECT NO.: 12188		**FIGURE NO.:** 41-3

STABILITY RESULTS

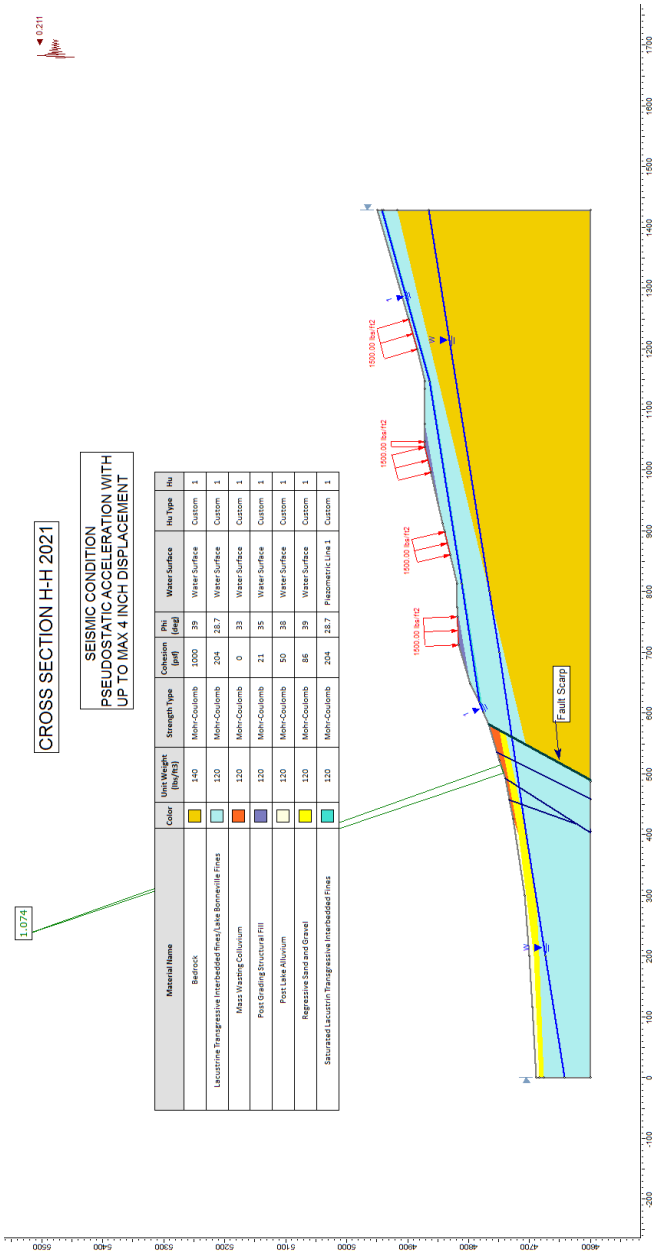
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: H-H
Static Condition



PROJECT NO.: 12188		**FIGURE NO.:** 42-1

STABILITY RESULTS

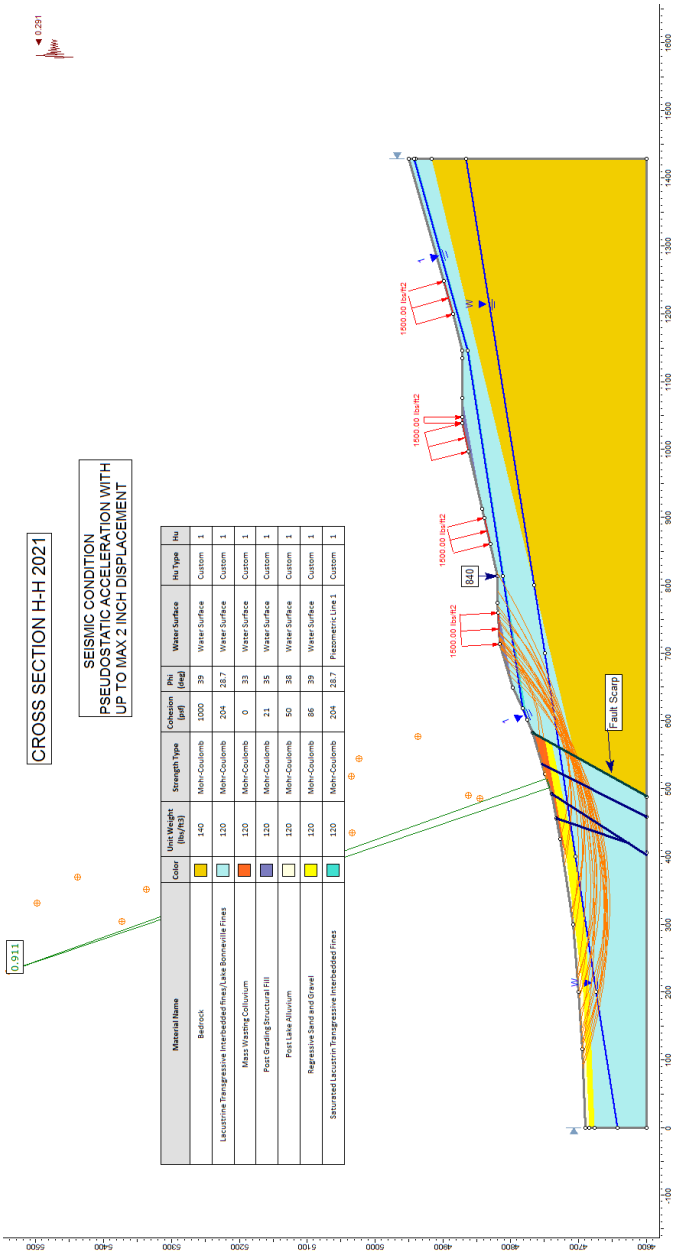
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: H-H
Seismic Condition
Pseudostatic Acceleration with Up to 4-Inch Displacement



PROJECT NO.: 12188		**FIGURE NO.:** 42-2

STABILITY RESULTS

Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: H-H
Seismic Condition
Pseudostatic Acceleration with Up to 2-Inch Displacement

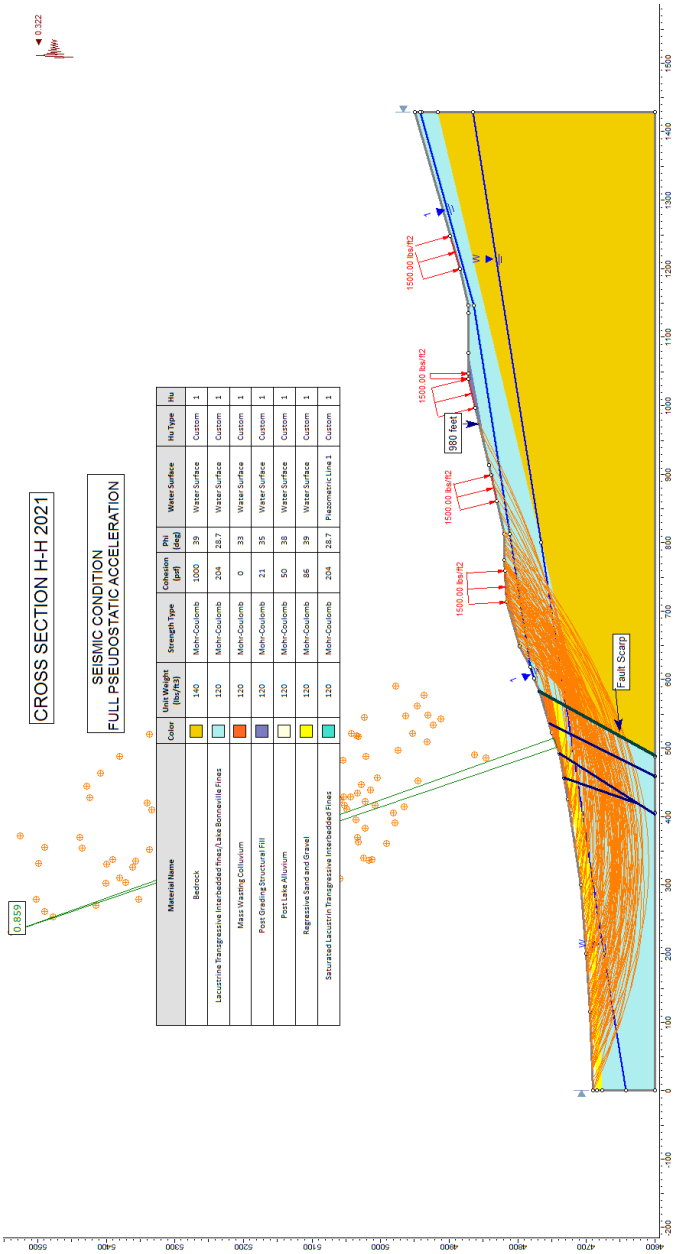


Material Name	Color	Unit Weight [lbs/ft3]	Strength Type	Cohesion [psf]	Phi [deg]	Water Surface	Hu Type	Hu
Bedrock		140	Mohr-Coulomb	1000	39	Water Surface	Custom	1
Lacustrine Transgressive Interbedded Fines/Lake Bonneville Fines		120	Mohr-Coulomb	204	28.7	Water Surface	Custom	1
Mass Wasting Colluvium		120	Mohr-Coulomb	0	33	Water Surface	Custom	1
Post Grading Structural Fill		120	Mohr-Coulomb	21	35	Water Surface	Custom	1
Post Lake Alluvium		120	Mohr-Coulomb	50	38	Water Surface	Custom	1
Regressive Sand and Gravel		120	Mohr-Coulomb	86	39	Water Surface	Custom	1
Saturated Lacustrin Transgressive Interbedded Fines		120	Mohr-Coulomb	204	28.7	Piezometric Line 1	Custom	1

PROJECT NO.: 12188		**FIGURE NO.:** 42-3

STABILITY RESULTS

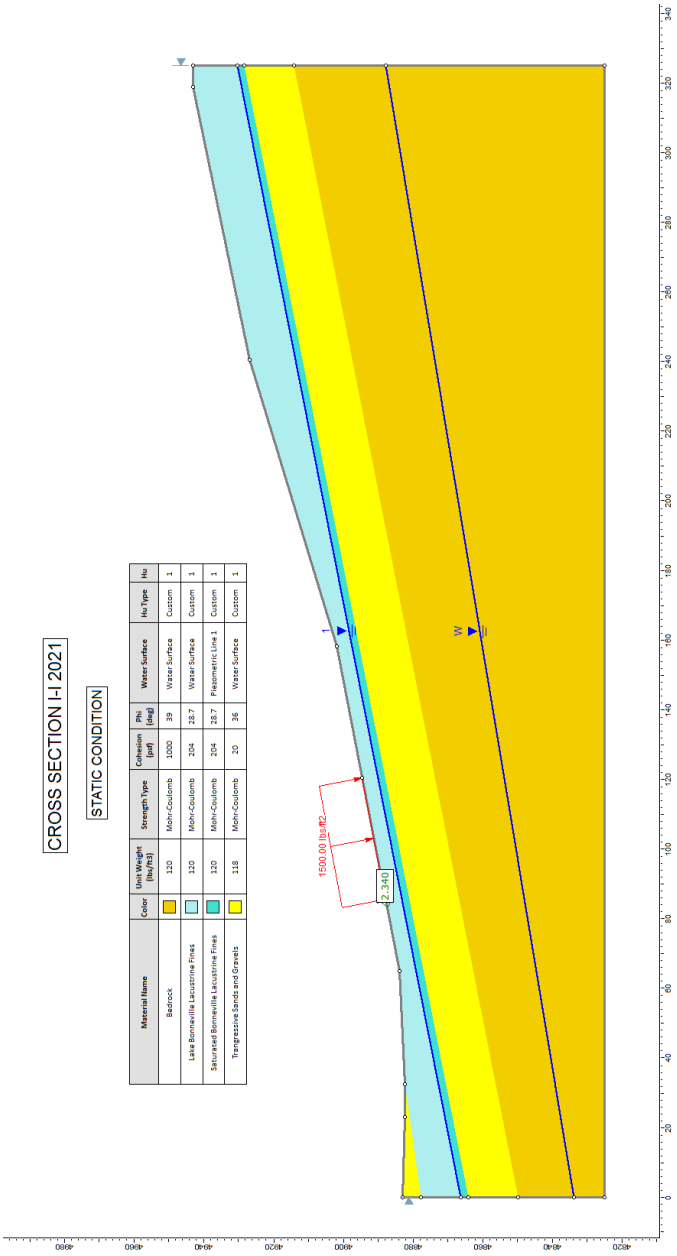
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: H-H
Seismic Condition
Full Pseudostatic Acceleration



PROJECT NO.: 12188		**FIGURE NO.:** 42-4

STABILITY RESULTS

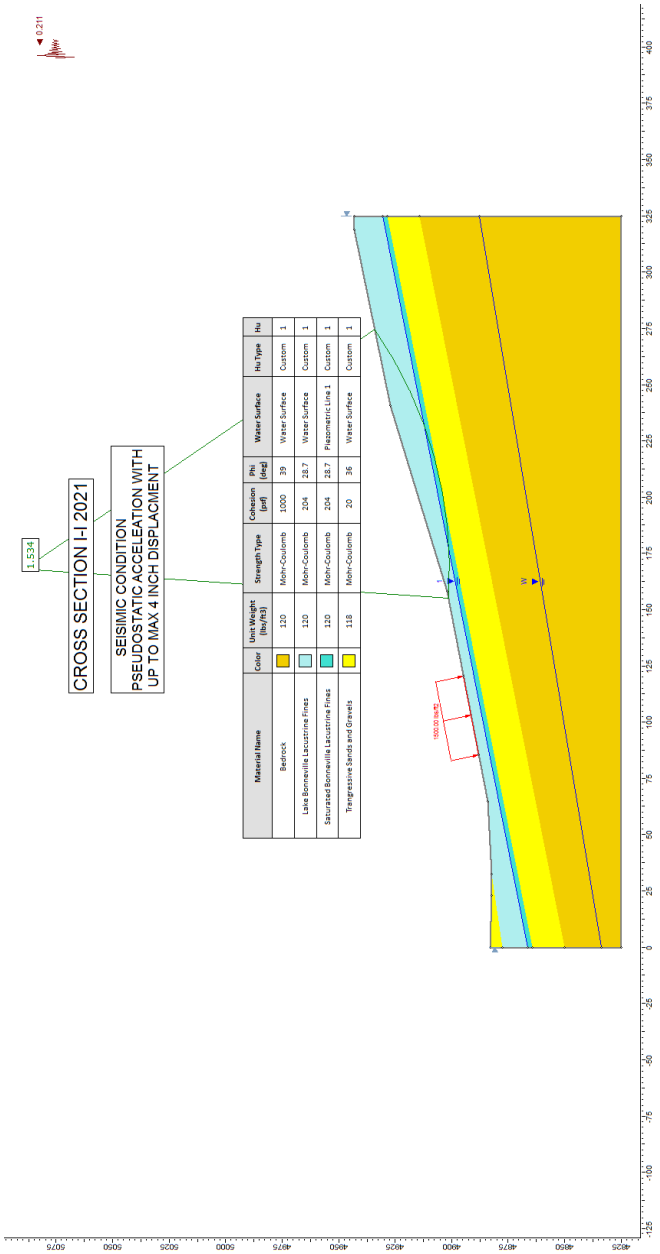
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: I-I
Static Condition



PROJECT NO.: 12188		**FIGURE NO.:** 43-1

STABILITY RESULTS

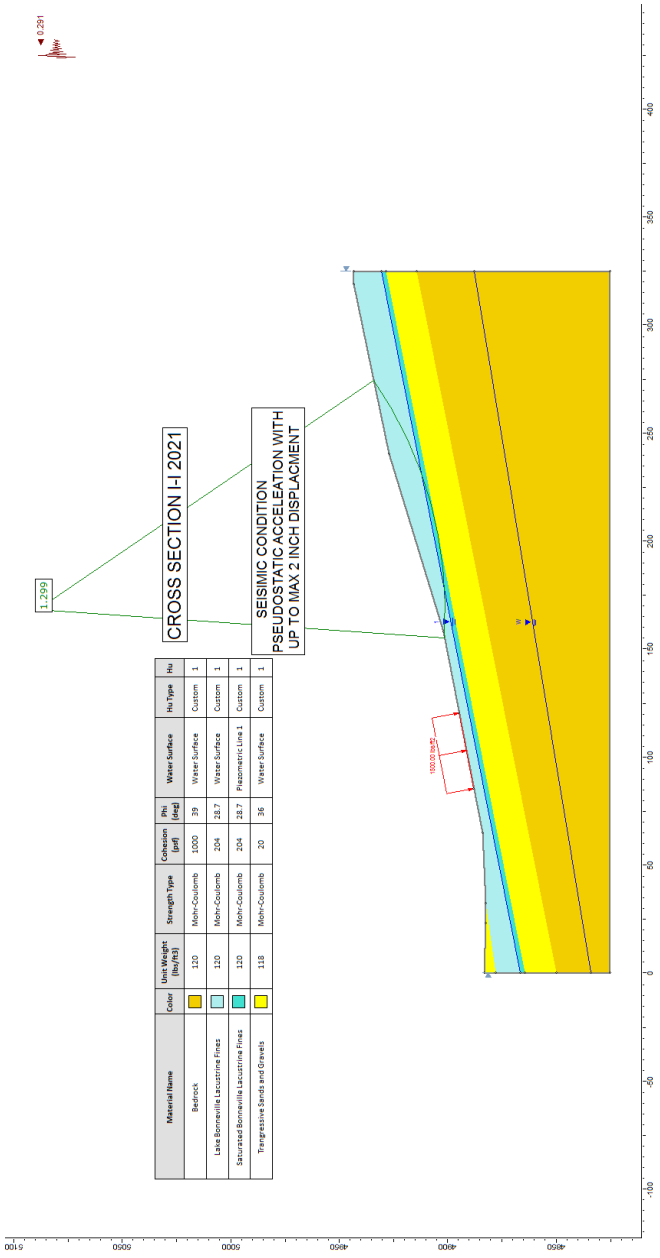
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: I-I
Seismic Condition
Pseudostatic Acceleration with Up to 4-Inch Displacement



PROJECT NO.: 12188		**FIGURE NO.:** 43-2

STABILITY RESULTS

Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: I-I
Seismic Condition
Pseudostatic Acceleration with Up to 2-Inch Displacement

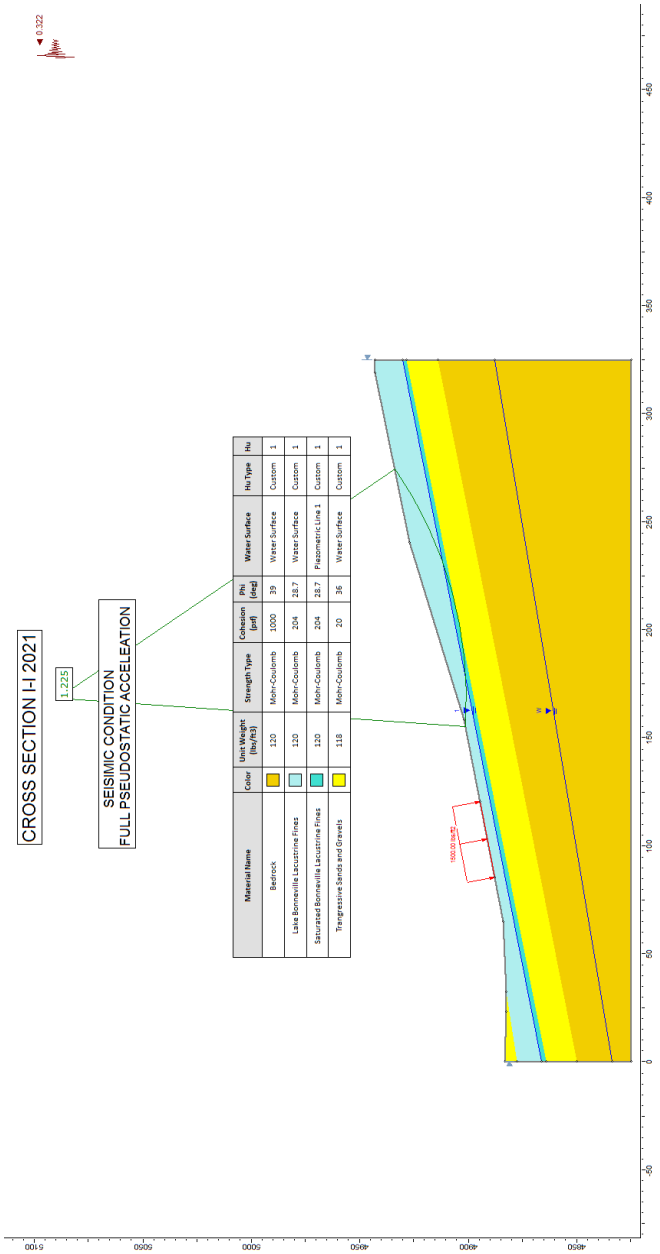


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Bedrock		120	Mohr-Coulomb	1000	39	Water Surface	Custom	1
Lake Bonneville Lacustrine Fines		120	Mohr-Coulomb	204	28.7	Water Surface	Custom	1
Saturated Bonneville Lacustrine Fines		120	Mohr-Coulomb	204	28.7	Piezometric Line 1	Custom	1
Transgressive Sands and Gravels		118	Mohr-Coulomb	20	36	Water Surface	Custom	1

PROJECT NO.: 12188		**FIGURE NO.:** 43-3

STABILITY RESULTS

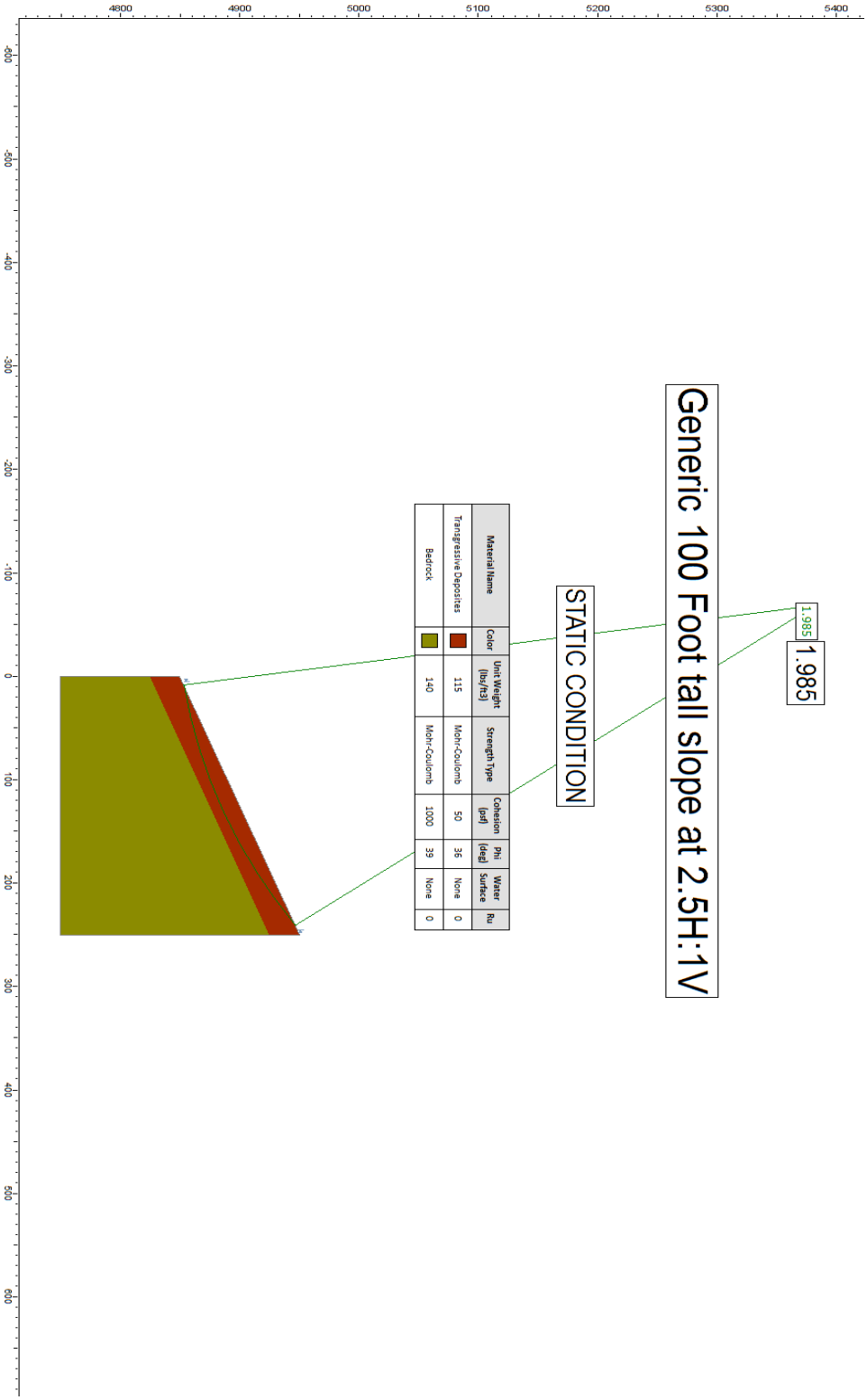
Proposed Rock Loft Ridge Subdivision, Fruit Heights, Utah
Cross Section: I-I
Seismic Condition
Full Pseudostatic Acceleration



PROJECT NO.: 12188		**FIGURE NO.:** 43-4

STABILITY RESULTS

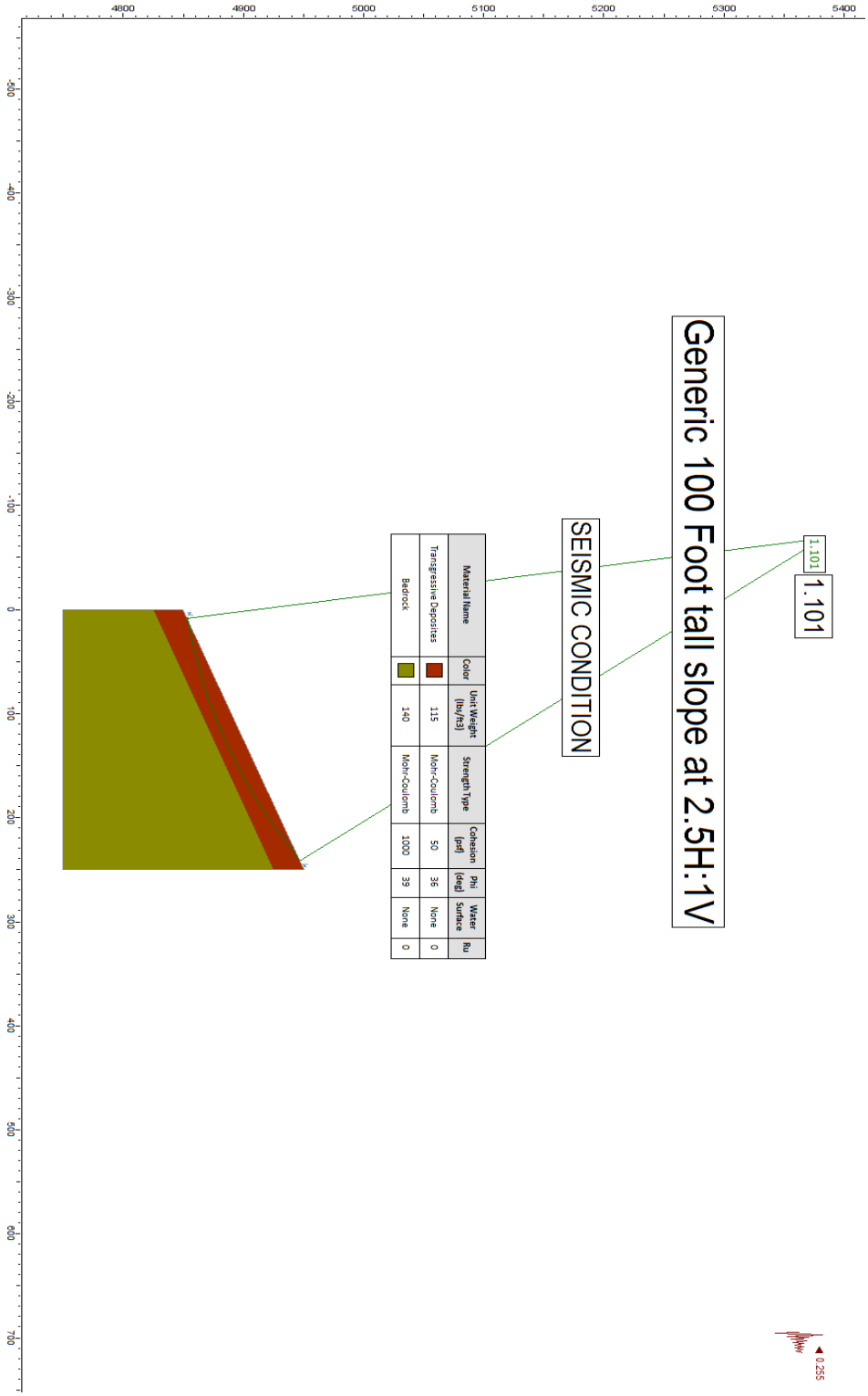
Proposed Rock Loft Ridge Subdivision
Fruit Heights, Utah
Generic 2.5H:1V Cross Section
STATIC



PROJECT NO.: 12188	Rock Loft Subdivision	**FIGURE NO.:** 44A

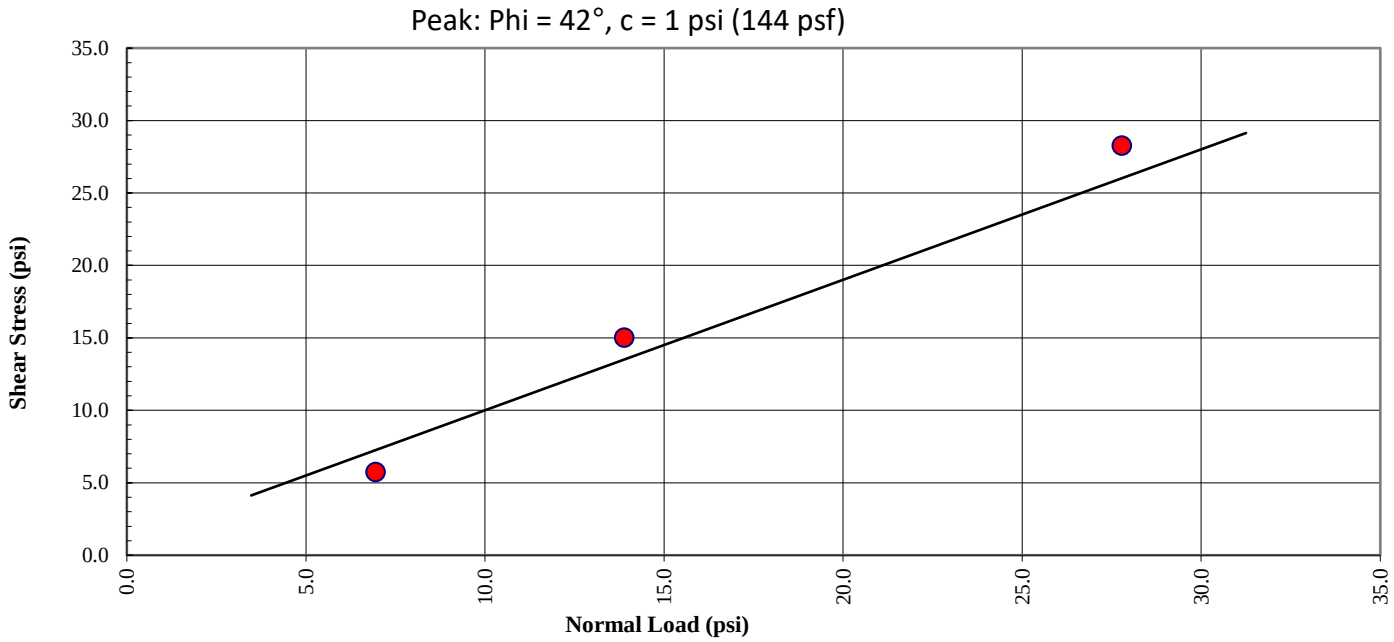
STABILITY RESULTS

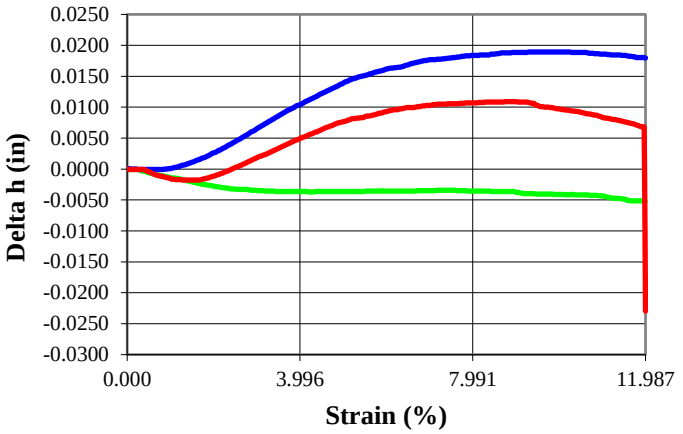
Proposed Rock Loft Ridge Subdivision
Fruit Heights, Utah
Generic 2.5H:1V Cross Section
SEISMIC

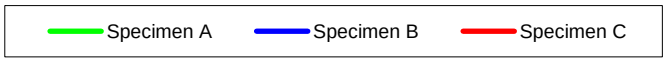


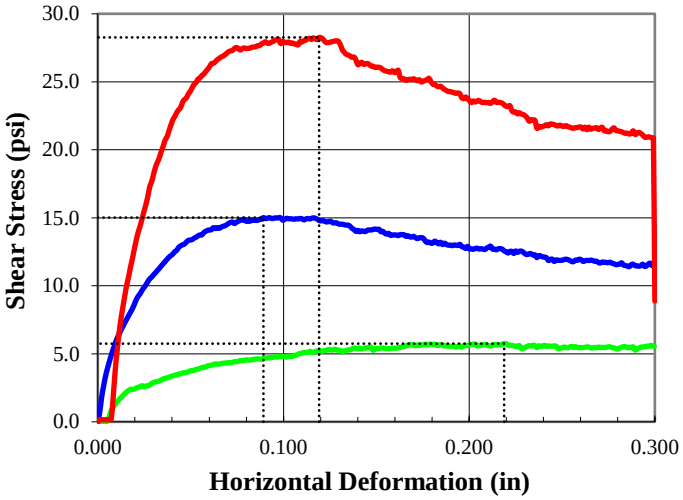
PROJECT NO.:	12188	Rock Loft Subdivision	**FIGURE NO.:** 44B

Direct Shear Test (ASTM D3080)









Initial	Specimen A	B	C
Moisture (%)	7.00	7.00	7.00
Dry Density (pcf)	100.53	98.72	100.02
Void Ratio	0.64	0.68	0.65
Saturation (%)	28.76	27.47	28.39
Diameter (in)	2.50	2.50	2.50
Height (in)	1.00	1.00	1.00

Final	A	B	C
Moisture (%)	18.57	18.57	18.57
Dry Density (pcf)	103.94	101.45	103.55
Void Ratio	0.59	0.63	0.60
Saturation (%)	83.3	78.1	82.4
Diameter (in)	2.50	2.50	2.50
Height (in)	0.99	0.99	0.98
Normal Stress (psi)	6.94	13.89	27.78
Shear Stress (psi)(@Peak)	5.74	15.01	28.26
Peak Strain (%)	8.74	3.56	4.77
Rate (in/min)	0.0033	0.0033	0.0033
Peak Deformation (in.)	0.219	0.089	0.119

Sample Information	
Test Pit Number:	B-1
Sample Number:	3
Depth:	10 ft
Sample Type:	Remolded
Description:	Sand (SP-SM)
Test Type:	Consolidated - Drained

Cherry Ridge Subdivision

Fruit Height , Utah

CMT ENGINEERING LABORATORIES

Lab Data

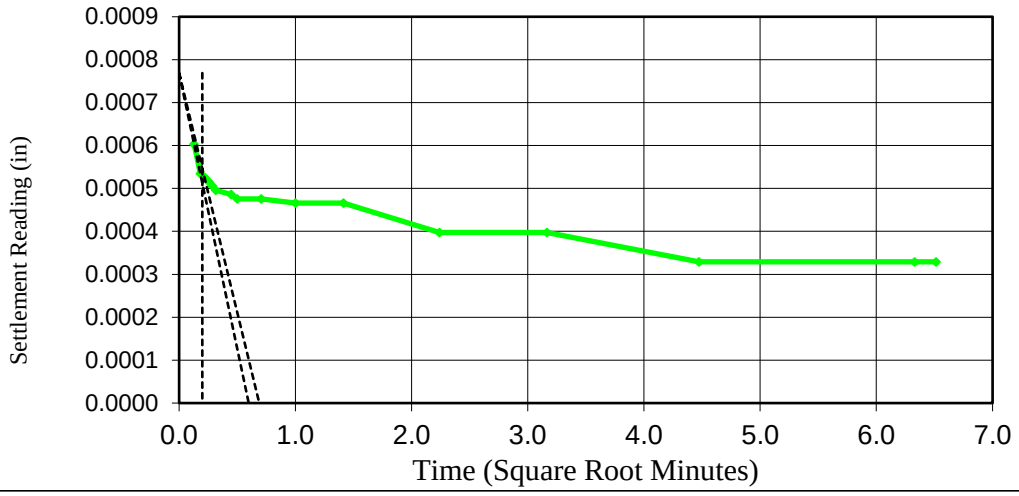
Date: 13-Nov-20

Job # 12188

Figure: 45A

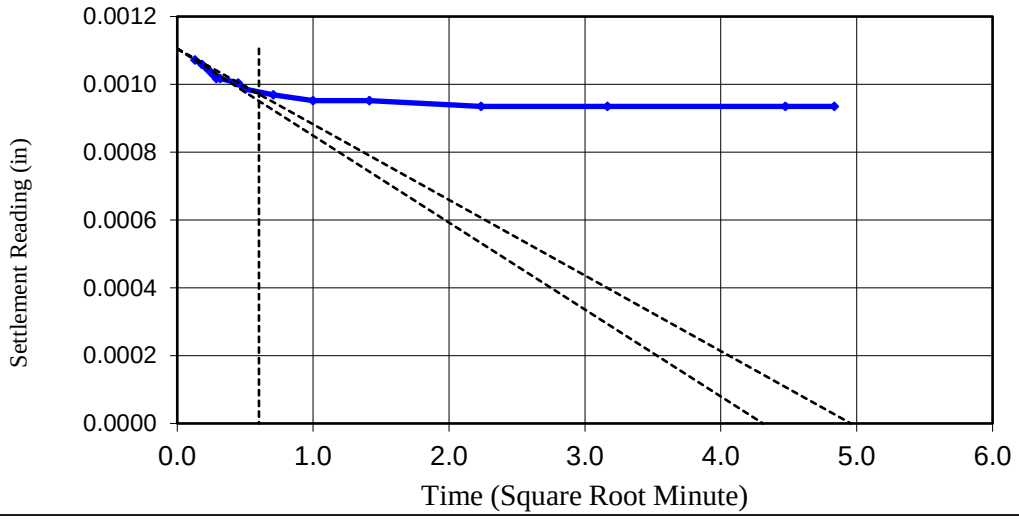
Direct Shear Test (ASTM D3080)

Consolidation Graph Specimen A



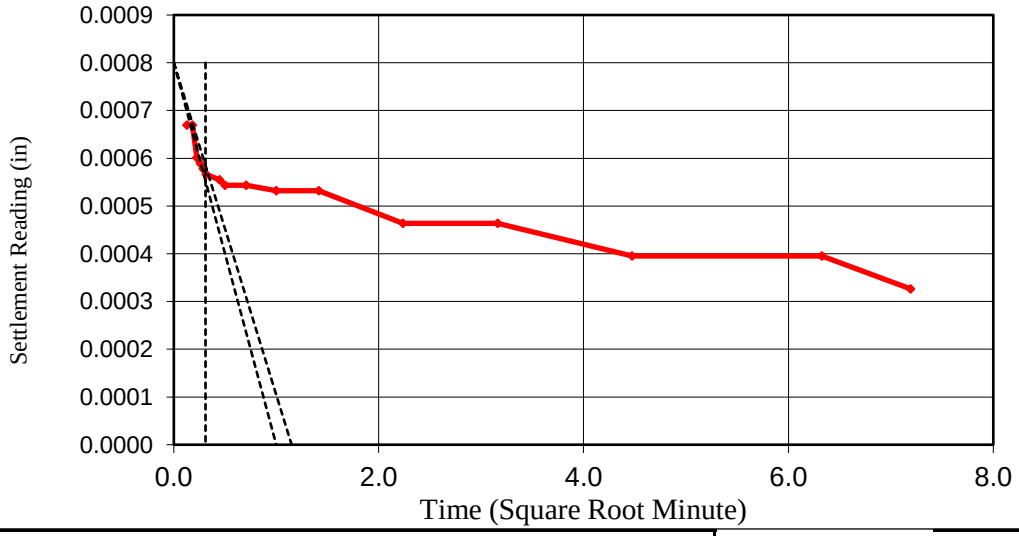
$sqrt(t_{90})$ =	0.2	
t_{90} =	0.0	min.
$t_f = 50\ t_{90} / 4.28$ =	0.5	min.
max $d_r = 0.5 / t_f$ =	1.0700	in./min.
selected d_r =	0.0033	in./min.

Consolidation Graph Specimen B



$sqrt(t_{90})$ =	0.6	
t_{90} =	0.4	min.
$t_f = 50\ t_{90} / 4.28$ =	4.2	min.
max $d_r = 0.5 / t_f$ =	0.1189	in./min.
selected d_r =	0.0033	in./min.

Consolidation Graph Specimen C



$sqrt(t_{90})$ =	0.31	
t_{90} =	0.1	min.
$t_f = 50\ t_{90} / 4.28$ =	1.1	min.
max $d_r = 0.5 / t_f$ =	0.4454	in./min.
selected d_r =	0.0033	in./min.

Cherry Ridge Subdivision

Fruit Height , Utah

CMT ENGINEERING LABORATORIES

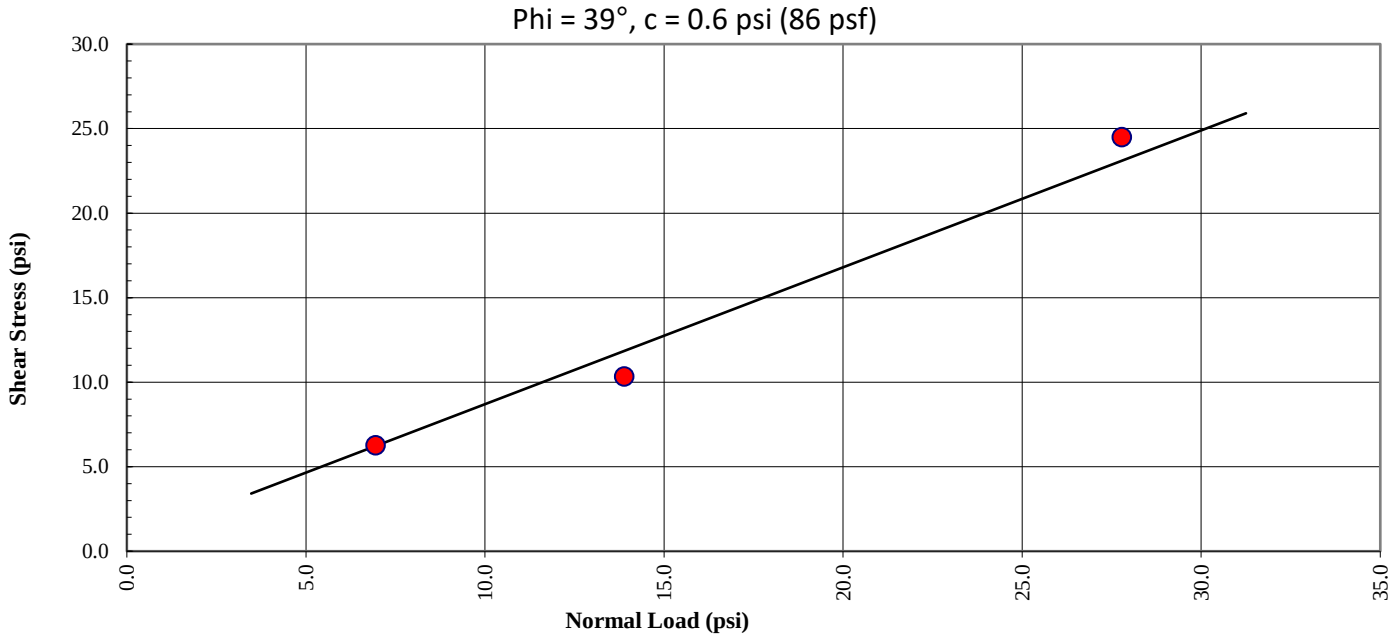
Lab Data

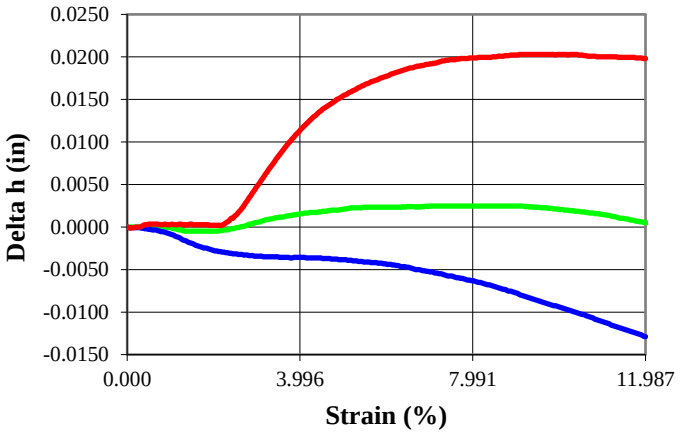
Date: 13-Nov-20

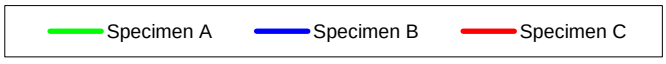
Job # 12188

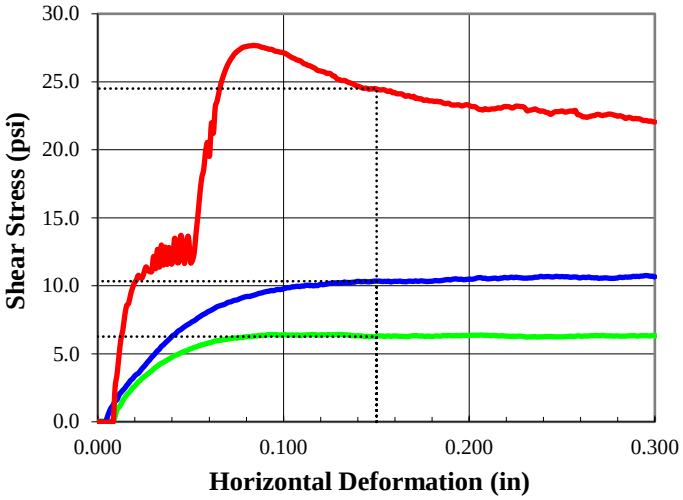
Figure: 45B

Direct Shear Test (ASTM D3080)









Initial	Specimen A	B	C
Moisture (%)	3.78	3.78	3.78
Dry Density (pcf)	114.35	111.95	112.85
Void Ratio	0.45	0.48	0.47
Saturation (%)	22.42	20.97	21.50
Diameter (in)	2.50	2.50	2.50
Height (in)	1.00	1.00	1.00

Final	A	B	C
Moisture (%)	15.92	15.92	15.92
Dry Density (pcf)	117.76	115.55	113.14
Void Ratio	0.40	0.43	0.46
Saturation (%)	100.0	97.9	91.4
Diameter (in)	2.50	2.50	2.50
Height (in)	0.98	0.98	0.99
Normal Stress (psi)	6.94	13.89	27.78
Shear Stress (psi)(@0.15")	6.26	10.34	24.50
Strain at 0.15 in. (%)	3.23	5.97	5.80
Rate (in/min)	0.0033	0.0033	0.0033
Deformation (in.)	0.150	0.150	0.150

Sample Information	
Test Pit Number:	B-2
Sample Number:	
Depth:	20 ft
Sample Type:	Remolded
Description:	Sand (SP)
Test Type:	Consolidated - Drained

Cherry Ridges Subdivision

Fruit Heights, Utah

CMT ENGINEERING LABORATORIES

Lab Data

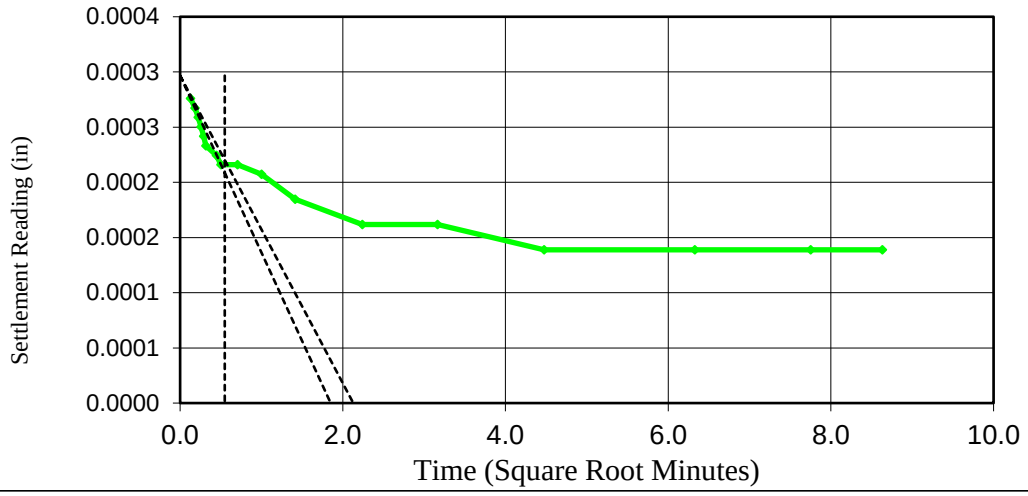
Date: 13-Nov-20

Job # 12188

Figure: 46A

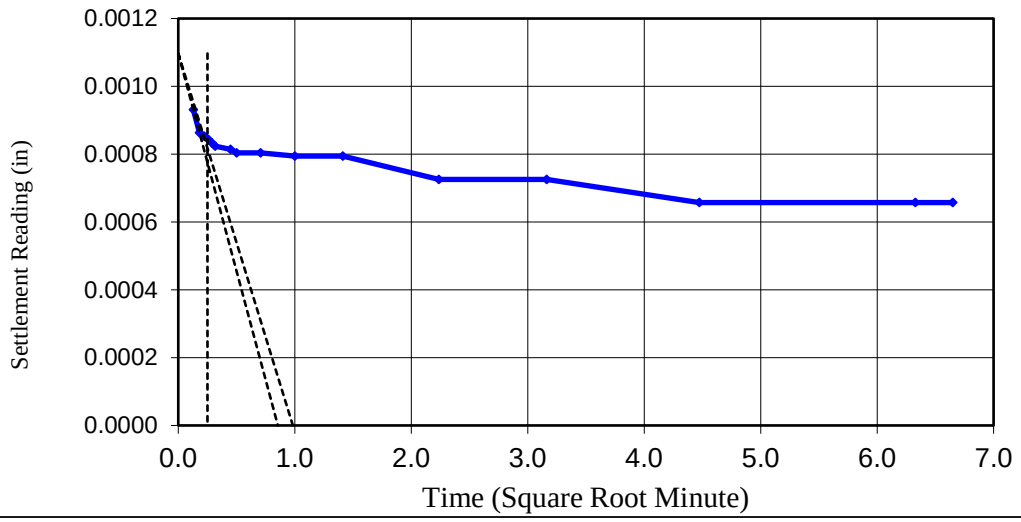
Direct Shear Test (ASTM D3080)

Consolidation Graph Specimen A



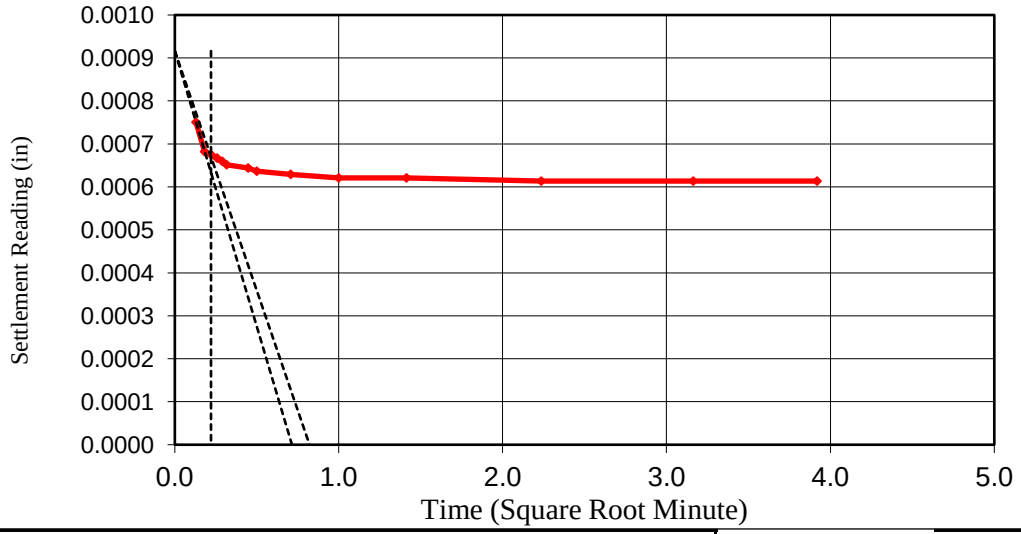
sqrt(t_{90}) =	0.55	
t_{90} =	0.3	min.
t_f = 50 t_{90} / 4.28 =	3.5	min.
max d_r = 0.5 / t_f =	0.1415	in./min.
selected d_r =	0.0033	in./min.

Consolidation Graph Specimen B



sqrt(t_{90}) =	0.25	
t_{90} =	0.1	min.
t_f = 50 t_{90} / 4.28 =	0.7	min.
max d_r = 0.5 / t_f =	0.6848	in./min.
selected d_r =	0.0033	in./min.

Consolidation Graph Specimen C



sqrt(t_{90}) =	0.22	
t_{90} =	0.0	min.
t_f = 50 t_{90} / 4.28 =	0.6	min.
max d_r = 0.5 / t_f =	0.8843	in./min.
selected d_r =	0.0033	in./min.

Cherry Ridges Subdivision

Fruit Heights, Utah

CMT ENGINEERING LABORATORIES

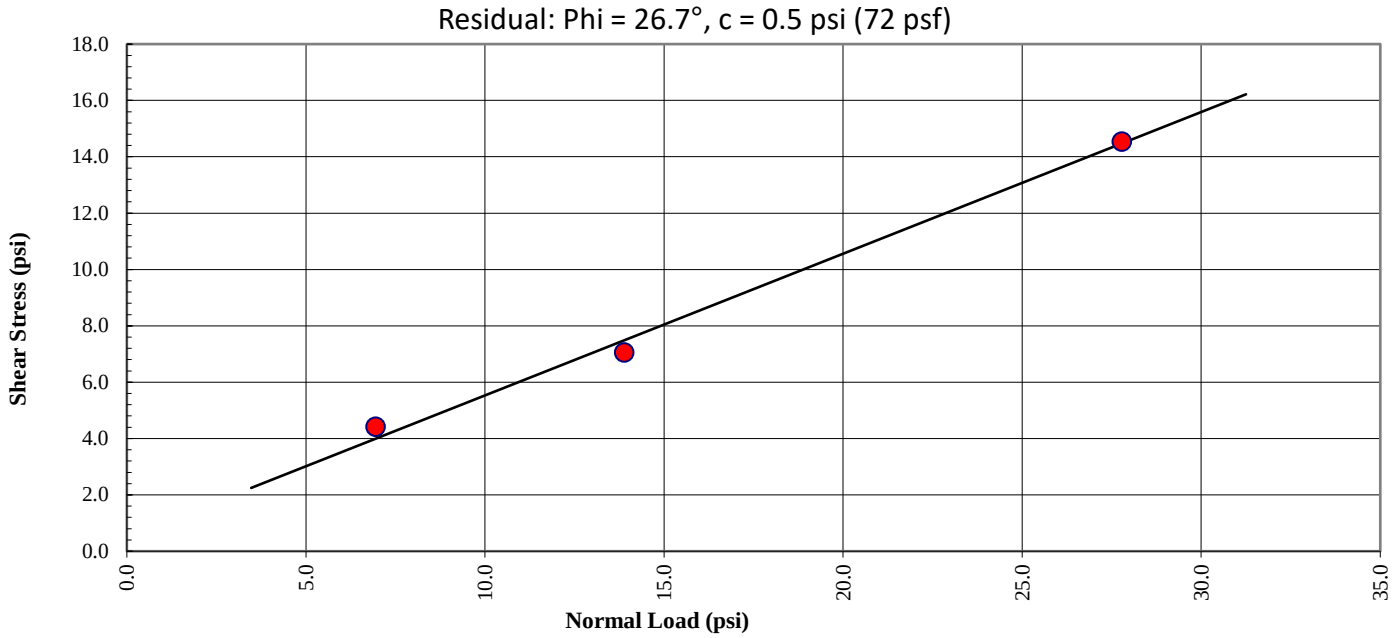
Lab Data

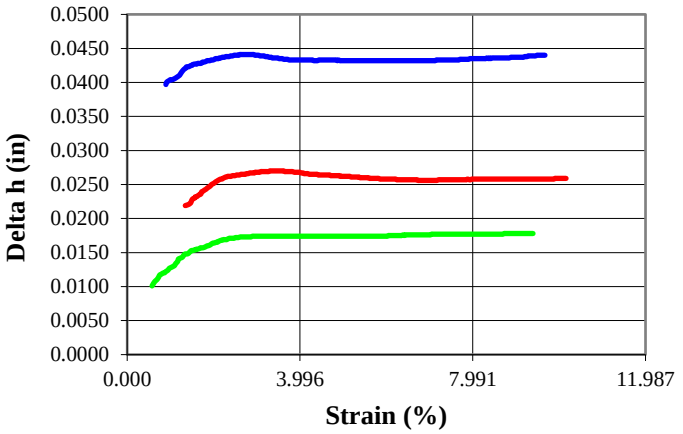
Date: 13-Nov-20

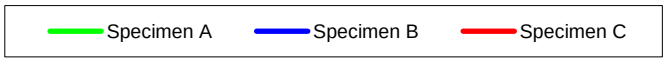
Job # 12188

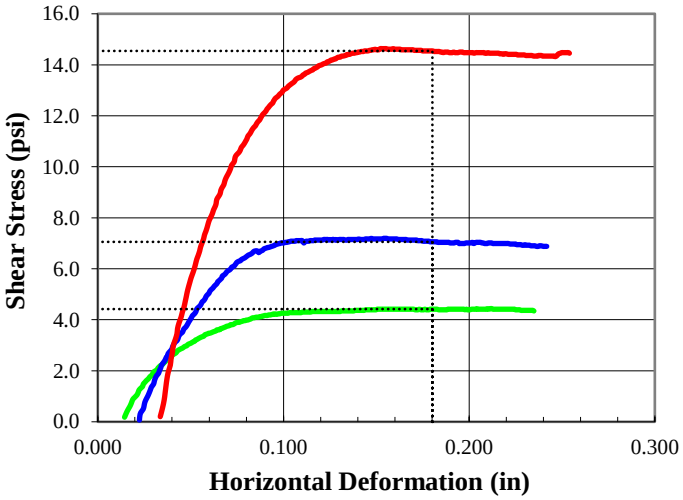
Figure: 46B

Direct Shear Test (ASTM D3080)









Initial	Specimen A	B	C
Moisture (%)	6.86	6.86	6.86
Dry Density (pcf)	109.81	108.58	113.37
Void Ratio	0.51	0.52	0.46
Saturation (%)	35.94	34.76	39.64
Diameter (in)	2.50	2.50	2.50
Height (in)	1.00	1.00	1.00

Final	A	B	C
Moisture (%)	26.97	26.97	26.97
Dry Density (pcf)	106.05	102.20	104.22
Void Ratio	0.56	0.62	0.59
Saturation (%)	100.0	100.0	100.0
Diameter (in)	2.50	2.50	2.50
Height (in)	1.00	1.00	0.97
Normal Stress (psi)	6.94	13.89	27.78
Shear Stress (psi)(@0.18")	4.41	7.06	14.54
Strain at 0.18 in. (%)	6.02	4.10	5.74
Rate (in/min)	0.0033	0.0033	0.0033
Deformation (in.)	0.180	0.180	0.180

Sample Information	
Test Pit Number:	B-3
Sample Number:	
Depth:	25 ft
Sample Type:	Remolded
Description:	Sandy Silt (ML)
Test Type:	Consolidated - Drained

Cherry Ridges Subdivision

Fruit Heights, Utah

CMT ENGINEERING LABORATORIES

Lab Data

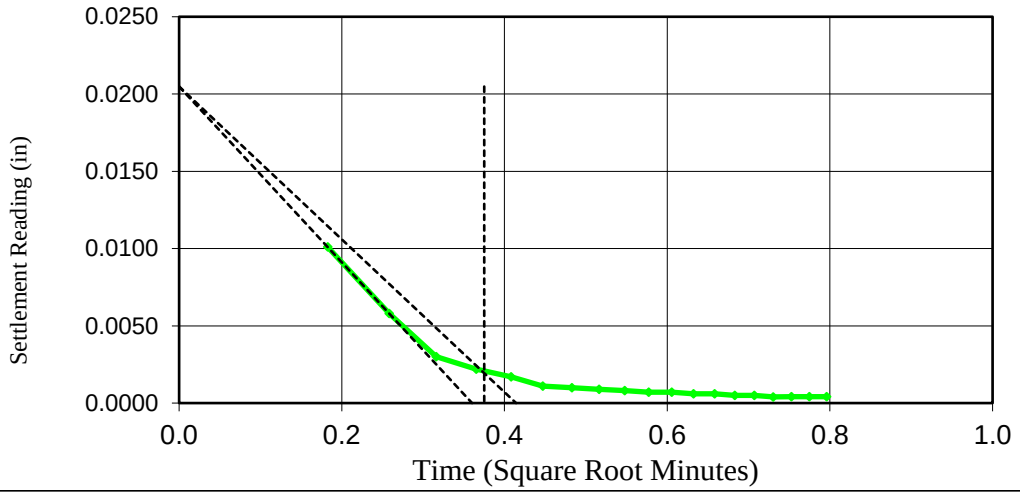
Date: 13-Nov-20

Job # 12188

Figure: 47A

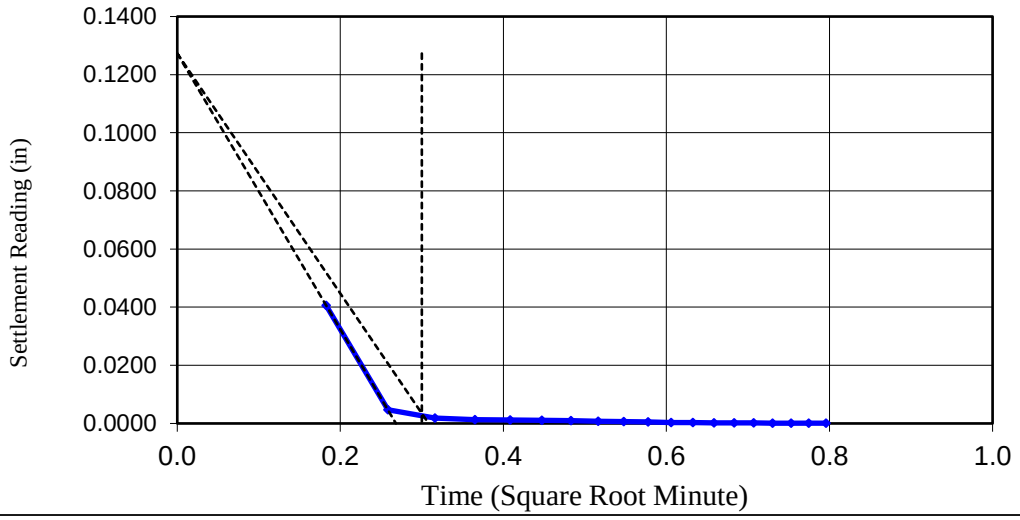
Direct Shear Test (ASTM D3080)

Consolidation Graph Specimen A



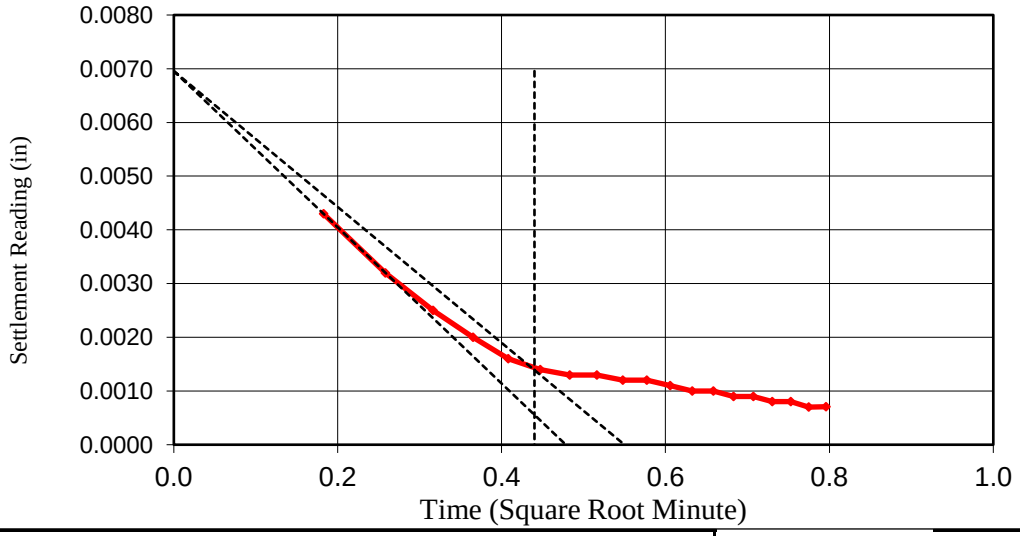
sqrt(t_{90}) =	0.375	
t_{90} =	0.1	min.
$t_f = 50\ t_{90} / 4.28$ =	1.6	min.
max $d_r = 0.5 / t_f$ =	0.3044	in./min.
selected d_r =	0.0033	in./min.

Consolidation Graph Specimen B



sqrt(t_{90}) =	0.3	
t_{90} =	0.1	min.
$t_f = 50\ t_{90} / 4.28$ =	1.1	min.
max $d_r = 0.5 / t_f$ =	0.4756	in./min.
selected d_r =	0.0033	in./min.

Consolidation Graph Specimen C



sqrt(t_{90}) =	0.44	
t_{90} =	0.2	min.
$t_f = 50\ t_{90} / 4.28$ =	2.3	min.
max $d_r = 0.5 / t_f$ =	0.2211	in./min.
selected d_r =	0.0033	in./min.

Cherry Ridges Subdivision

Fruit Heights, Utah

CMT ENGINEERING LABORATORIES

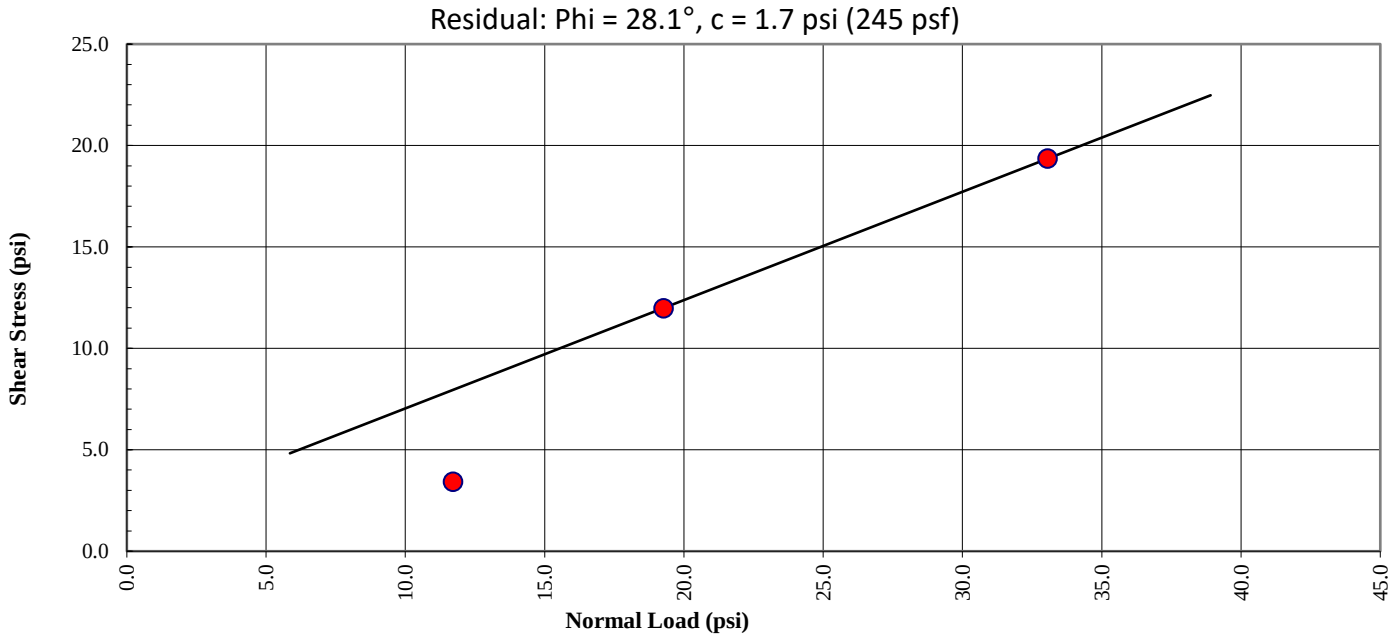
Lab Data

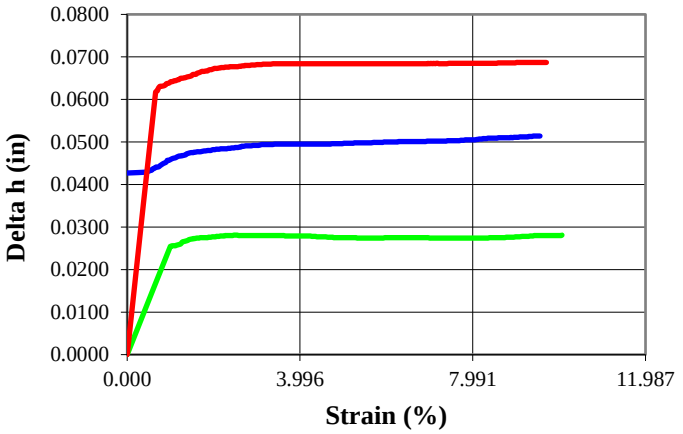
Date: 13-Nov-20

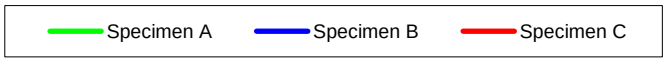
Job # 12188

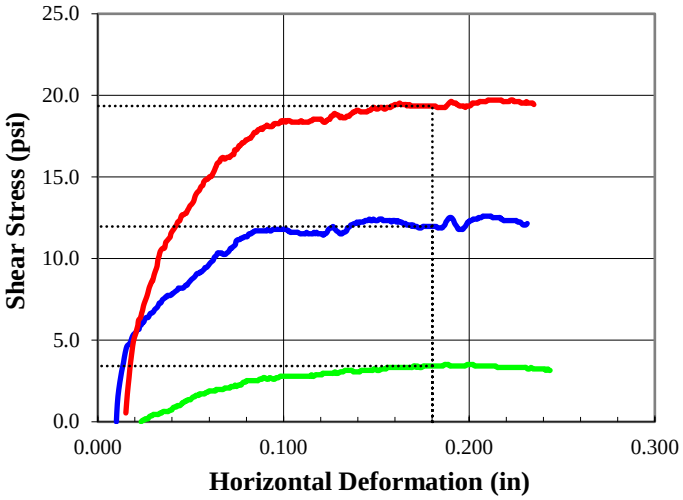
Figure: 47B

Direct Shear Test (ASTM D3080)









Initial	Specimen A	B	C
Moisture (%)	16.61	16.61	16.61
Dry Density (pcf)	92.98	90.56	94.97
Void Ratio	0.78	0.83	0.74
Saturation (%)	56.53	53.28	59.37
Diameter (in)	2.42	2.42	2.42
Height (in)	1.00	1.00	1.00

Final	A	B	C
Moisture (%)	25.59	25.59	25.59
Dry Density (pcf)	96.35	95.63	94.90
Void Ratio	0.72	0.73	0.74
Saturation (%)	94.7	93.0	91.3
Diameter (in)	2.42	2.42	2.42
Height (in)	1.00	1.00	0.97
Normal Stress (psi)	11.71	19.26	33.05
Shear Stress (psi)(@0.18")	3.42	11.97	19.35
Strain at 0.18 in. (%)	7.02	5.23	6.52
Rate (in/min)	0.0024	0.0024	0.0024
Deformation (in.)	0.180	0.180	0.180

Sample Information	
Test Pit Number:	B-4
Sample Number:	
Depth:	10 ft
Sample Type:	Remolded
Description:	Silt with sand (ML)
Test Type:	Consolidated - Drained

Cherry Ridges Subdivision

Fruit Heights, Utah

CMT ENGINEERING LABORATORIES

Lab Data

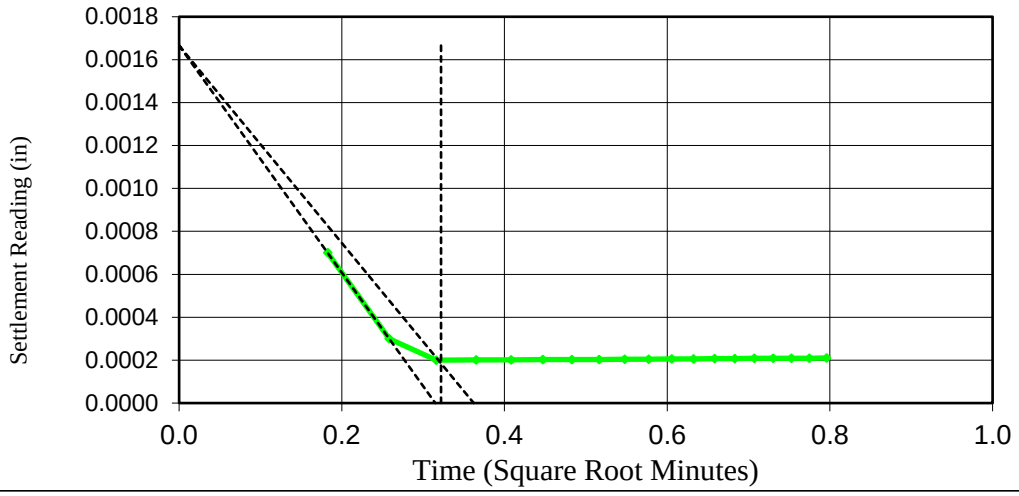
Date: 13-Nov-20

Job # 12188

Figure: 48A

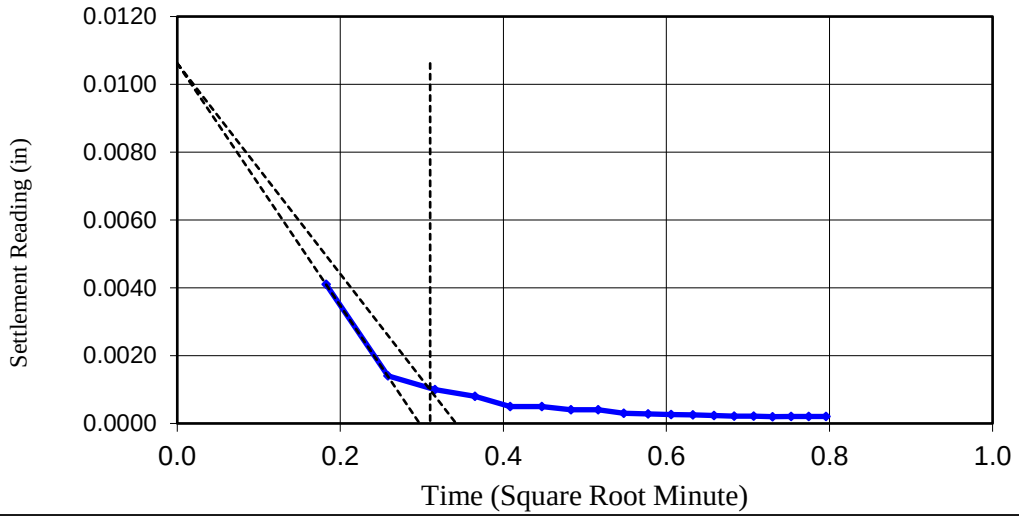
Direct Shear Test (ASTM D3080)

Consolidation Graph Specimen A



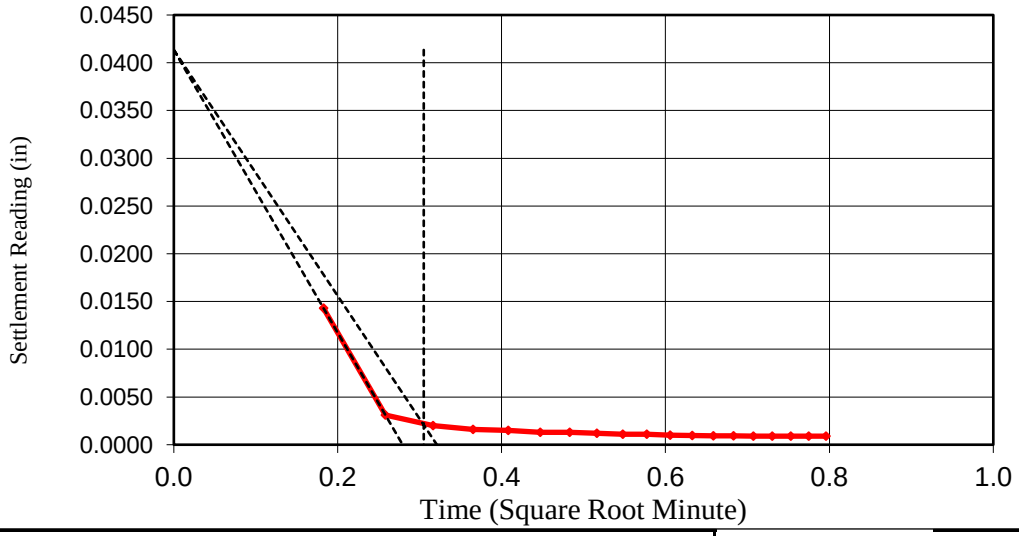
sqrt(t_{90}) =	0.322	
t_{90} =	0.1	min.
$t_f = 50\ t_{90} / 4.28$ =	1.2	min.
max $d_r = 0.5 / t_f$ =	0.4128	in./min.
selected d_r =	0.0024	in./min.

Consolidation Graph Specimen B



sqrt(t_{90}) =	0.31	
t_{90} =	0.1	min.
$t_f = 50\ t_{90} / 4.28$ =	1.1	min.
max $d_r = 0.5 / t_f$ =	0.4454	in./min.
selected d_r =	0.0024	in./min.

Consolidation Graph Specimen C



sqrt(t_{90}) =	0.305	
t_{90} =	0.1	min.
$t_f = 50\ t_{90} / 4.28$ =	1.1	min.
max $d_r = 0.5 / t_f$ =	0.4601	in./min.
selected d_r =	0.0024	in./min.

Cherry Ridges Subdivision

Fruit Heights, Utah

CMT ENGINEERING LABORATORIES

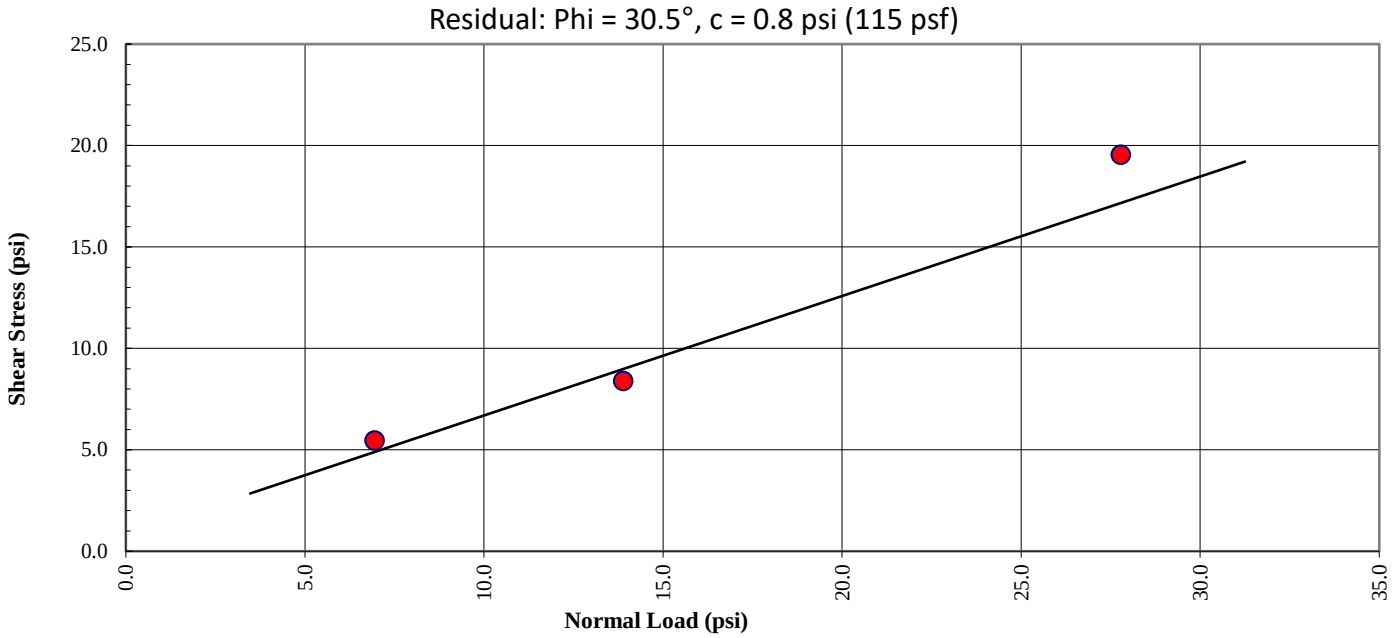
Lab Data

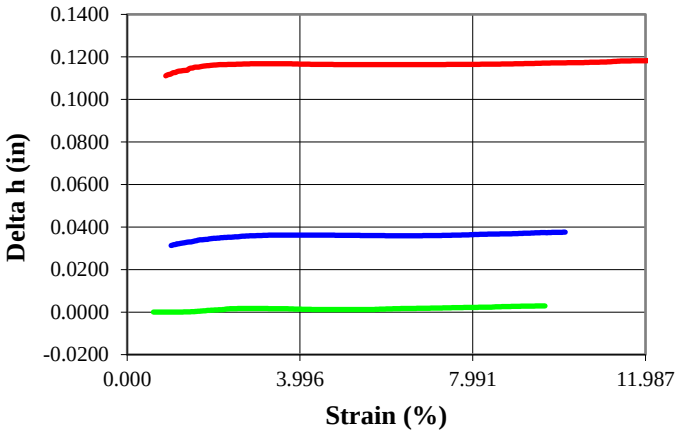
Date: 13-Nov-20

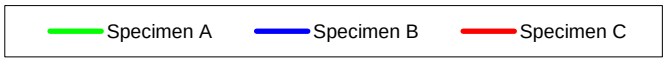
Job # 12188

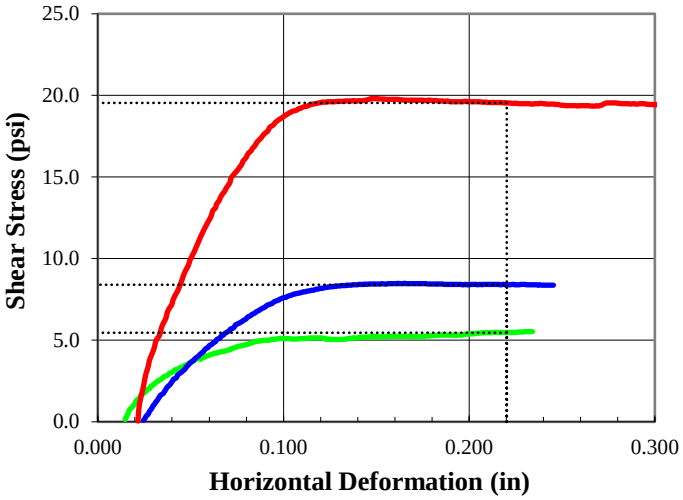
Figure: 48B

Direct Shear Test (ASTM D3080)









Initial	Specimen A	B	C
Moisture (%)	15.98	15.98	15.98
Dry Density (pcf)	114.26	110.76	112.19
Void Ratio	0.45	0.49	0.47
Saturation (%)	94.70	85.92	89.36
Diameter (in)	2.42	2.42	2.42
Height (in)	1.00	1.00	1.00

Final	A	B	C
Moisture (%)	23.87	23.87	23.87
Dry Density (pcf)	113.00	108.52	110.26
Void Ratio	0.46	0.52	0.50
Saturation (%)	100.0	100.0	100.0
Diameter (in)	2.42	2.42	2.42
Height (in)	1.00	1.00	0.97
Normal Stress (psi)	6.94	13.89	27.78
Shear Stress (psi)(@0.22")	5.45	8.39	19.54
Strain at 0.22 in. (%)	8.52	5.67	4.91
Rate (in/min)	0.0024	0.0024	0.0024
Deformation (in.)	0.220	0.220	0.220

Sample Information	
Test Pit Number:	B-4
Sample Number:	36
Depth:	30 ft
Sample Type:	Undisturbed (Rings)
Description:	Sandy Clay (CL)
Test Type:	Consolidated - Drained

Cherry Ridges Subdivision

Fruit Heights, Utah

CMT ENGINEERING LABORATORIES

Lab Data

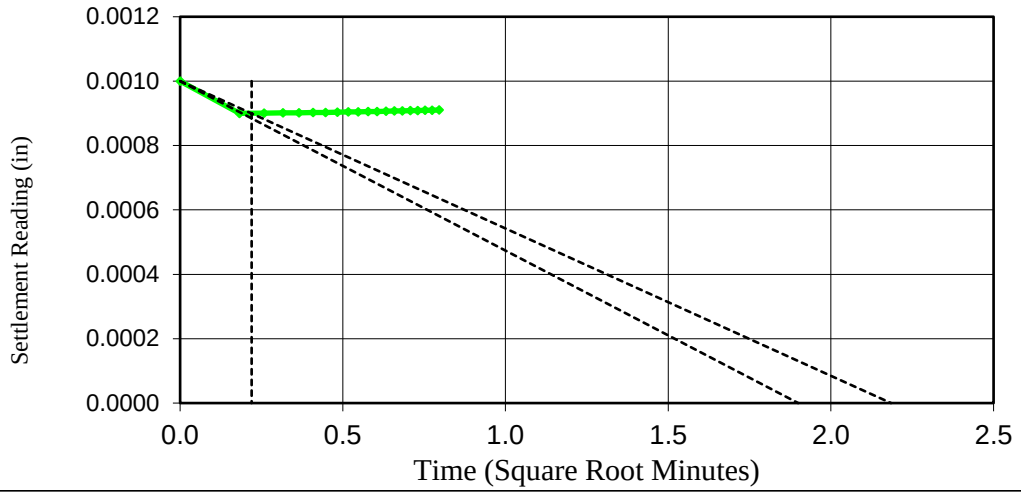
Date: 13-Nov-20

Job # 12188

Figure: 49A

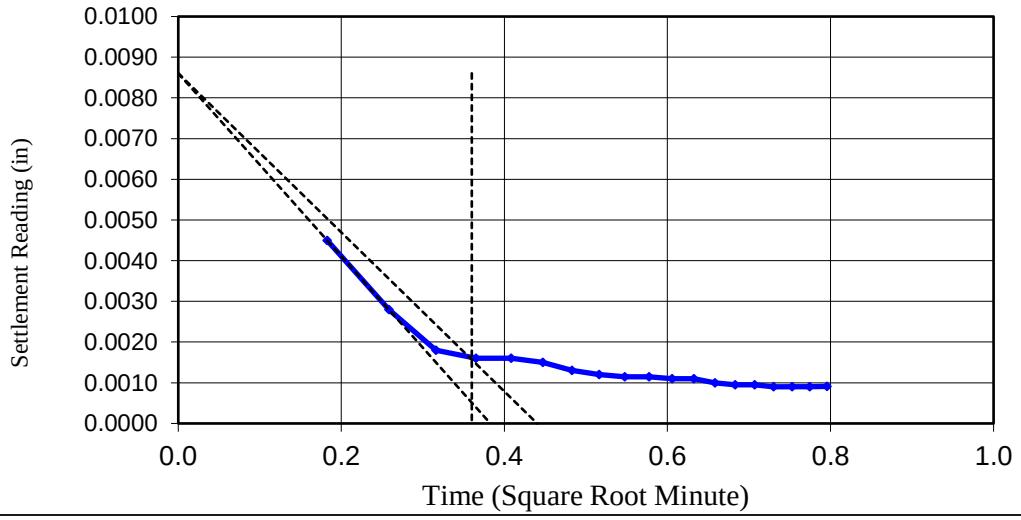
Direct Shear Test (ASTM D3080)

Consolidation Graph Specimen A



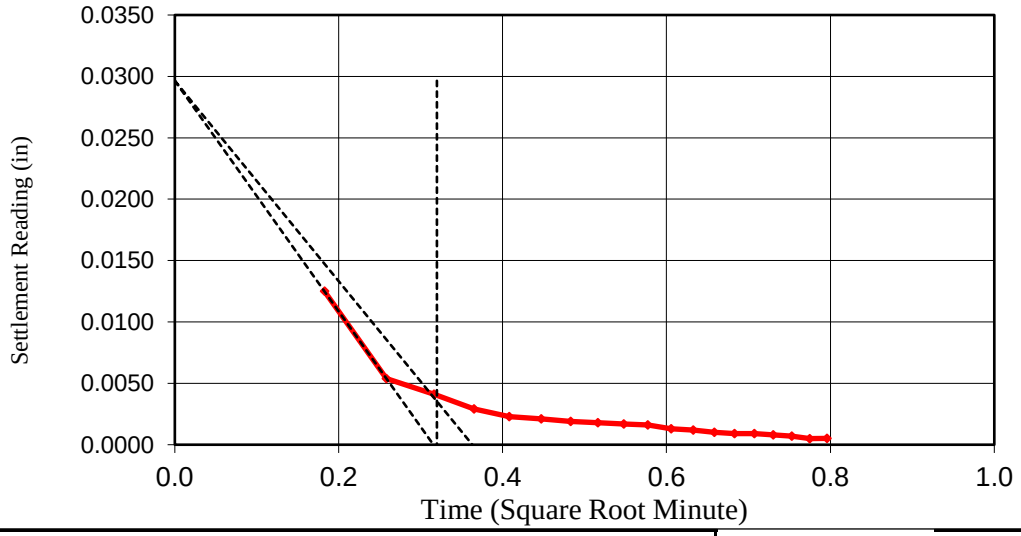
sqrt(t_{90}) =	0.22	
t_{90} =	0.0	min.
t_f = 50 t_{90} / 4.28 =	0.6	min.
max d_r = 0.5 / t_f =	0.8843	in./min.
selected d_r =	0.0024	in./min.

Consolidation Graph Specimen B



sqrt(t_{90}) =	0.36	
t_{90} =	0.1	min.
t_f = 50 t_{90} / 4.28 =	1.5	min.
max d_r = 0.5 / t_f =	0.3302	in./min.
selected d_r =	0.0024	in./min.

Consolidation Graph Specimen C



sqrt(t_{90}) =	0.32	
t_{90} =	0.1	min.
t_f = 50 t_{90} / 4.28 =	1.2	min.
max d_r = 0.5 / t_f =	0.4180	in./min.
selected d_r =	0.0024	in./min.

Cherry Ridges Subdivision

Fruit Heights, Utah

CMT ENGINEERING LABORATORIES

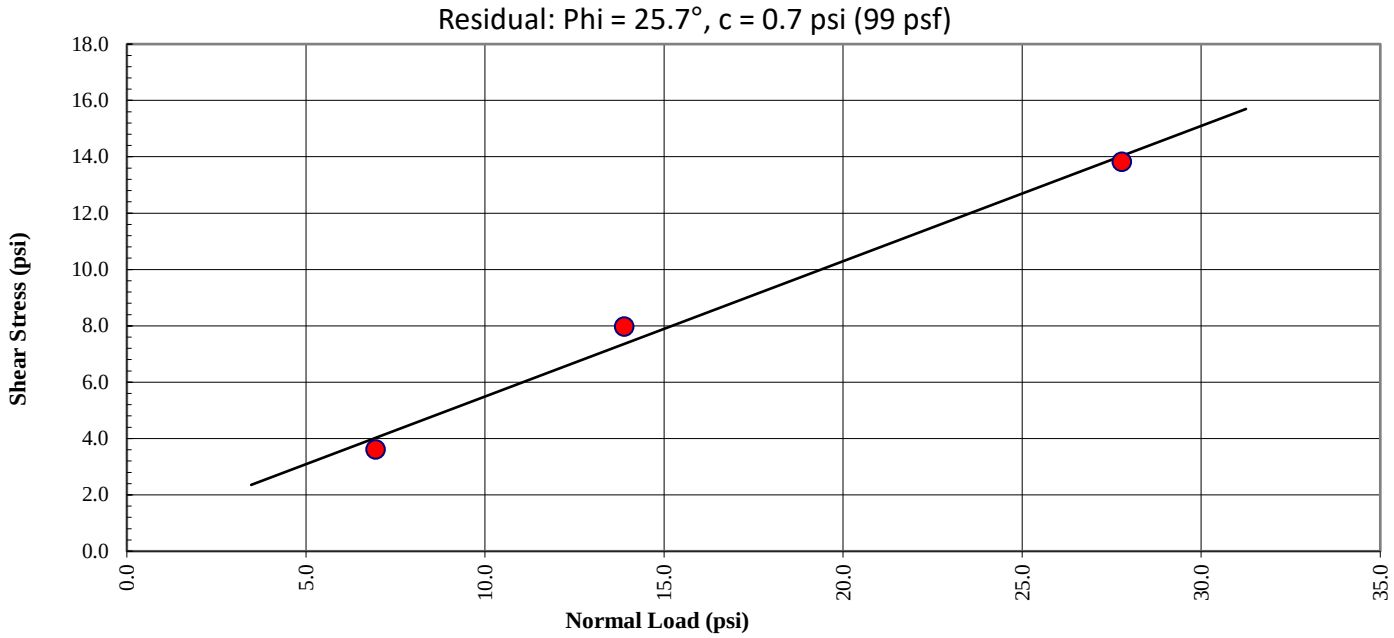
Lab Data

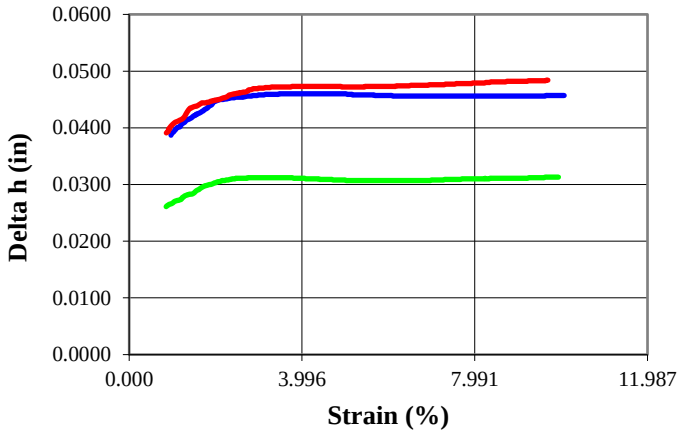
Date: 13-Nov-20

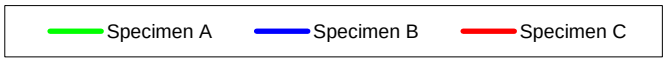
Job # 12188

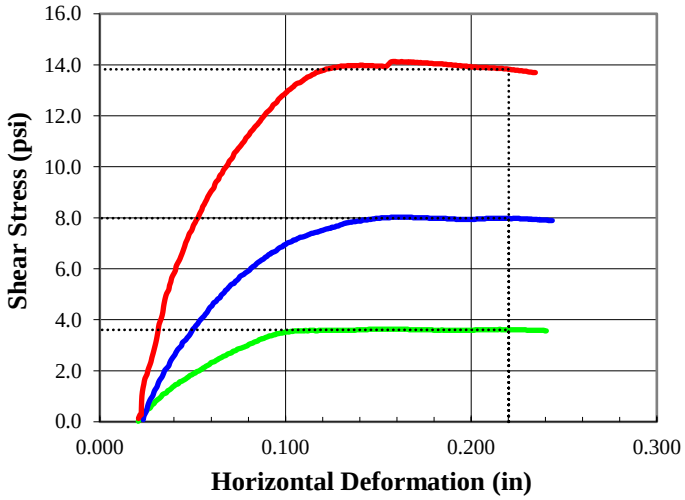
Figure: 49B

Direct Shear Test (ASTM D3080)









Initial	Specimen A	B	C
Moisture (%)	28.65	28.65	28.65
Dry Density (pcf)	96.89	96.32	97.86
Void Ratio	0.71	0.72	0.69
Saturation (%)	107.43	105.89	110.06
Diameter (in)	2.42	2.42	2.42
Height (in)	1.00	1.00	1.00

Final	A	B	C
Moisture (%)	25.37	25.37	25.37
Dry Density (pcf)	100.35	96.65	98.17
Void Ratio	0.65	0.71	0.68
Saturation (%)	100.0	94.6	98.2
Diameter (in)	2.42	2.42	2.42
Height (in)	1.00	1.00	0.97
Normal Stress (psi)	6.94	13.89	27.78
Shear Stress (psi)(@0.22")	3.61	7.98	13.82
Strain at 0.22 in. (%)	5.71	6.14	5.02
Rate (in/min)	0.0050	0.0050	0.0050
Deformation (in.)	0.220	0.220	0.220

Sample Information	
Test Pit Number:	B-4
Sample Number:	36
Depth:	40 ft
Sample Type:	Undisturbed (Rings)
Description:	Silty Clay (CL)
Test Type:	Consolidated - Drained

Cherry Ridges Subdivision

Fruit Heights, Utah

CMT ENGINEERING LABORATORIES

Lab Data

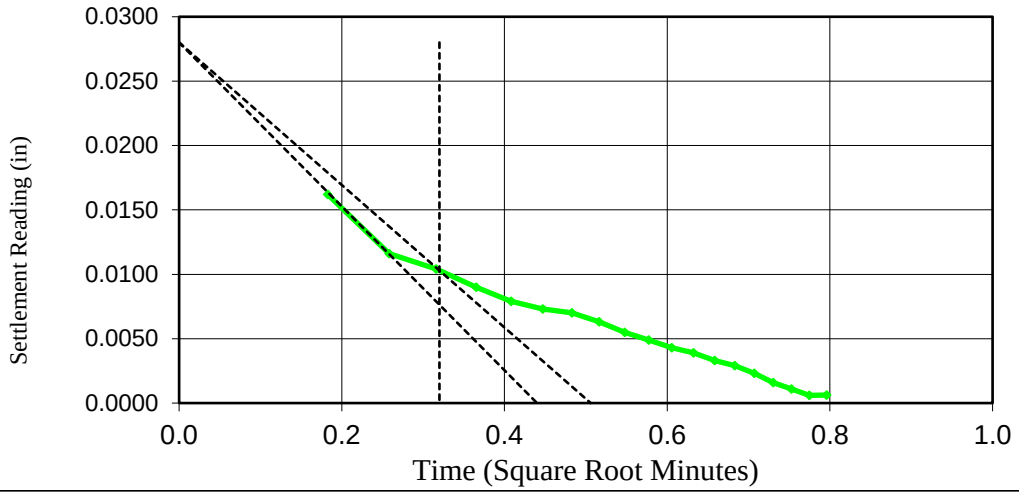
Date: 13-Nov-20

Job # 12188

Figure: 50A

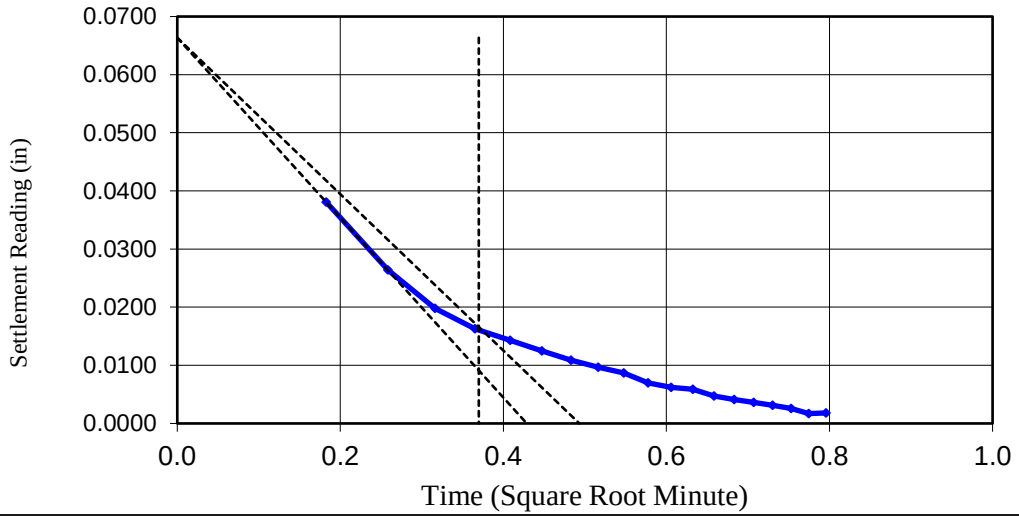
Direct Shear Test (ASTM D3080)

Consolidation Graph Specimen A



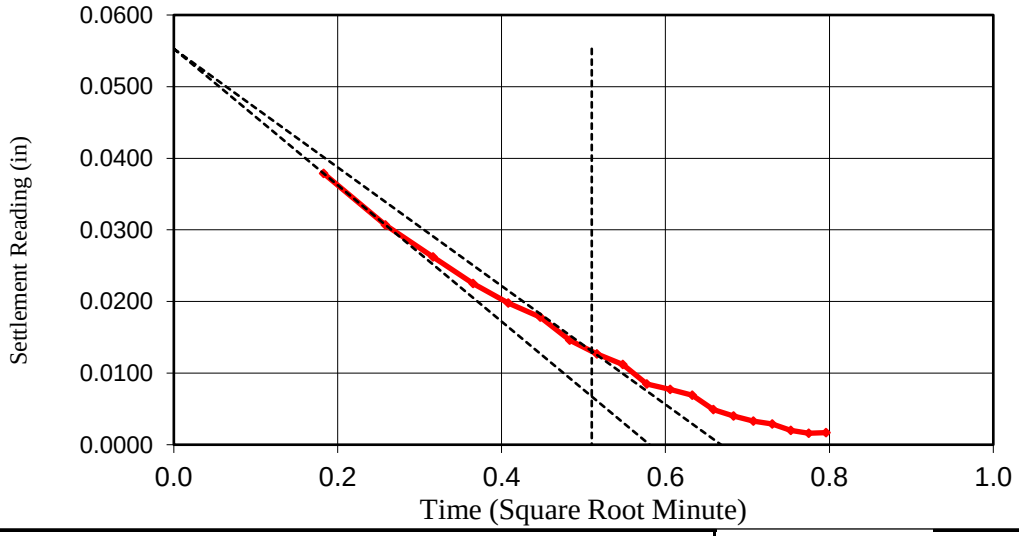
sqrt(t_{90}) =	0.32	
t_{90} =	0.1	min.
t_f = 50 t_{90} / 4.28 =	1.2	min.
max d_r = 0.5 / t_f =	0.4180	in./min.
selected d_r =	0.0050	in./min.

Consolidation Graph Specimen B



sqrt(t_{90}) =	0.37	
t_{90} =	0.1	min.
t_f = 50 t_{90} / 4.28 =	1.6	min.
max d_r = 0.5 / t_f =	0.3126	in./min.
selected d_r =	0.0050	in./min.

Consolidation Graph Specimen C



sqrt(t_{90}) =	0.51	
t_{90} =	0.3	min.
t_f = 50 t_{90} / 4.28 =	3.0	min.
max d_r = 0.5 / t_f =	0.1646	in./min.
selected d_r =	0.0050	in./min.

Cherry Ridges Subdivision

Fruit Heights, Utah

CMT ENGINEERING LABORATORIES

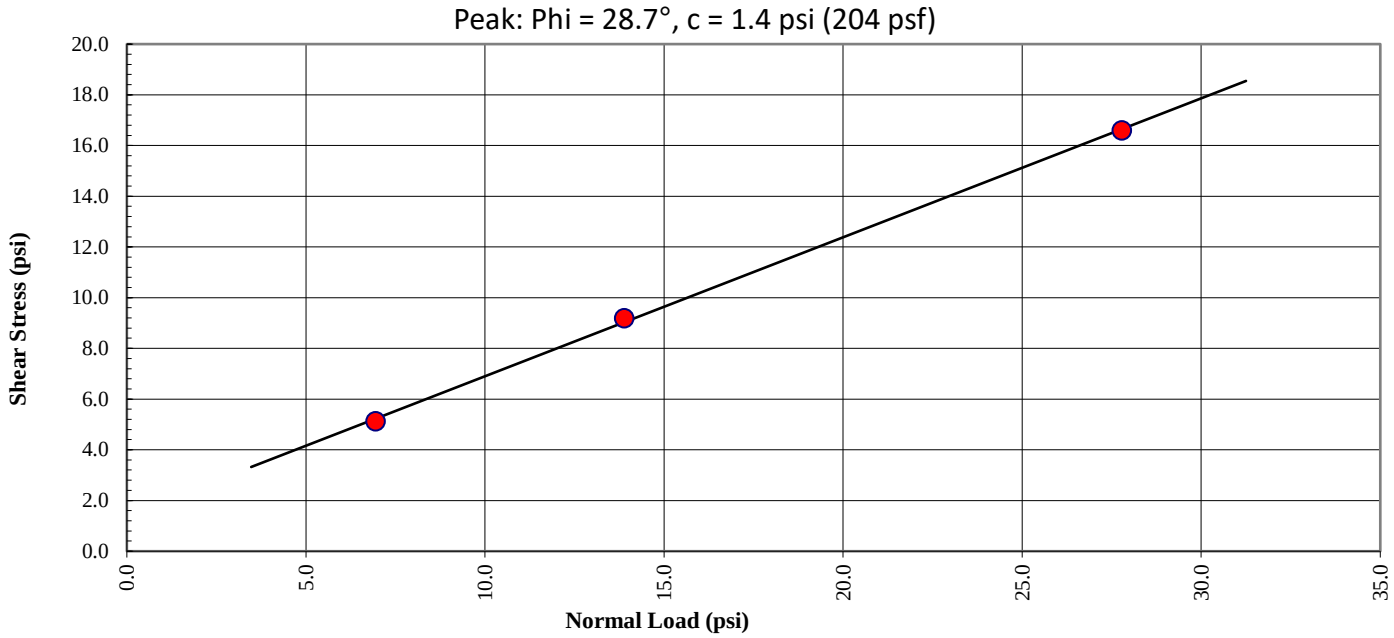
Lab Data

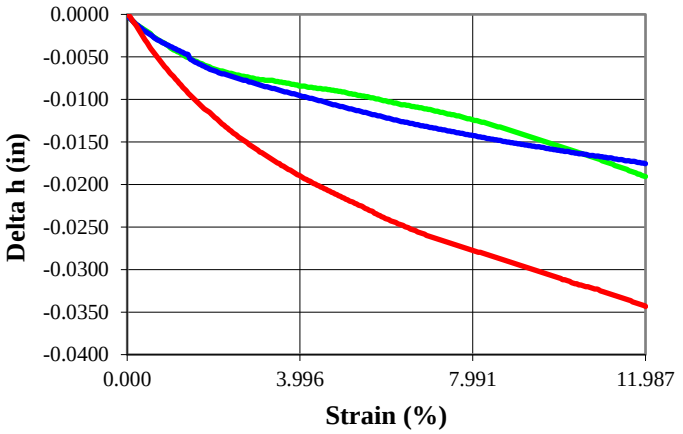
Date: 13-Nov-20

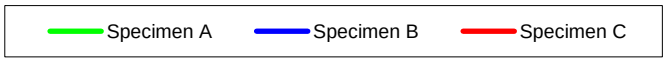
Job # 12188

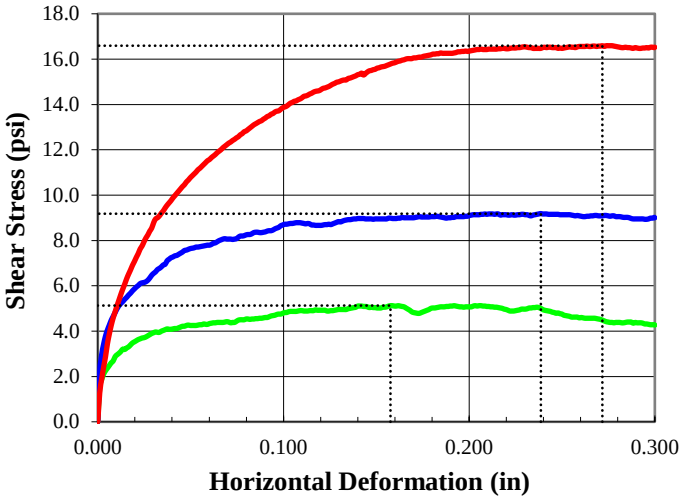
Figure: 50B

Direct Shear Test (ASTM D3080)









Initial	Specimen A	B	C
Moisture (%)	2.65	2.65	2.65
Dry Density (pcf)	108.19	105.84	106.60
Void Ratio	0.53	0.56	0.55
Saturation (%)	13.30	12.50	12.75
Diameter (in)	2.50	2.50	2.50
Height (in)	1.00	1.00	1.00

Final	A	B	C
Moisture (%)	18.87	18.87	18.87
Dry Density (pcf)	103.94	100.94	102.31
Void Ratio	0.59	0.64	0.62
Saturation (%)	84.6	78.3	81.1
Diameter (in)	2.50	2.50	2.50
Height (in)	0.93	0.96	0.86
Normal Stress (psi)	6.94	13.89	27.78
Shear Stress (psi)(@Peak)	5.12	9.19	16.59
Peak Strain (%)	6.30	9.53	10.86
Rate (in/min)	0.0024	0.0024	0.0024
Peak Deformation (in.)	0.157	0.238	0.271

Sample Information	
Test Pit Number:	TP-15
Sample Number:	
Depth:	7 ft
Sample Type:	Remolded
Description:	Clay (CL)
Test Type:	Consolidated - Drained

Cherry Ridges Subdivision

Fruit Heights, Utah

CMT ENGINEERING LABORATORIES

Lab Data

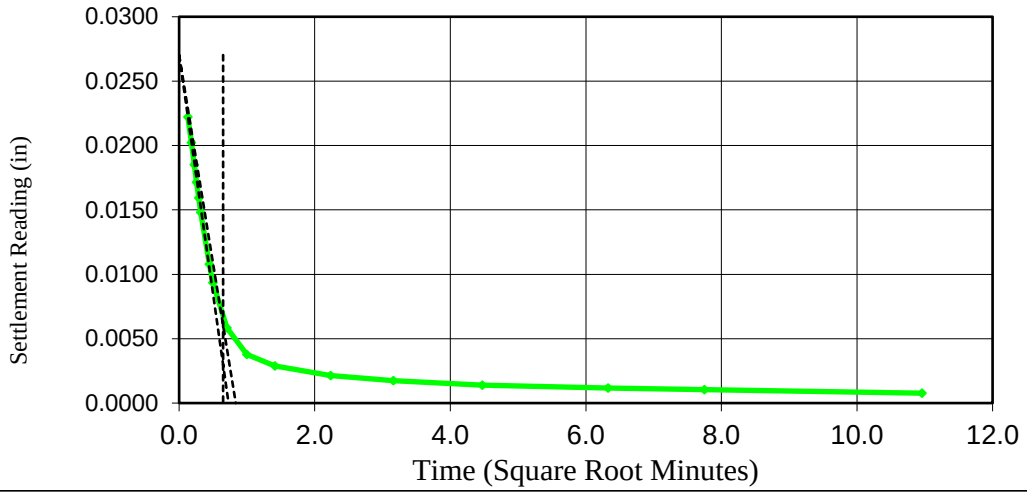
Date: 13-Nov-20

Job # 12188

Figure: 51A

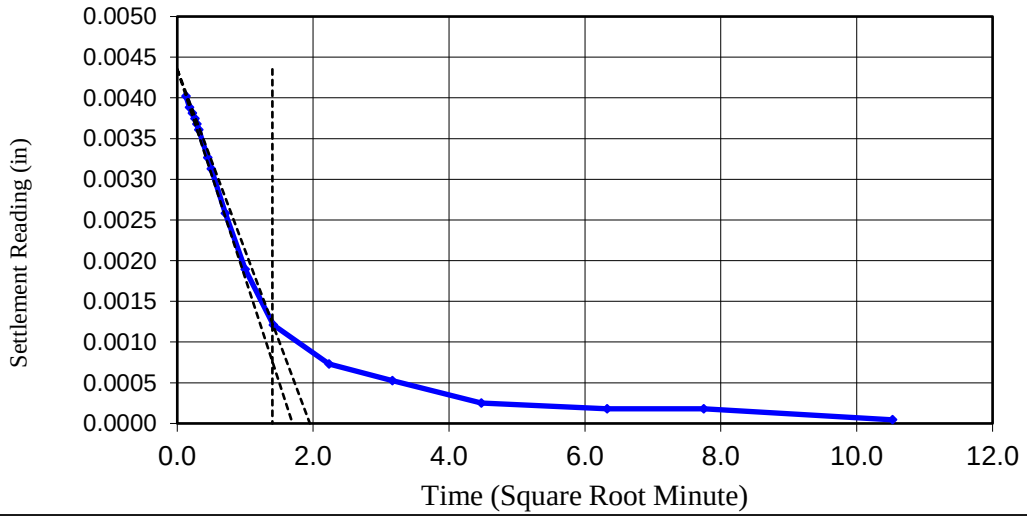
Direct Shear Test (ASTM D3080)

Consolidation Graph Specimen A



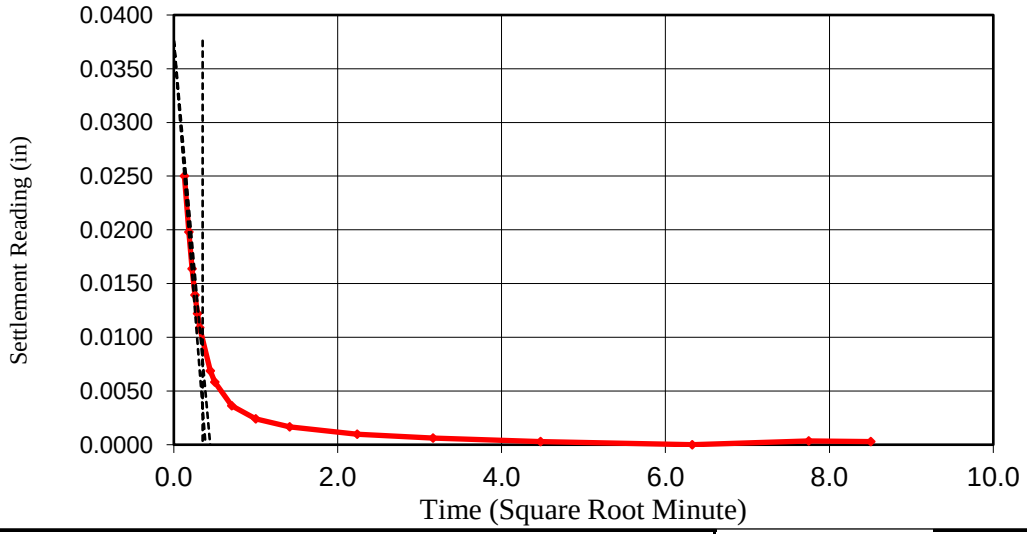
sqrt(t_{90}) =	0.65	
t_{90} =	0.4	min.
t_f = 50 t_{90} / 4.28 =	4.9	min.
max d_r = 0.5 / t_f =	0.1013	in./min.
selected d_r =	0.0024	in./min.

Consolidation Graph Specimen B



sqrt(t_{90}) =	1.4	
t_{90} =	2.0	min.
t_f = 50 t_{90} / 4.28 =	22.9	min.
max d_r = 0.5 / t_f =	0.0218	in./min.
selected d_r =	0.0024	in./min.

Consolidation Graph Specimen C



sqrt(t_{90}) =	0.35	
t_{90} =	0.1	min.
t_f = 50 t_{90} / 4.28 =	1.4	min.
max d_r = 0.5 / t_f =	0.3494	in./min.
selected d_r =	0.0024	in./min.

Cherry Ridges Subdivision

Fruit Heights, Utah

CMT ENGINEERING LABORATORIES

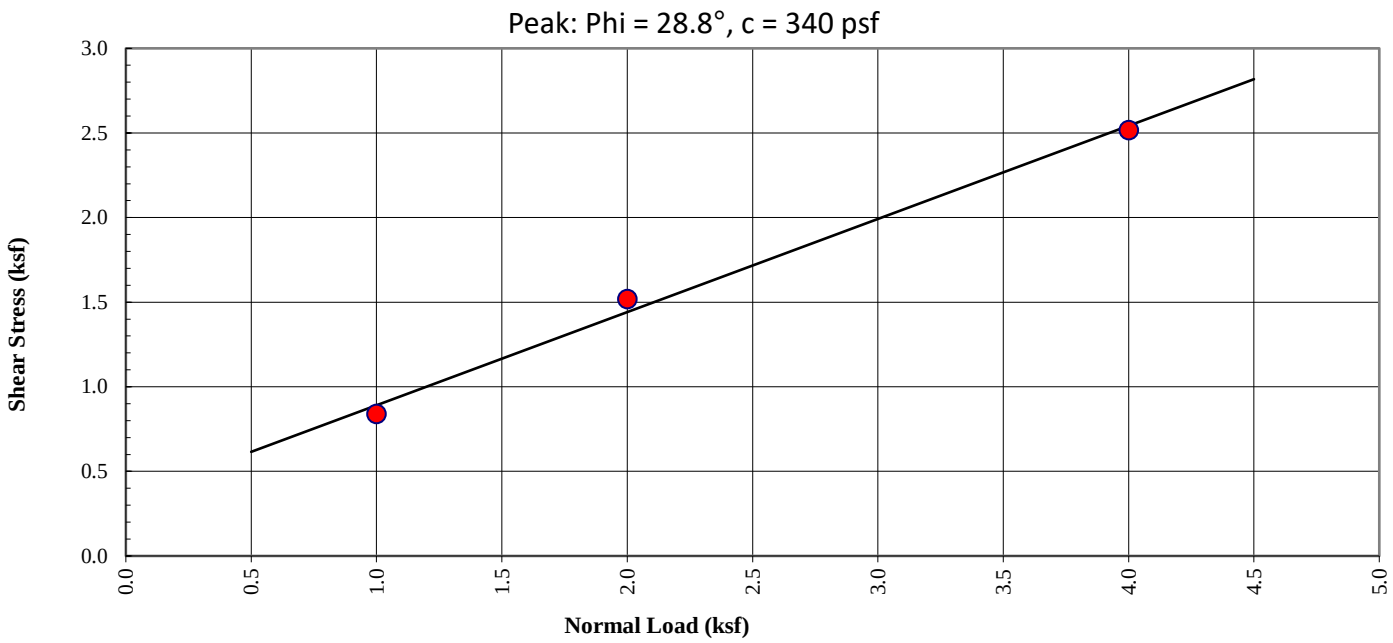
Lab Data

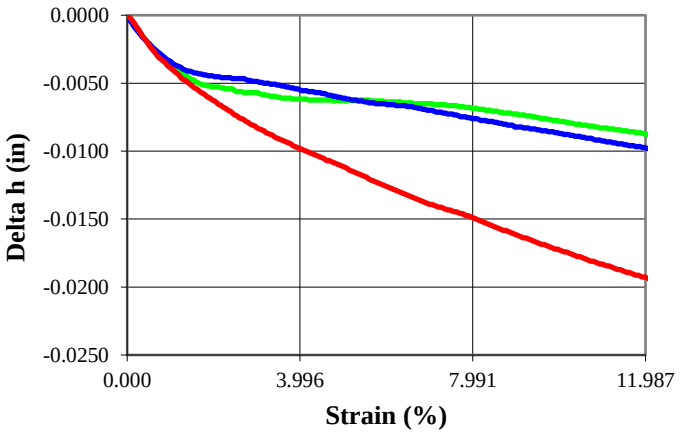
Date: 13-Nov-20

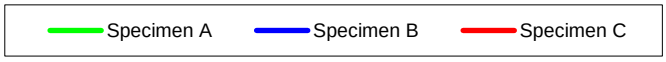
Job # 12188

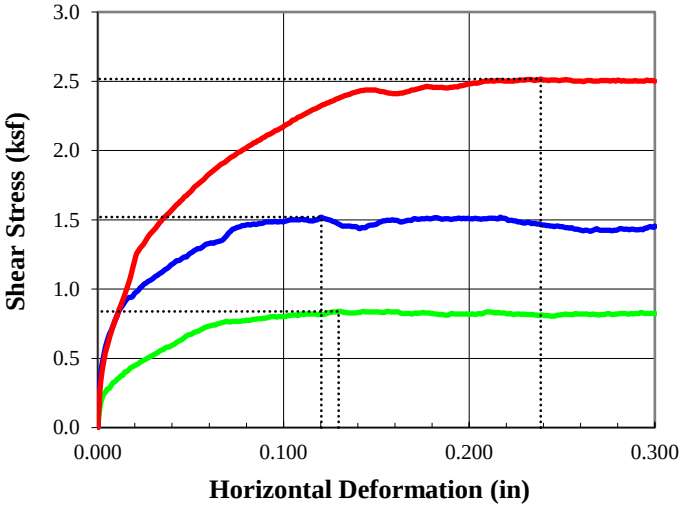
Figure: 51B

Direct Shear Test (ASTM D3080)









Initial	Specimen A	B	C
Moisture (%)	27.25	27.25	27.25
Dry Density (pcf)	83.95	88.62	86.91
Void Ratio	0.97	0.87	0.90
Saturation (%)	74.48	83.41	80.01
Diameter (in)	2.00	2.00	2.00
Height (in)	1.00	1.00	1.00

Final	A	B	C
Moisture (%)	25.75	25.75	25.75
Dry Density (pcf)	90.84	91.22	90.26
Void Ratio	0.82	0.81	0.83
Saturation (%)	83.2	84.0	82.0
Diameter (in)	2.00	2.00	2.00
Height (in)	0.96	0.94	0.86
Normal Load (ksf)	1.00	2.00	4.00
Shear Stress (ksf)(@Peak)	0.84	1.52	2.52
Peak Strain (%)	6.48	6.01	11.91
Rate (in/min)	0.0024	0.0024	0.0024
Peak Deformation (in.)	0.130	0.120	0.238

Sample Information	
Test Pit Number:	B-4
Sample Number:	-
Depth:	85 ft
Sample Type:	Remolded
Description:	Clay (CH)
Test Type:	Consolidated - Drained

Rock Loft Estates

-

CMT ENGINEERING LABORATORIES

Lab Data

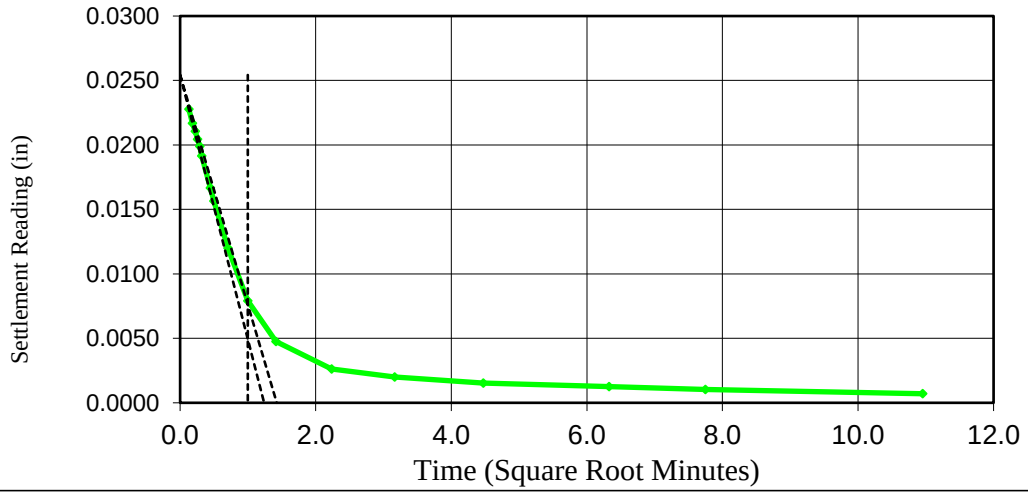
Date: 13-Nov-20

Job # 12188

Figure: 52A

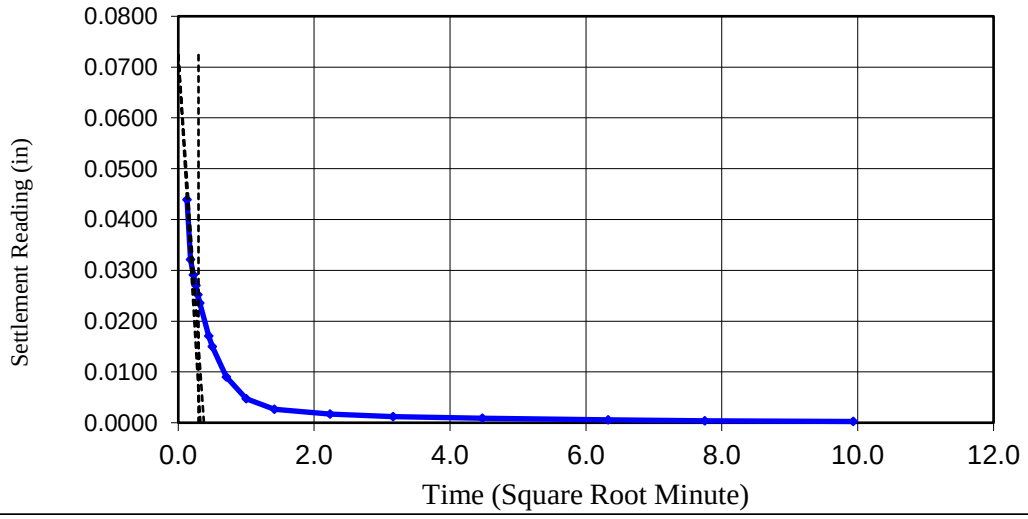
Direct Shear Test (ASTM D3080)

Consolidation Graph Specimen A



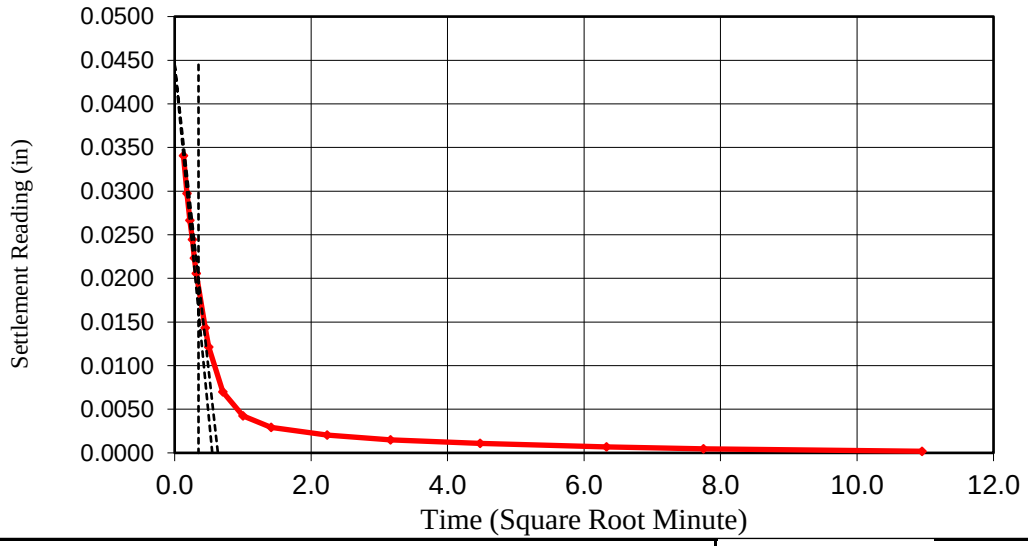
sqrt(t_{90}) =	1	
t_{90} =	1.0	min.
$t_f = 50\ t_{90} / 4.28$ =	11.7	min.
max $d_r = 0.5 / t_f$ =	0.0428	in./min.
selected d_r =	0.0024	in./min.
condition =	Drained	

Consolidation Graph Specimen B



sqrt(t_{90}) =	0.3	
t_{90} =	0.1	min.
$t_f = 50\ t_{90} / 4.28$ =	1.1	min.
max $d_r = 0.5 / t_f$ =	0.4756	in./min.
selected d_r =	0.0024	in./min.
condition =	Drained	

Consolidation Graph Specimen C



sqrt(t_{90}) =	0.35	
t_{90} =	0.1	min.
$t_f = 50\ t_{90} / 4.28$ =	1.4	min.
max $d_r = 0.5 / t_f$ =	0.3494	in./min.
selected d_r =	0.0024	in./min.
condition =	Drained	

Rock Loft Estates

-

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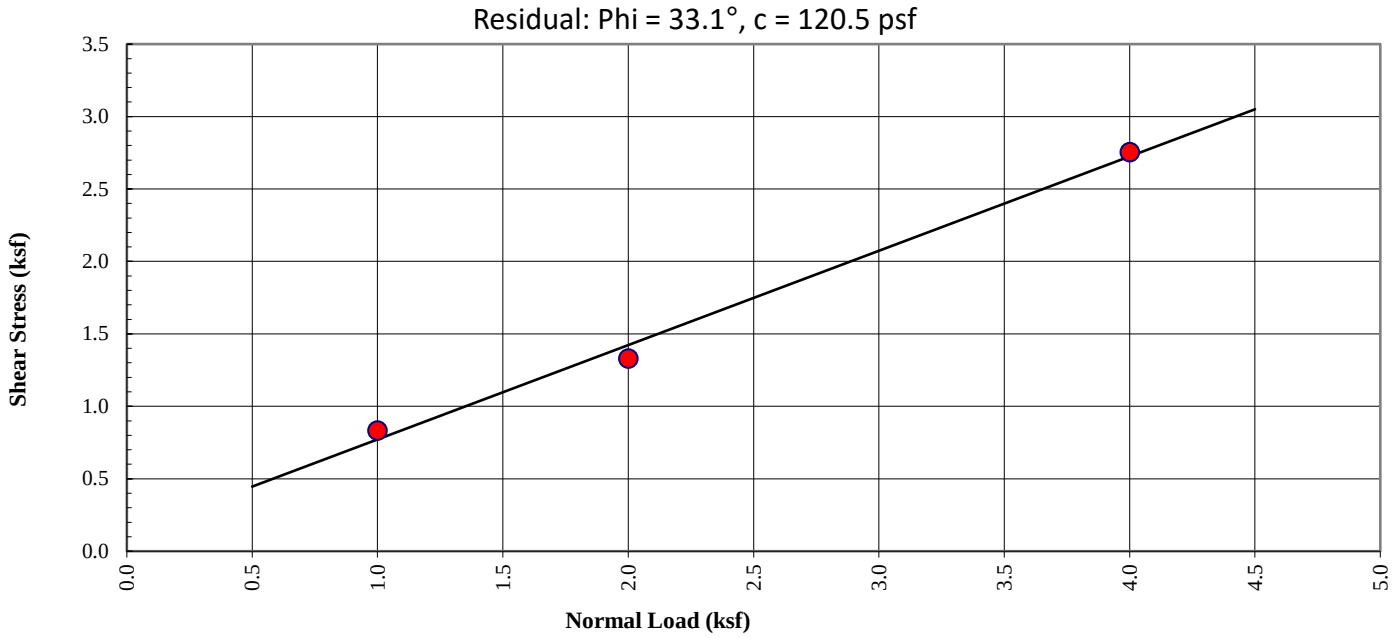
Lab Data

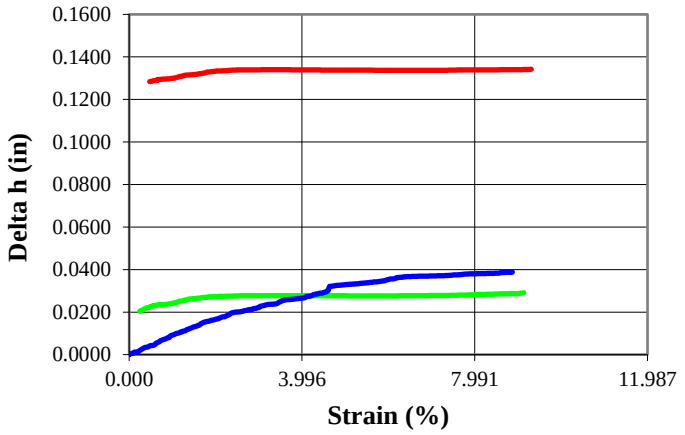
Date: 13-Nov-20

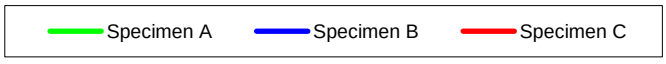
Job # 12188

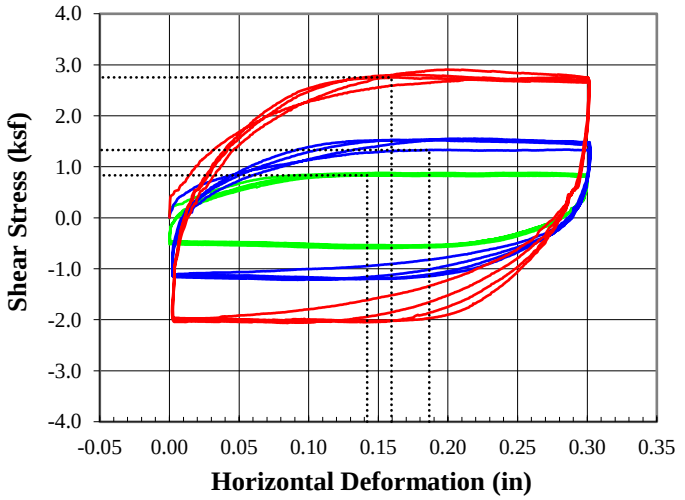
Figure: 52B

Direct Shear Test (ASTM D3080)









Initial	Specimen A	B	C
Moisture (%)	2.95	2.95	2.95
Dry Density (pcf)	98.31	99.11	96.62
Void Ratio	0.68	0.67	0.71
Saturation (%)	11.47	11.70	11.00
Diameter (in)	2.42	2.42	2.42
Height (in)	1.00	1.00	1.00

Final	A	B	C
Moisture (%)	22.53	22.53	22.53
Dry Density (pcf)	94.63	93.35	92.67
Void Ratio	0.75	0.77	0.78
Saturation (%)	79.9	77.4	76.1
Diameter (in)	2.42	2.42	2.42
Height (in)	1.00	1.00	0.97
Normal Load (ksf)	1.00	2.00	4.00
Shear Stress (ksf)(@Peak)	0.83	1.33	2.76
Peak Residual Strain (%)	5.86	7.71	6.59
Rate (in/min)	0.0024	0.0024	0.0024
Peak Deformation (in.)	0.142	0.187	0.160

Sample Information	
Test Pit Number:	North Trench
Sample Number:	
Depth:	Sta.0+15 @ 4 ft
Sample Type:	Remolded
Description:	Silty Gravel/Sand (GM/SM)
Test Type:	Consolidated - Drained

Rock Loft Estates

About 250 North 1450 East, Fruit Heights, Utah

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Lab Data

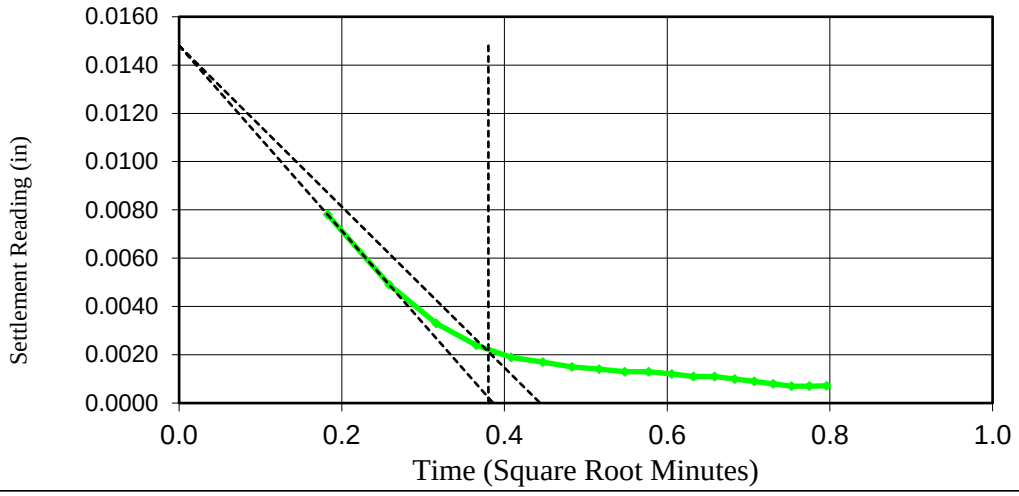
Date: 13-Nov-20

Job # 12188

Figure: 53A

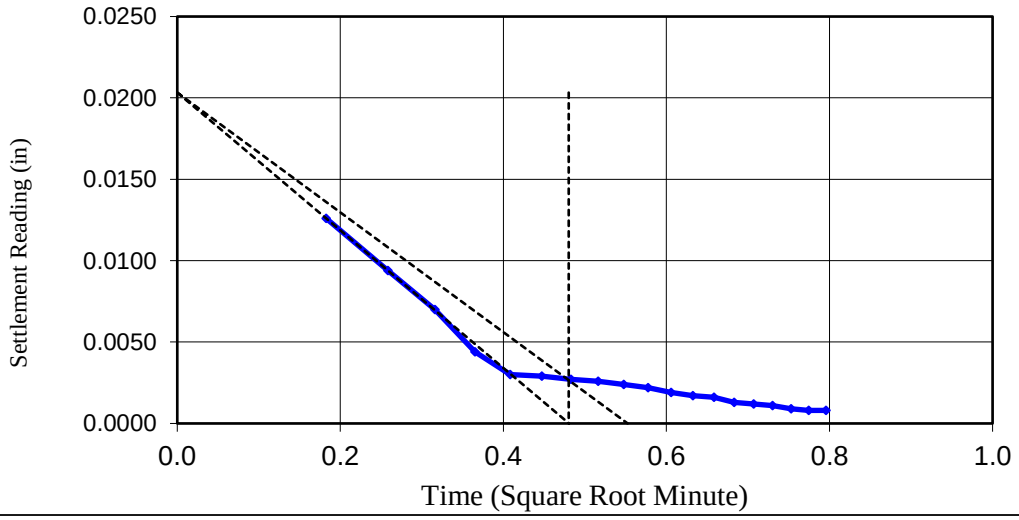
Direct Shear Test (ASTM D3080)

Consolidation Graph Specimen A



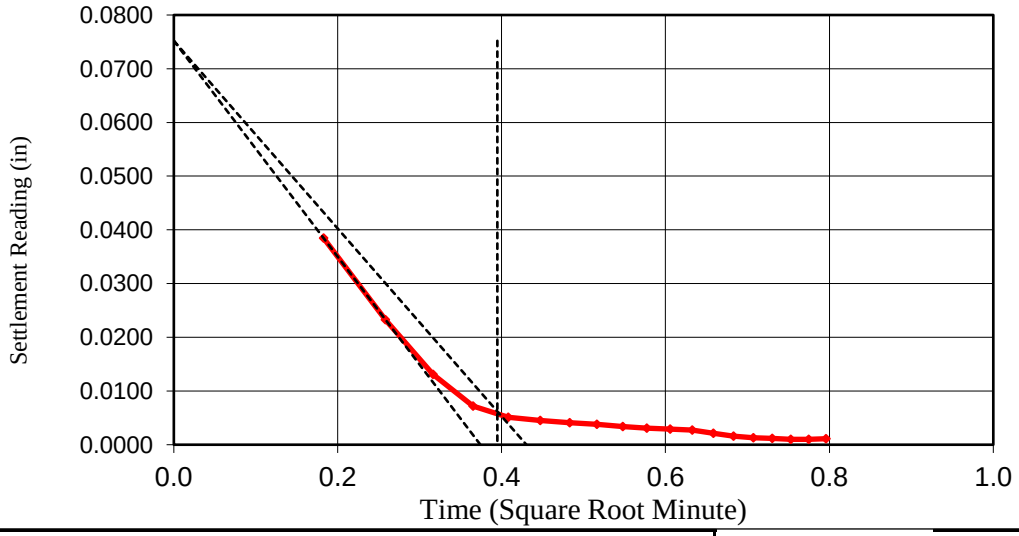
sqrt(t_{90}) =	0.38	
t_{90} =	0.1	min.
$t_f = 50\ t_{90} / 4.28$ =	1.7	min.
max $d_r = 0.5 / t_f$ =	0.2964	in./min.
selected d_r =	0.0024	in./min.

Consolidation Graph Specimen B



sqrt(t_{90}) =	0.48	
t_{90} =	0.2	min.
$t_f = 50\ t_{90} / 4.28$ =	2.7	min.
max $d_r = 0.5 / t_f$ =	0.1858	in./min.
selected d_r =	0.0024	in./min.

Consolidation Graph Specimen C



sqrt(t_{90}) =	0.395	
t_{90} =	0.2	min.
$t_f = 50\ t_{90} / 4.28$ =	1.8	min.
max $d_r = 0.5 / t_f$ =	0.2743	in./min.
selected d_r =	0.0024	in./min.

Rock Loft Estates

About 250 North 1450 East, Fruit Heights, Utah

CMT ENGINEERING LABORATORIES

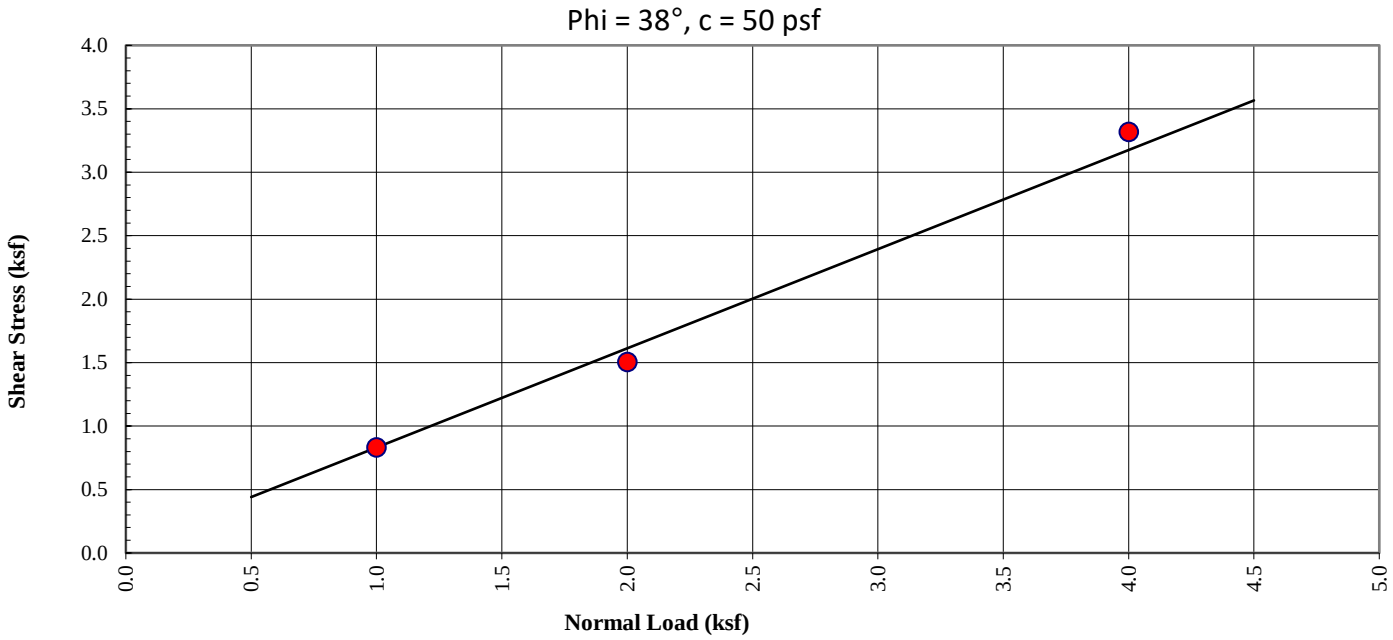
Lab Data

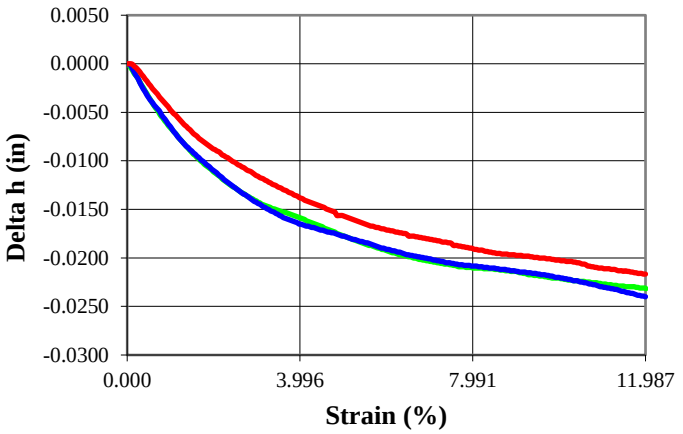
Date: 13-Nov-20

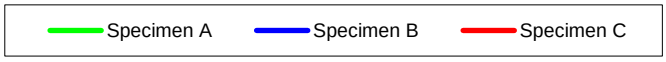
Job # 12188

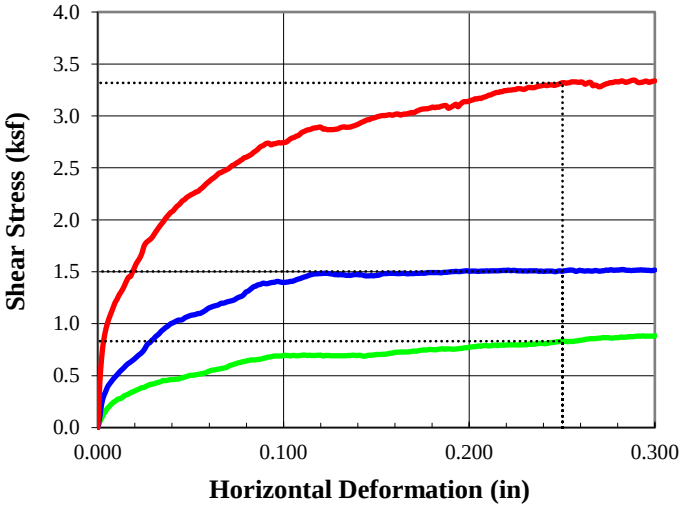
Figure: 53B

Direct Shear Test (ASTM D3080)









Initial	Specimen A	B	C
Moisture (%)	1.68	1.68	1.68
Dry Density (pcf)	111.51	109.99	109.68
Void Ratio	0.48	0.50	0.51
Saturation (%)	9.22	8.85	8.77
Diameter (in)	2.50	2.50	2.50
Height (in)	1.00	1.00	1.00

Final	A	B	C
Moisture (%)	15.18	15.18	15.18
Dry Density (pcf)	110.44	109.56	108.82
Void Ratio	0.50	0.51	0.52
Saturation (%)	80.9	79.0	77.4
Diameter (in)	2.50	2.50	2.50
Height (in)	0.95	0.92	0.87
Normal Load (ksf)	1.00	2.00	4.00
Shear Stress (ksf)(@0.25")	0.83	1.50	3.32
Strain at 0.25 in. (%)	9.99	7.71	9.99
Rate (in/min)	0.0033	0.0033	0.0033
Deformation (in.)	0.250	0.250	0.250

Sample Information	
Test Pit Number:	North Test Pit
Sample Number:	
Depth:	7 ft
Sample Type:	Remolded
Description:	Silty Gravel w/sand (GM)
Test Type:	Consolidated - Drained

Rock Loft Estates

About 250 North 1450 East, Fruit Heights, Utah

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Lab Data

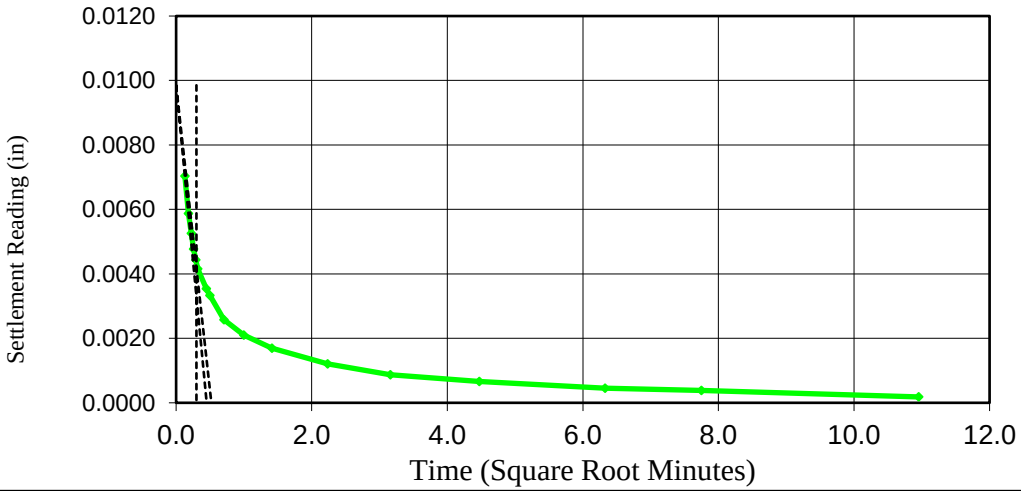
Date: 13-Nov-20

Job # 12188

Figure: 54A

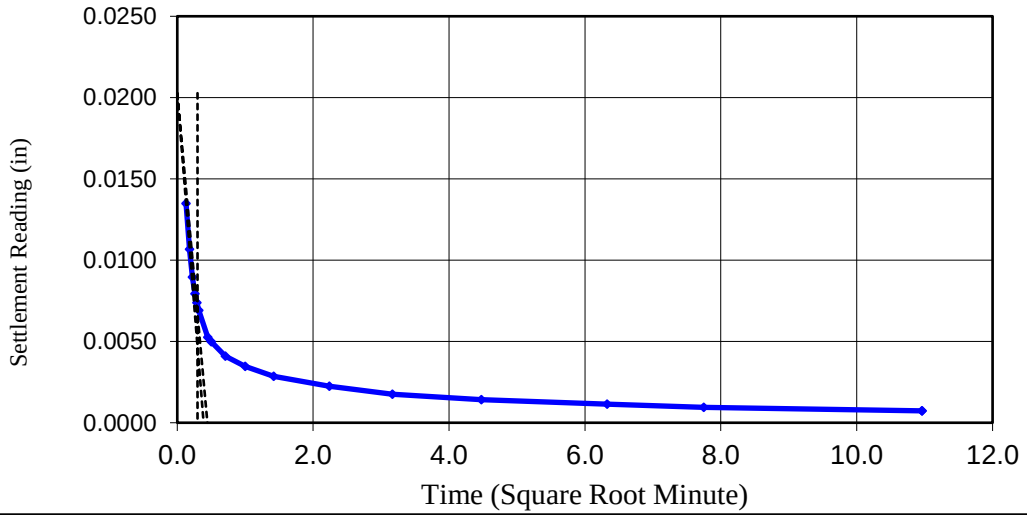
Direct Shear Test (ASTM D3080)

Consolidation Graph Specimen A



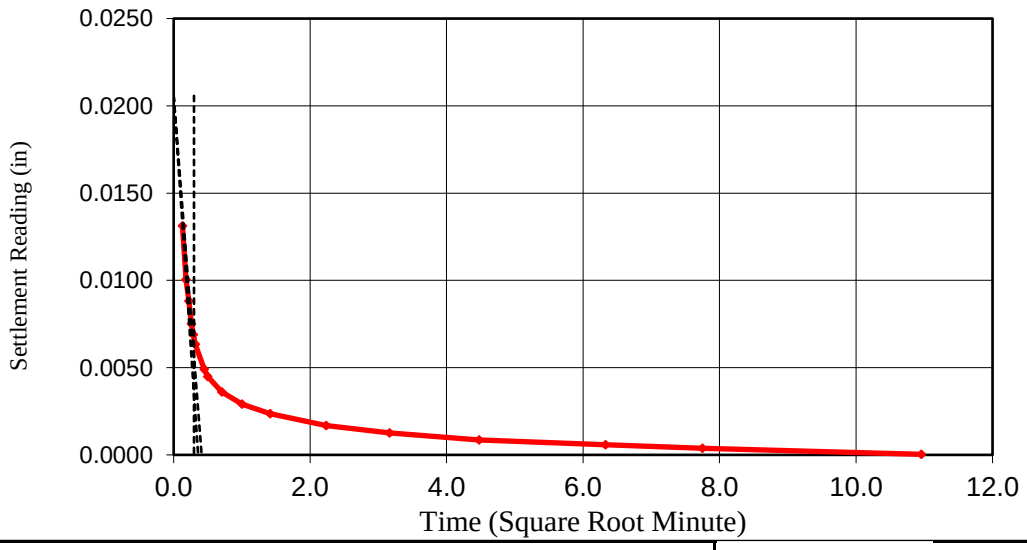
sqrt(t_{90}) =	0.3	
t_{90} =	0.1	min.
$t_f = 50\ t_{90} / 4.28$ =	1.1	min.
max $d_r = 0.5 / t_f$ =	0.4756	in./min.
selected d_r =	0.0033	in./min.
condition =	Drained	

Consolidation Graph Specimen B



sqrt(t_{90}) =	0.3	
t_{90} =	0.1	min.
$t_f = 50\ t_{90} / 4.28$ =	1.1	min.
max $d_r = 0.5 / t_f$ =	0.4756	in./min.
selected d_r =	0.0033	in./min.
condition =	Drained	

Consolidation Graph Specimen C



sqrt(t_{90}) =	0.3	
t_{90} =	0.1	min.
$t_f = 50\ t_{90} / 4.28$ =	1.1	min.
max $d_r = 0.5 / t_f$ =	0.4756	in./min.
selected d_r =	0.0033	in./min.
condition =	Drained	

Rock Loft Estates

About 250 North 1450 East, Fruit Heights, Utah

CMT ENGINEERING LABORATORIES

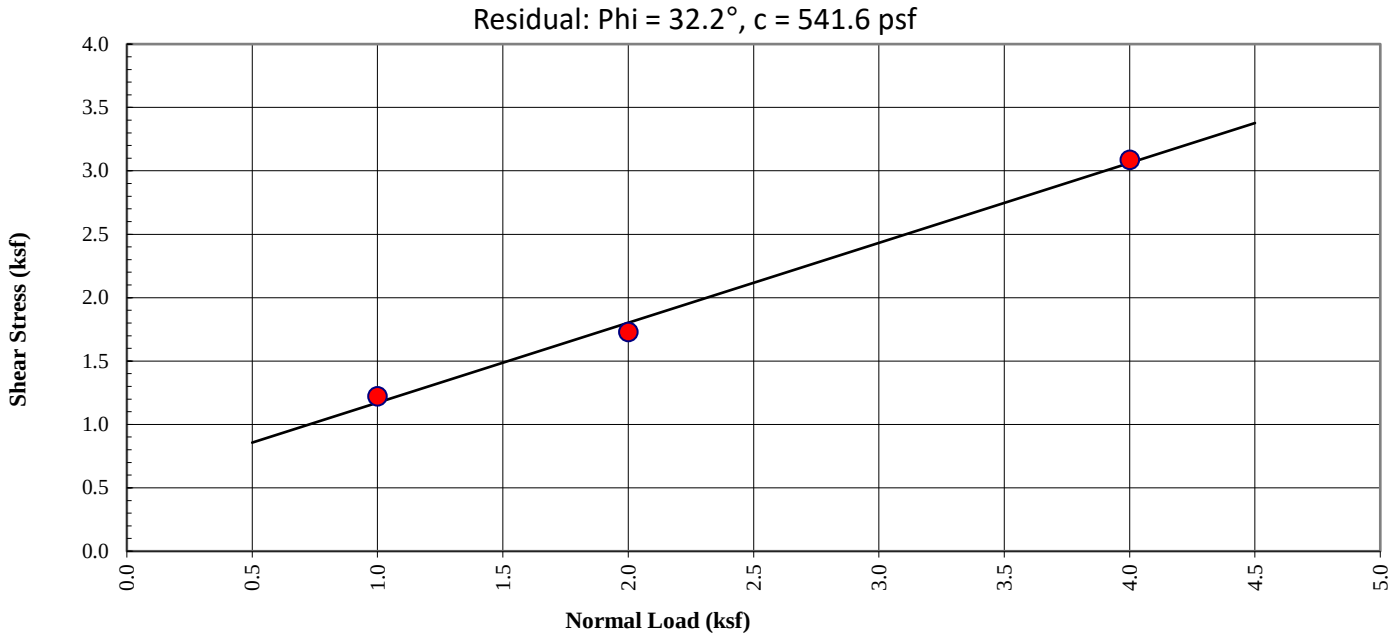
Lab Data

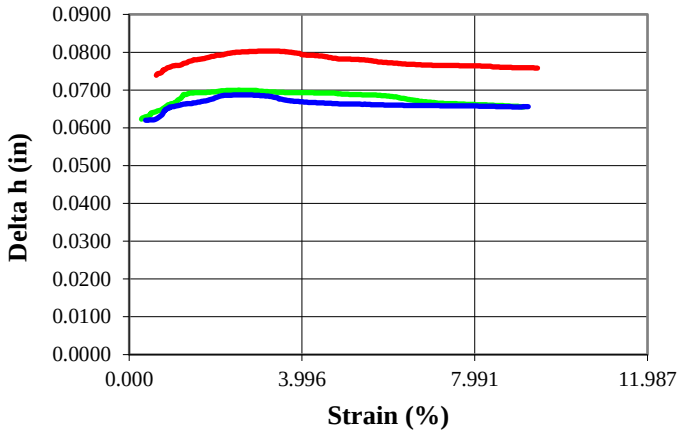
Date: 13-Nov-20

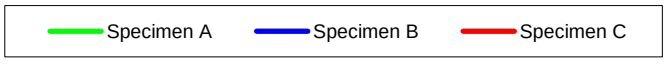
Job # 12188

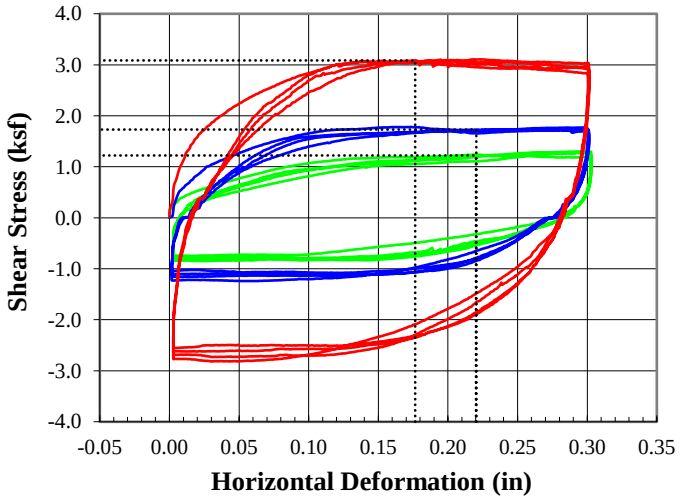
Figure: 54B

Direct Shear Test (ASTM D3080)









Initial	Specimen A	B	C
Moisture (%)	20.96	20.96	20.96
Dry Density (pcf)	108.19	105.45	107.50
Void Ratio	0.53	0.57	0.54
Saturation (%)	105.11	97.77	103.21
Diameter (in)	2.42	2.42	2.42
Height (in)	1.00	1.00	1.00

Final	A	B	C
Moisture (%)	17.17	17.17	17.17
Dry Density (pcf)	107.52	106.46	108.09
Void Ratio	0.54	0.55	0.53
Saturation (%)	84.6	82.2	85.8
Diameter (in)	2.42	2.42	2.42
Height (in)	1.00	1.00	0.97
Normal Load (ksf)	1.00	2.00	4.00
Shear Stress (ksf)(@Peak)	1.22	1.73	3.09
Peak Residual Strain (%)	9.10	9.10	7.29
Rate (in/min)	0.0024	0.0024	0.0024
Peak Deformation (in.)	0.220	0.220	0.176

Sample Information	
Test Pit Number:	South Trench
Sample Number:	
Depth:	Sta.0+02 @ 8 ft
Sample Type:	Remolded
Description:	Sandy Gravel with silt (GW-GM)
Test Type:	Consolidated - Drained

Rock Loft Estates

About 250 North 1450 East, Fruit Heights, Utah

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Lab Data

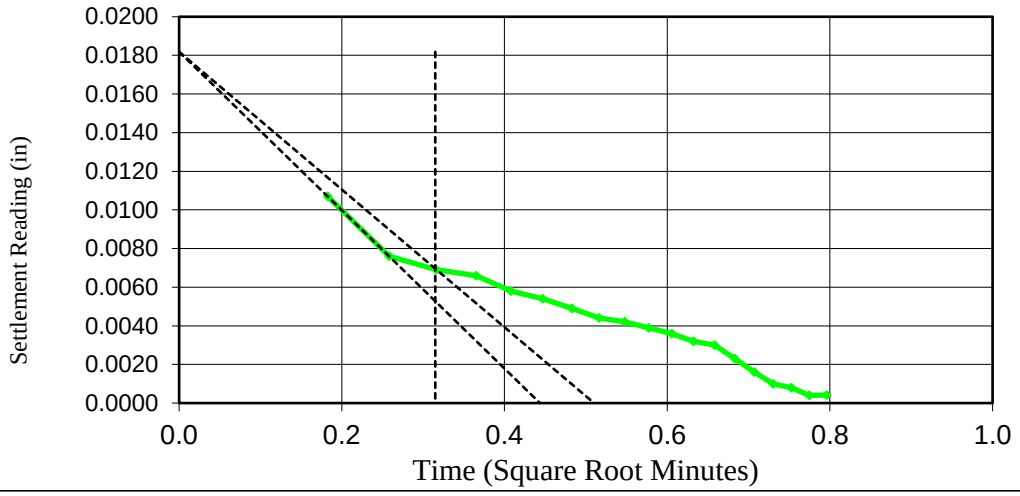
Date: 13-Nov-20

Job # 12188

Figure: 55A

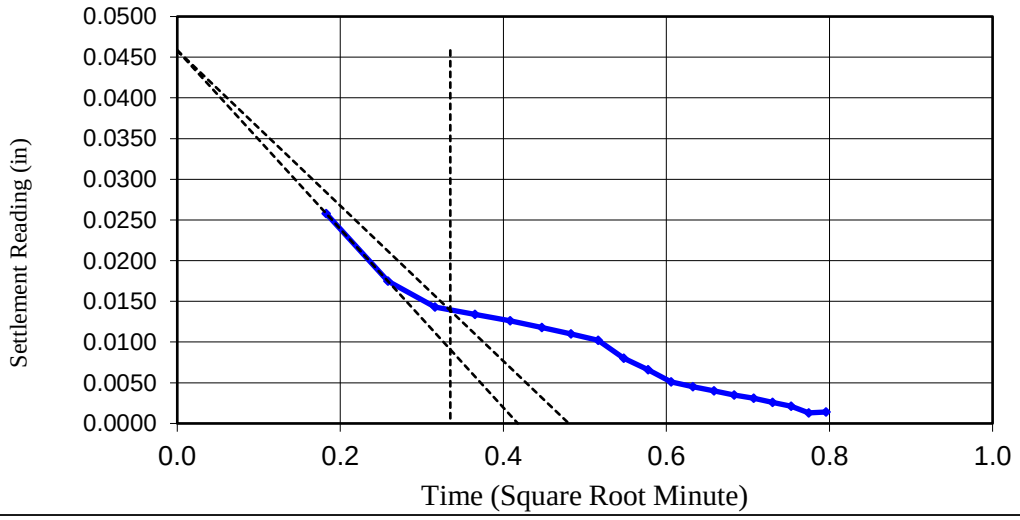
Direct Shear Test (ASTM D3080)

Consolidation Graph Specimen A



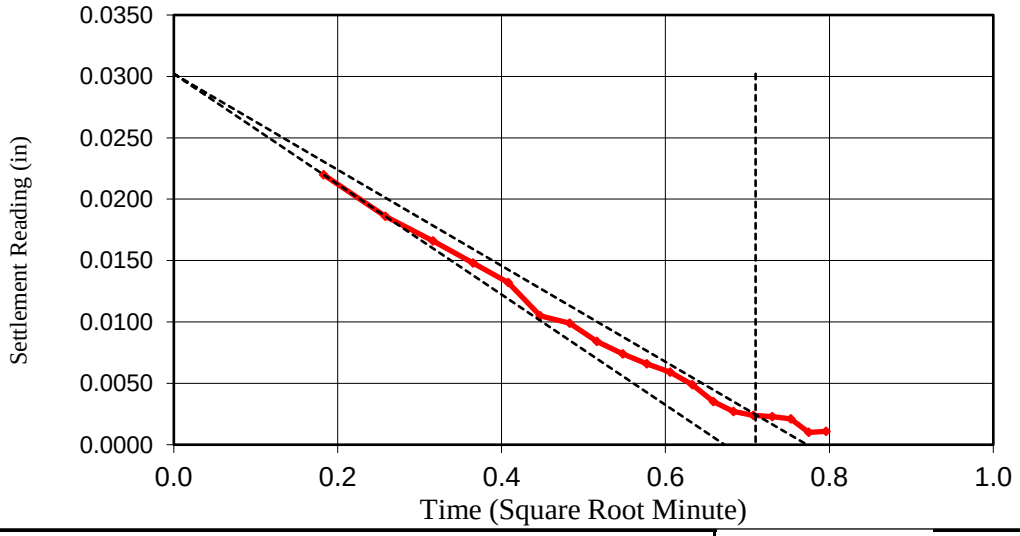
sqrt(t_{90}) =	0.315	
t_{90} =	0.1	min.
$t_f = 50\ t_{90} / 4.28$ =	1.2	min.
max $d_r = 0.5 / t_f$ =	0.4313	in./min.
selected d_r =	0.0024	in./min.

Consolidation Graph Specimen B



sqrt(t_{90}) =	0.335	
t_{90} =	0.1	min.
$t_f = 50\ t_{90} / 4.28$ =	1.3	min.
max $d_r = 0.5 / t_f$ =	0.3814	in./min.
selected d_r =	0.0024	in./min.

Consolidation Graph Specimen C



sqrt(t_{90}) =	0.71	
t_{90} =	0.5	min.
$t_f = 50\ t_{90} / 4.28$ =	5.9	min.
max $d_r = 0.5 / t_f$ =	0.0849	in./min.
selected d_r =	0.0024	in./min.

Rock Loft Estates

About 250 North 1450 East, Fruit Heights, Utah

CMT ENGINEERING LABORATORIES

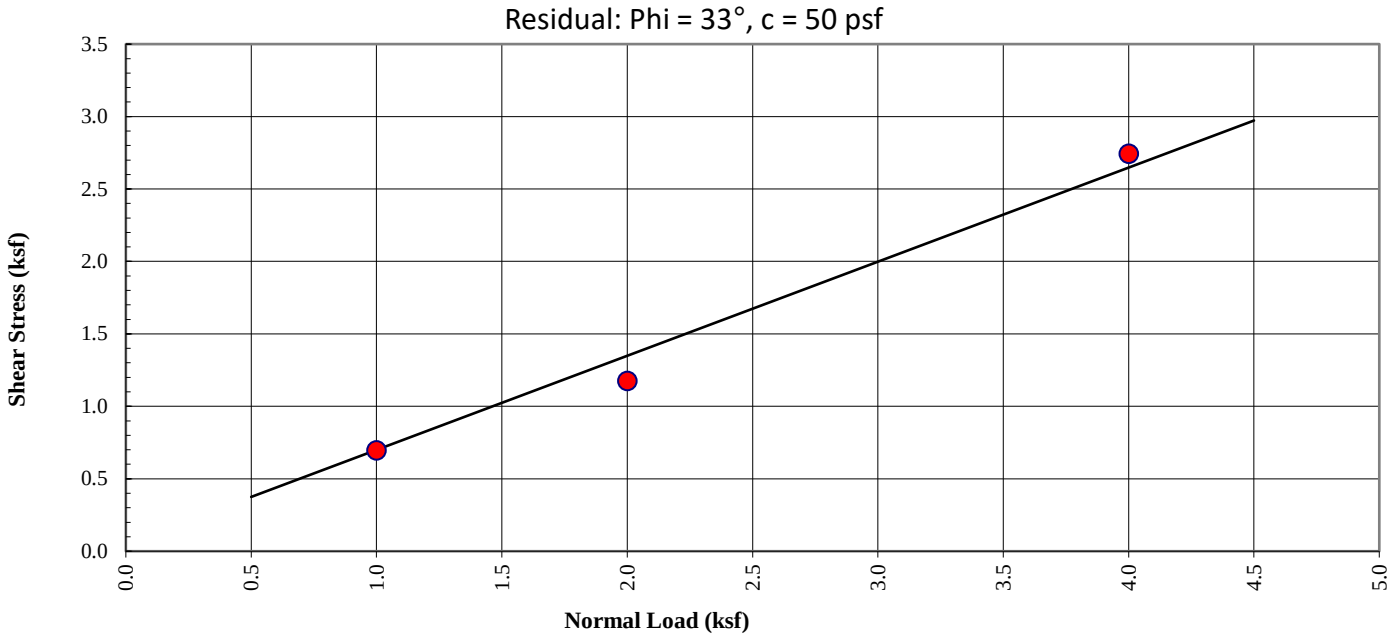
Lab Data

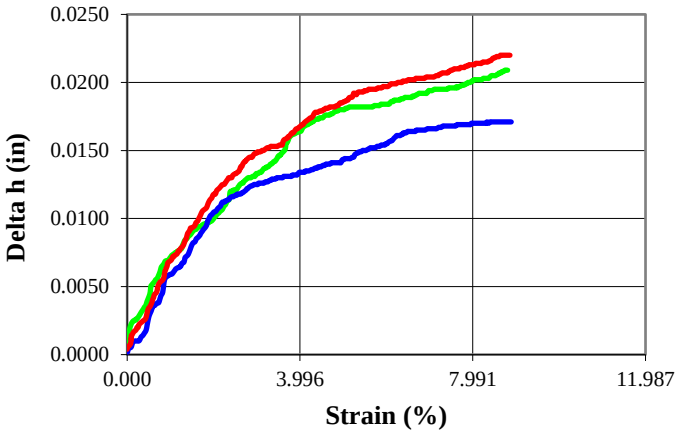
Date: 13-Nov-20

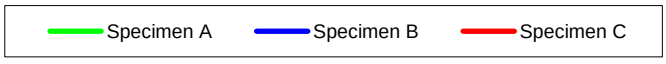
Job # 12188

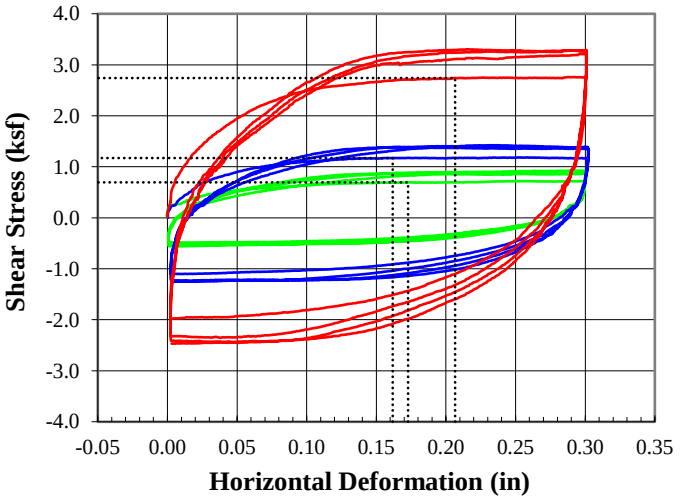
Figure: 55B

Direct Shear Test (ASTM D3080)









Initial	Specimen A	B	C
Moisture (%)	19.97	19.97	19.97
Dry Density (pcf)	93.47	94.30	92.78
Void Ratio	0.77	0.75	0.78
Saturation (%)	68.83	70.25	67.67
Diameter (in)	2.42	2.42	2.42
Height (in)	1.00	1.00	1.00

Final	A	B	C
Moisture (%)	21.74	21.74	21.74
Dry Density (pcf)	97.22	92.25	92.12
Void Ratio	0.70	0.79	0.80
Saturation (%)	82.2	72.7	72.5
Diameter (in)	2.42	2.42	2.42
Height (in)	1.00	1.00	0.97
Normal Load (ksf)	1.00	2.00	4.00
Shear Stress (ksf)(@Peak)	0.70	1.17	2.74
Peak Residual Strain (%)	7.14	6.69	8.54
Rate (in/min)	0.0024	0.0024	0.0024
Peak Deformation (in.)	0.173	0.162	0.207

Sample Information	
Test Pit Number:	South Trench
Sample Number:	
Depth:	Sta.0+28 @ 6.5 ft
Sample Type:	Remolded
Description:	Sandy Gravel with silt (GW-GM)
Test Type:	Consolidated - Drained

Rock Loft Estates

About 250 North 1450 East, Fruit Heights, Utah

CMT ENGINEERING LABORATORIES

Lab Data

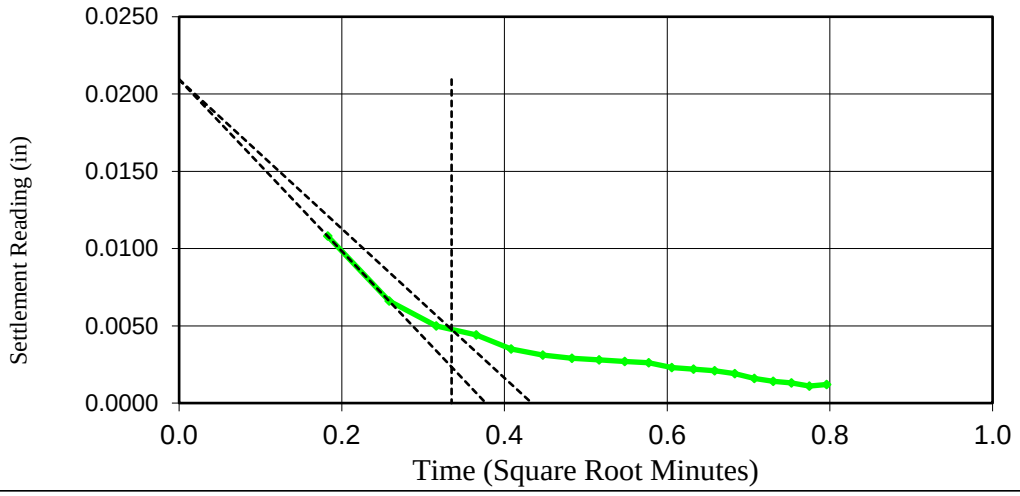
Date: 13-Nov-20

Job # 12188

Figure: 56A

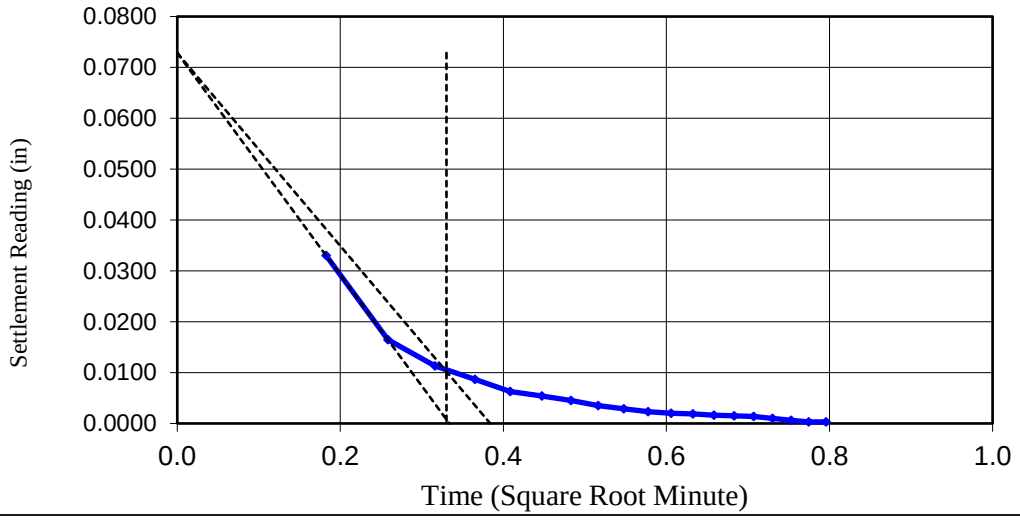
Direct Shear Test (ASTM D3080)

Consolidation Graph Specimen A



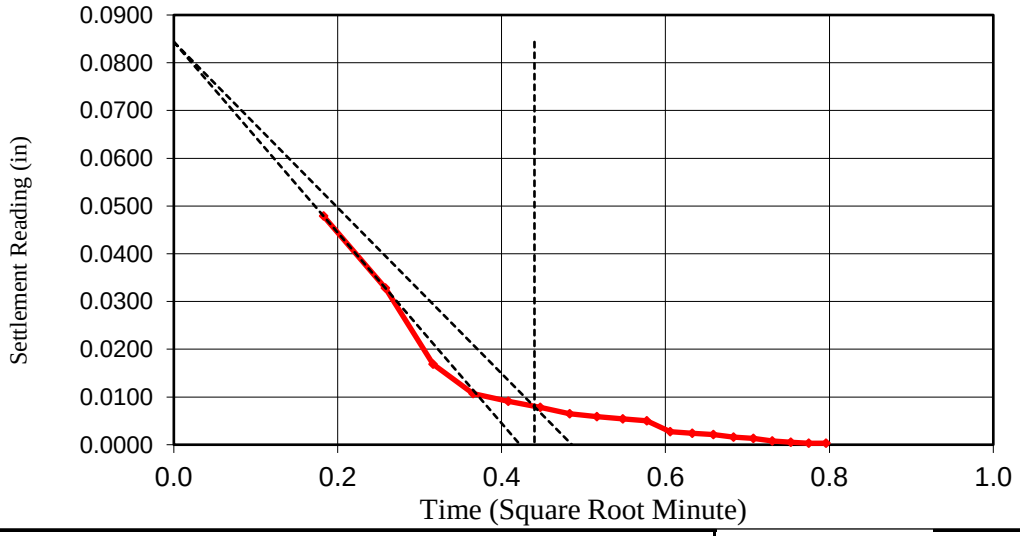
sqrt(t_{90}) =	0.335	
t_{90} =	0.1	min.
t_f = 50 t_{90} / 4.28 =	1.3	min.
max d_r = 0.5 / t_f =	0.3814	in./min.
selected d_r =	0.0024	in./min.

Consolidation Graph Specimen B



sqrt(t_{90}) =	0.33	
t_{90} =	0.1	min.
t_f = 50 t_{90} / 4.28 =	1.3	min.
max d_r = 0.5 / t_f =	0.3930	in./min.
selected d_r =	0.0024	in./min.

Consolidation Graph Specimen C



sqrt(t_{90}) =	0.44	
t_{90} =	0.2	min.
t_f = 50 t_{90} / 4.28 =	2.3	min.
max d_r = 0.5 / t_f =	0.2211	in./min.
selected d_r =	0.0024	in./min.

Rock Loft Estates

About 250 North 1450 East, Fruit Heights, Utah

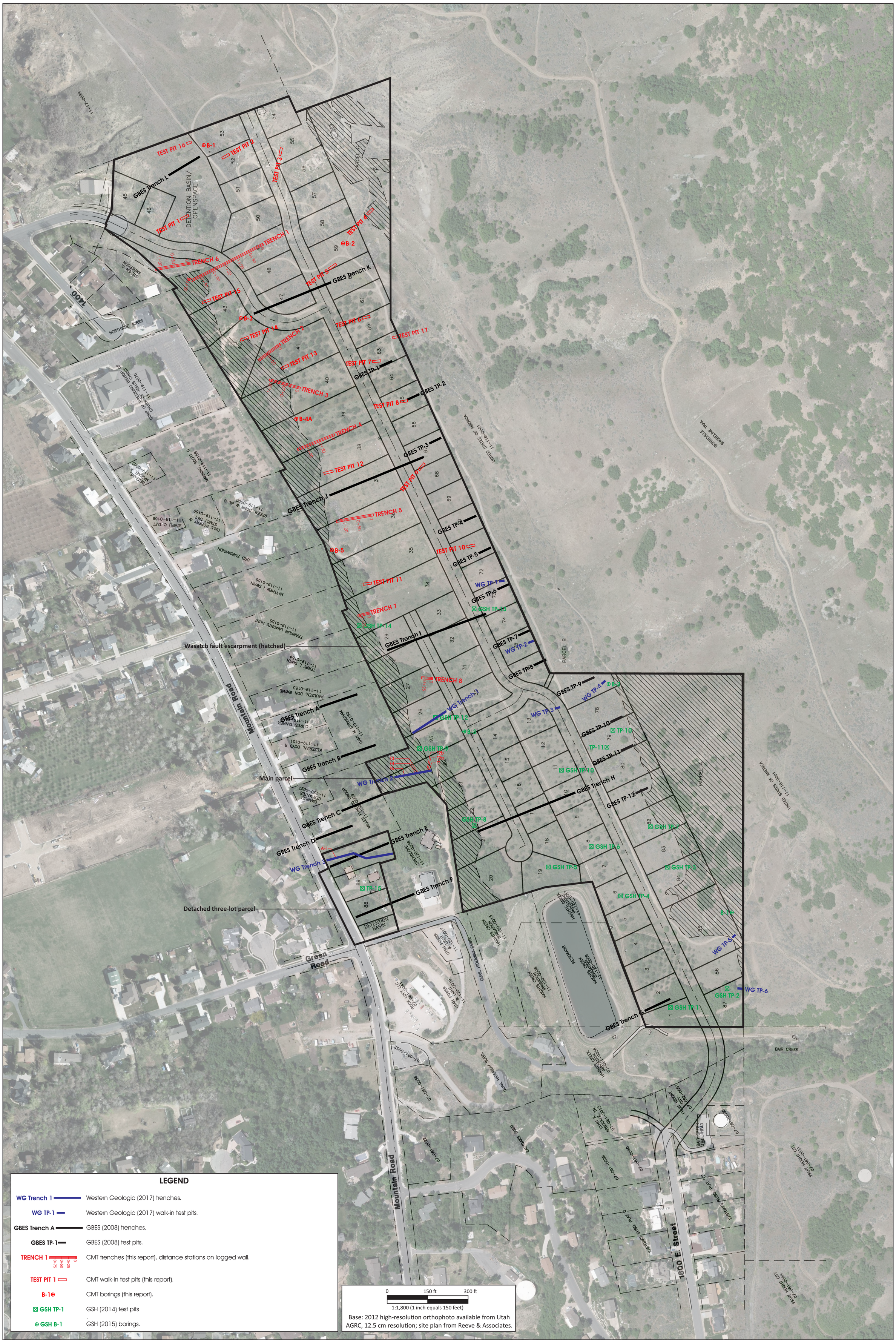
CMT ENGINEERING LABORATORIES

Lab Data

Date: 13-Nov-20

Job # 12188

Figure: 56B



CMT ENGINEERING LABORATORIES

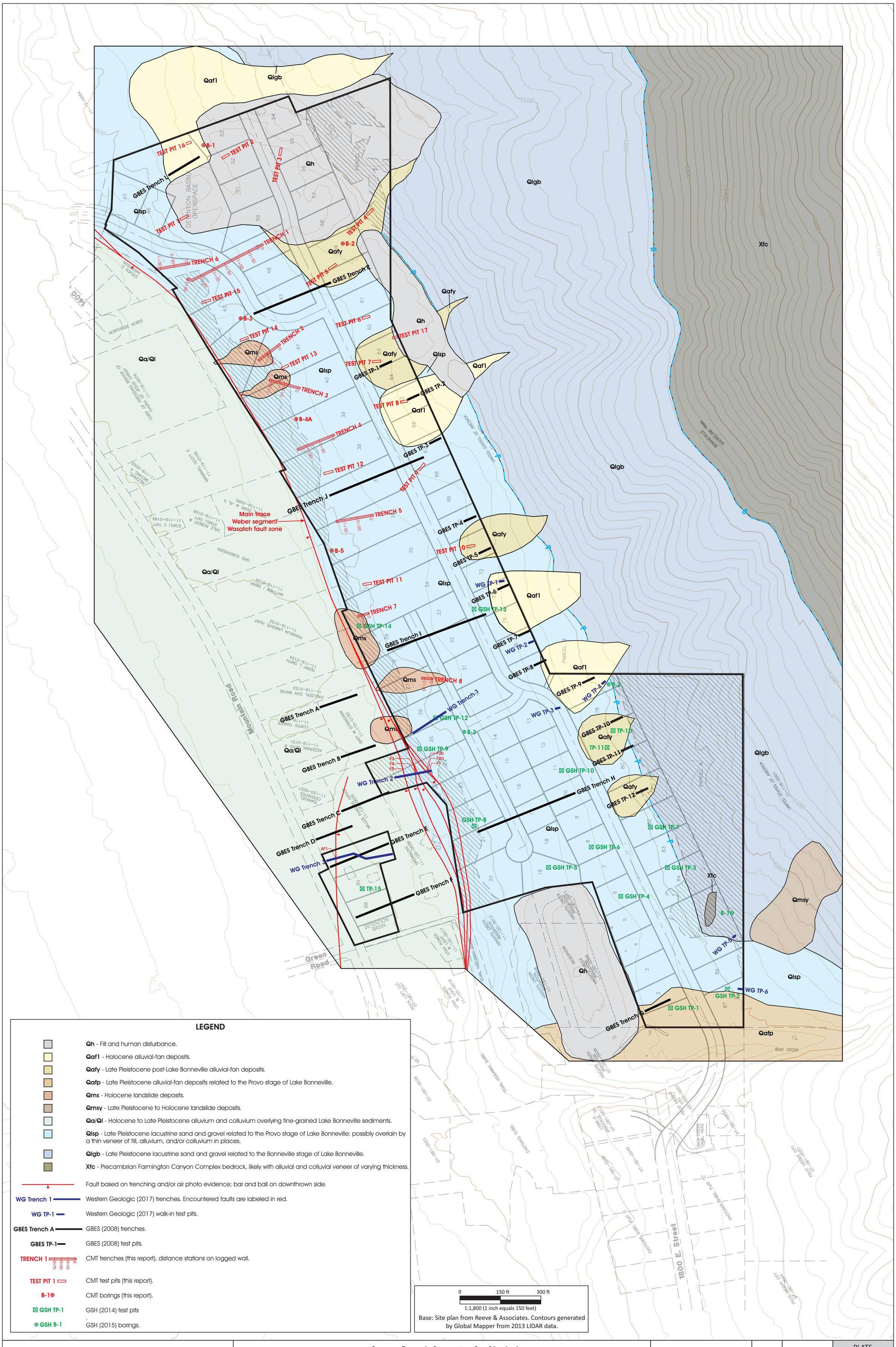
Rock Loft Ridge Subdivision

About 250 North 1450 East, Fruit Heights, Utah

SITE EVALUATION

Date:	12-Jan-2021
CMT No.:	12188

PLATE 1



LEGEND

- **Qh** - Fill and human disturbance.
- **Qaf1** - Holocene alluvial-fan deposits.
- **Qafy** - Late Pleistocene post-Lake Bonneville alluvial-fan deposits.
- **Qafp** - Late Pleistocene alluvial-fan deposits related to the Provo stage of Lake Bonneville.
- **Qms** - Holocene landslide deposits.
- **Qmsy** - Late Pleistocene to Holocene landslide deposits.
- **Qa/Ql** - Holocene to Late Pleistocene alluvium and colluvium overlying fine-grained Lake Bonneville sediments.
- **Qlsp** - Late Pleistocene lacustrine sand and gravel related to the Provo stage of Lake Bonneville; possibly overlain by a thin veneer of fill, alluvium, and/or colluvium in places.
- **Qlgb** - Late Pleistocene lacustrine sand and gravel related to the Bonneville stage of Lake Bonneville.
- **Xfc** - Precambrian Farmington Canyon Complex bedrock, likely with alluvial and colluvial veneer of varying thickness.

Fault based on trenching and/or air photo evidence; bar and ball on downthrown side.

WG Trench 1 — Western Geologic (2017) trenches. Encountered faults are labeled in red.

WG TP-1 — Western Geologic (2017) walk-in test pits.

GBES Trench A — GBES (2008) trenches.

GBES TP-1 — GBES (2008) test pits.

TRENCH 1 — CMT trenches (this report), distance stations on logged wall.

TEST PIT 1 — CMT test pits (this report).

B-1⊕ — CMT borings (this report).

☒ GSH TP-1 — GSH (2014) test pits

⊕ GSH B-1 — GSH (2015) borings.

0 150 ft 300 ft

1:1,800 (1 inch equals 150 feet)

Base: Site plan from Reeve & Associates. Contours generated by Global Mapper from 2013 LIDAR data.

CMT ENGINEERING LABORATORIES

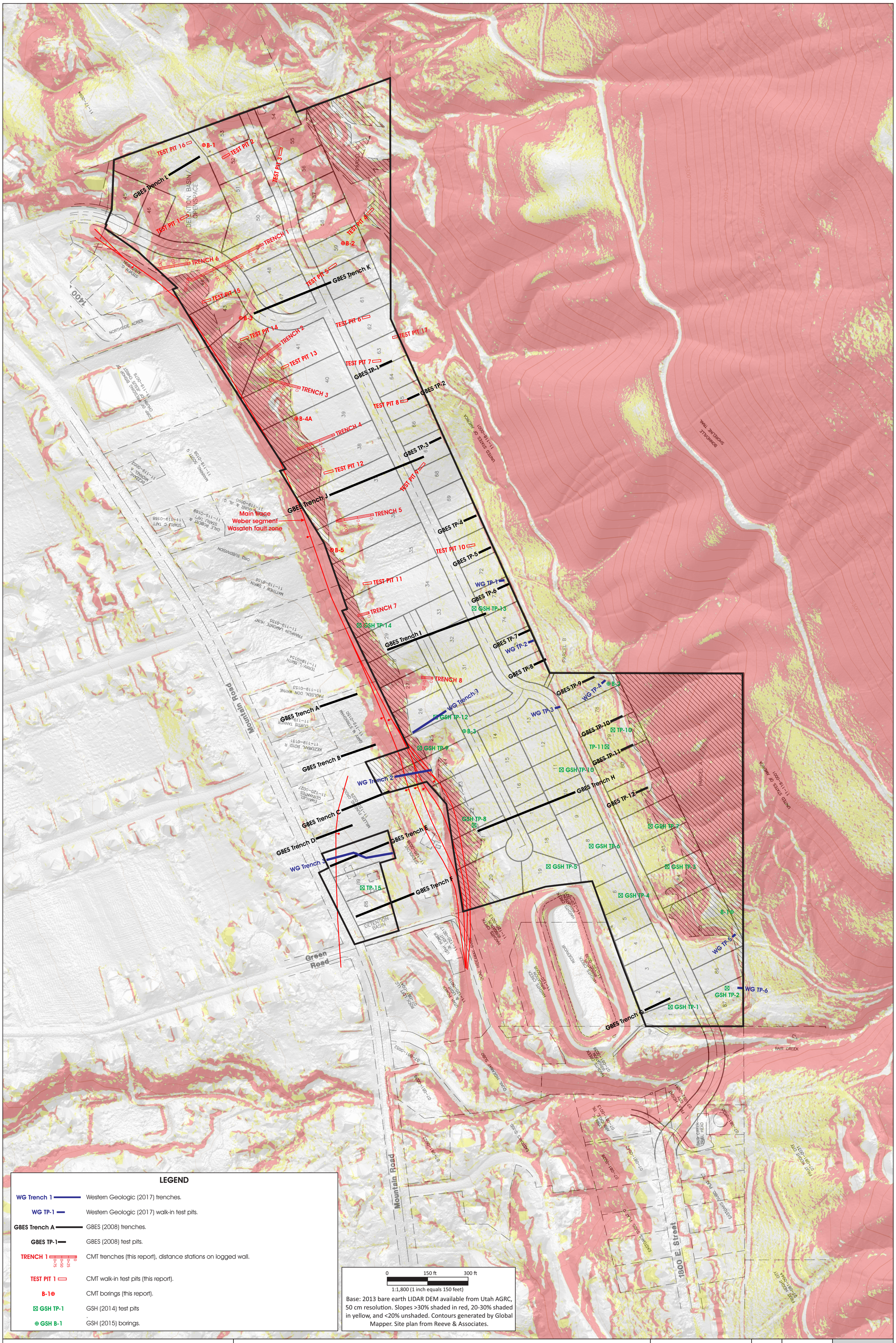
Rock Loft Ridge Subdivision

About 250 North 1450 East, Fruit Heights, Utah

SITE-SPECIFIC GEOLOGIC MAP

Date:	12-Jan-2021
CMT No.:	12188

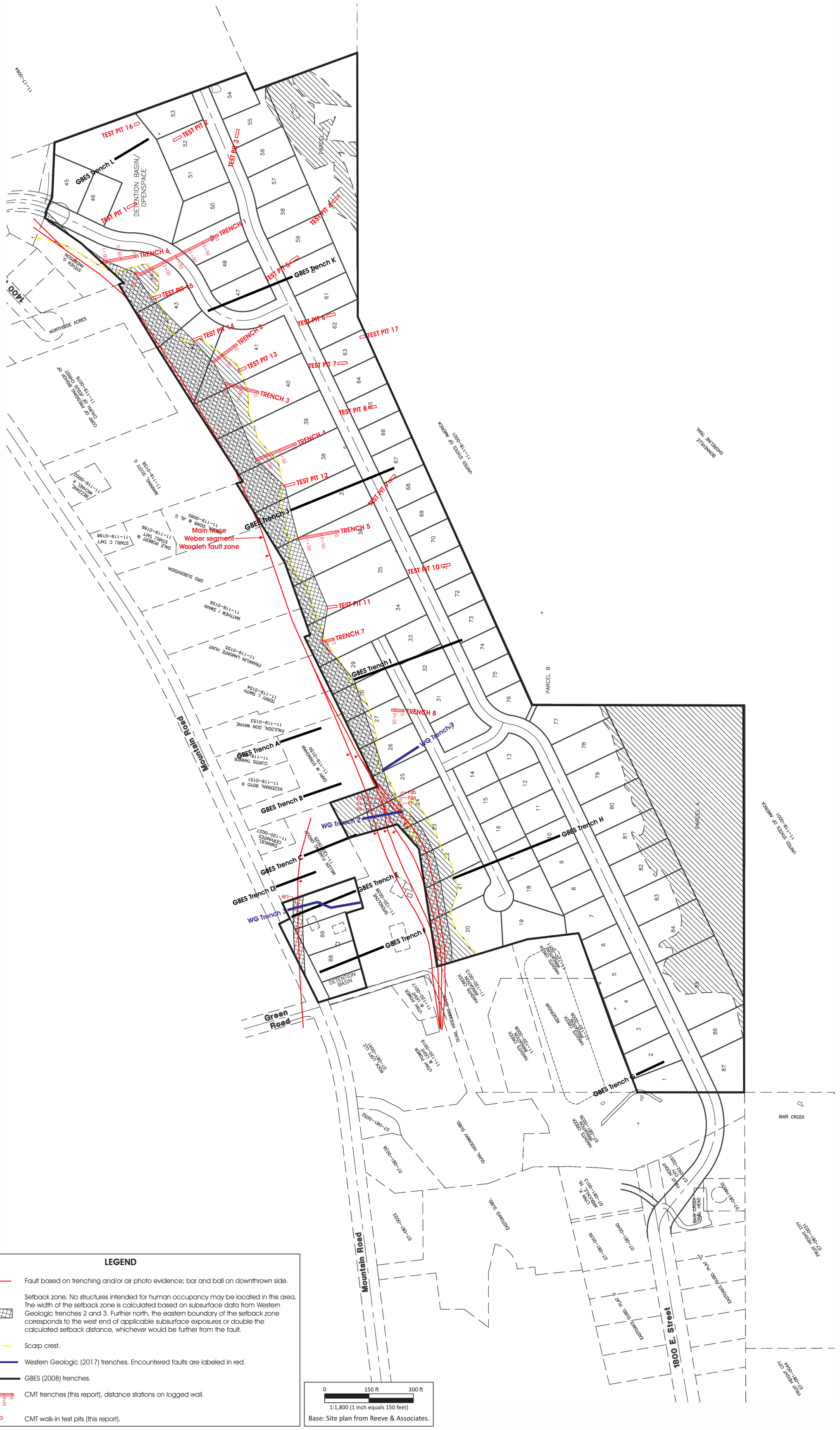
PLATE 2



LEGEND

WG Trench 1	Western Geologic (2017) trenches.
WG TP-1	Western Geologic (2017) walk-in test pits.
GBES Trench A	GBES (2008) trenches.
GBES TP-1	GBES (2008) test pits.
TRENCH 1 (0, 0+25, 0+50, 0+75)	CMT trenches (this report), distance stations on logged wall.
TEST PIT 1	CMT walk-in test pits (this report).
B-1	CMT borings (this report).
GSH TP-1	GSH (2014) test pits
GSH B-1	GSH (2015) borings.

CMT ENGINEERING LABORATORIES	**Rock Loft Ridge Subdivision** About 250 North 1450 East, Fruit Heights, Utah	**LIDAR ANALYSIS**	Date: 12-Jan-2021 CMT No.: 12188 PLATE 3



LEGEND

- Fault based on trenching and/or air photo evidence; bar and ball on downthrown side.
- Setback zone. No structures intended for human occupancy may be located in this area. The width of the setback zone is calculated based on subsurface data from Western Geologic trenches 2 and 3. Further north, the eastern boundary of the setback zone corresponds to the west end of applicable subsurface exposures or double the calculated setback distance, whichever would be further from the fault.
- Scarp crest.
- WG Trench 1 — Western Geologic (2017) trenches. Encountered faults are labeled in red.
- GBES Trench A — GBES (2008) trenches.
- TRENCH 1 (0, 0+25, 0+50, 0+75) — CMT trenches (this report), distance stations on logged wall.
- TEST PIT 1 — CMT walk-in test pits (this report).

CMT ENGINEERING LABORATORIES

Rock Loft Ridge Subdivision

About 250 North 1450 East, Fruit Heights, Utah

SURFACE FAULT RUPTURE HAZARD

Date:	12-Jan-2021
CMT No.:	12188

PLATE 4



LEGEND

Fan A (△24")	Alluvial fan and identifier (see Figure 3). For crosshatched lots: (1) building pads should be raised by at least the value shown in inches; (2) homes should include no upslope-facing, below-grade windows or entryways that may present a pathway for debris to enter sub-grade areas; and (3) grading should incorporate the debris-flow and runoff mitigation features indicated in the typical lot exhibit in the upper right-hand corner.
	Debris fencing.
WG Trench 1	Western Geologic (2017) trenches.
GBES Trench A	GBES (2008) trenches.
GBES TP-1	GBES (2008) test pits.
TRENCH 1 (0, 0+25, 0+50, 0+75)	CMT trenches (this report), distance stations on logged wall.
TEST PIT 1	CMT Test Pits (this report).

Typical Lot
SCALE: 1"=30'

Typical Cut-off Ditch
SCALE: 1"=3'

Typical Side Channel Ditch
SCALE: 1"=3'

Rock Loft Estates
FRUIT HEIGHTS CITY, DAVIS COUNTY, UTAH

Typical Lot Run-off/ Debris Flow Mitigation Plan

Date: November 6, 2020 | Project#: 4825-12
Engineer: J. Nate Reeve, P.E. | Drawn: R. Hansen

Sheet 1 of 1 Sheets

Reeve & Associates, Inc.
54 South 100 West, Bountiful, Utah 84010
TEL: (801) 621-3100 FAX: (801) 621-2666
LAND PLANNERS • CIVIL ENGINEERS • LAND SURVEYORS
WATER ENGINEERS • STRUCTURAL ENGINEERS • LANDSCAPE ARCHITECTS

CMT ENGINEERING LABORATORIES

Rock Loft Ridge Subdivision

About 250 North 1450 East, Fruit Heights, Utah

DEBRIS FLOW HAZARD

Date: 12-Jan-2021
CMT No.: 12188

PLATE 5



LEGEND

- Mapped landslide deposits (Plate 2).
- Slope crest (from Reeve & Associates).
- WG Trench 1 — Western Geologic (2017) trenches.
- WG TP-1 — Western Geologic (2017) walk-in test pits.
- GBES Trench A — GBES (2008) trenches.
- GBES TP-1 — GBES (2008) test pits.
- TRENCH 1 (0, 0+25, 0+50, 0+75) — CMT trenches (this report), distance stations on logged wall.
- TEST PIT 1 — CMT walk-in test pits (this report).
- B-1⊕ — CMT borings (this report).
- ☒ GSH TP-1 — GSH (2014) test pits
- ⊕ GSH B-1 — GSH (2015) borings.

CMT ENGINEERING LABORATORIES

Rock Loft Ridge Subdivision

About 250 North 1450 East, Fruit Heights, Utah

PRE-GRADING LANDSLIDE HAZARD

Date:	12-Jan-2021
CMT No.:	12188

PLATE 6A

Cut/Fill Report

Generated: 2021-01-04 16:35:02

By user: nficklin

Drawing: G:\4825\12 Raymond Property, Fruit Heights\Improvements\G:\4825\12 Raymond Property, Fruit Heights\Improvements\RockLoft_PrelimGrading.dwg

Volume Summary

Name	Type	Cut Factor	Fill Factor	2d Area (Sq. Ft.)	Cut (Cu. Yd.)	Fill (Cu. Yd.)	Net (Cu. Yd.)
Cut and Fill	full	1.000	1.000	2735797.16	328494.80	371378.82	42884.02<Fill>

Totals

	2d Area (Sq. Ft.)	Cut (Cu. Yd.)	Fill (Cu. Yd.)	Net (Cu. Yd.)
Total	2735797.16	328494.80	371378.82	42884.02<Fill>

* Value adjusted by cut or fill factor other than 1.0

Elevations Table			
Number	Minimum Elevation	Maximum Elevation	Color
1	−37.353	−9.001	
2	−9.000	−5.001	
3	−5.000	−2.501	
4	−2.500	−0.001	
5	0.000	2.500	
6	2.501	5.000	
7	5.001	9.000	
8	9.001	15.000	
9	15.001	35.000	
10	35.001	56.565	

Cut Slope Failures on A-A

Pseudostatic Stability Line with 0"

Pseudostatic Stability Line with 2"

Pseudostatic Stability Line with 5"

Static Stability Line

Main trace
Weber segment
Wasatch fault zone

Green Road

Mountain Road

1800 E. Street

BAIR CREEK

PARCEL A

PARCEL B

UNITED STATES OF AMERICA 11-118-0001

LEGEND

- Mapped landslide deposits (Plate 2).
- Native slope crest (from Reeve & Associates).
- Static and dynamic stability distances at expected displacement in inches (<min. FS to west, >min. FS to east).
- Slope stability lines at various parameters.
- WG Trench 1 — Western Geologic (2017) trenches.
- WG TP-1 — Western Geologic (2017) walk-in test pits.
- A — A — Post-grading cross sections.
- GBES Trench A — GBES (2008) trenches.
- GBES TP-1 — GBES (2008) test pits.
- TRENCH 1 — CMT trenches (this report), distance stations on logged wall.
- TEST PIT 1 — CMT walk-in test pits (this report).
- B-1⊕ — CMT borings (this report).
- ⊠ GSH TP-1 — GSH (2014) test pits
- ⊕ GSH B-1 — GSH (2015) borings.

0 150 ft 300 ft

1:1,800 (1 inch equals 150 feet)

Base: Grading plan from Reeve & Associates.

CMT ENGINEERING LABORATORIES

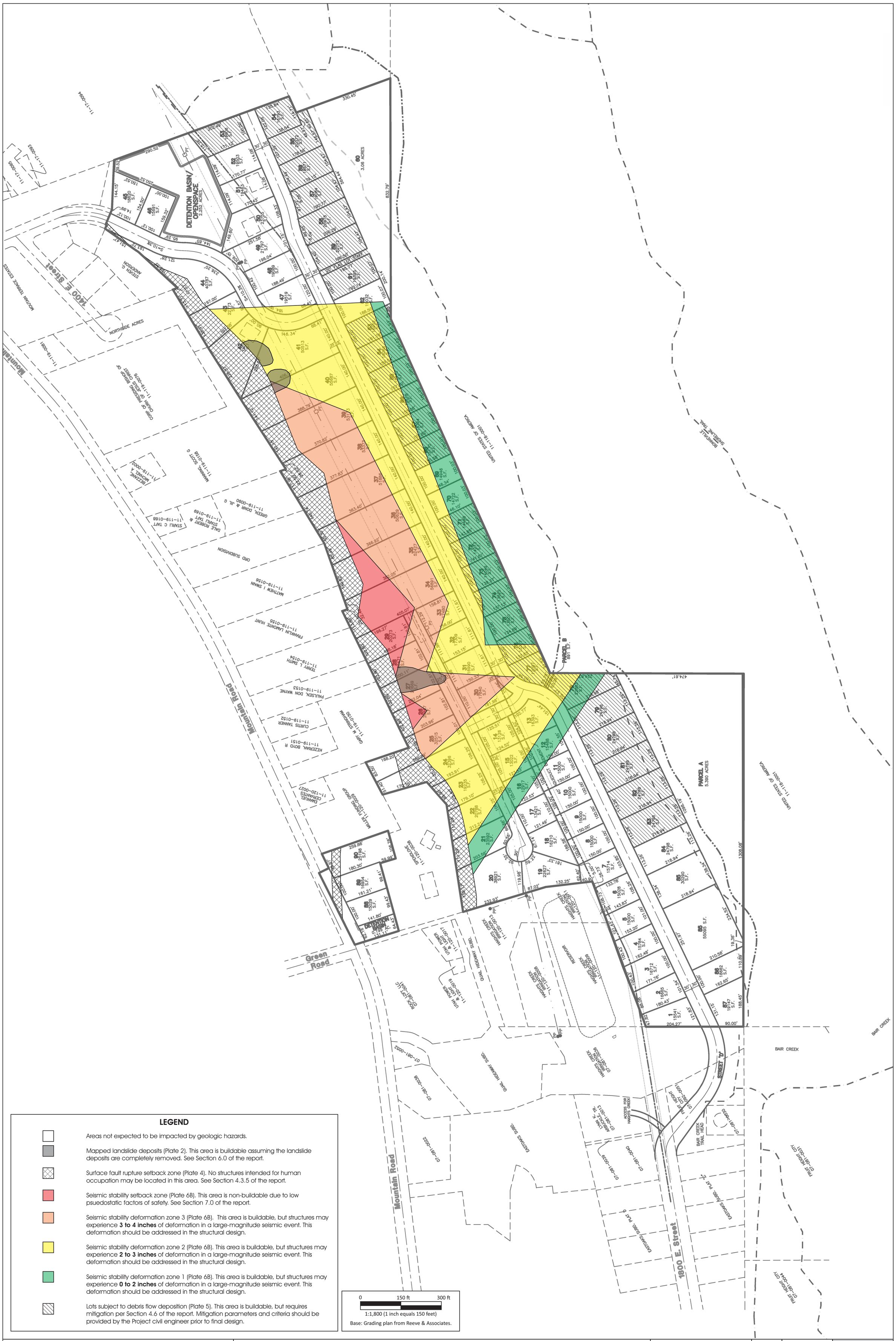
Rock Loft Ridge Subdivision

About 250 North 1450 East, Fruit Heights, Utah

POST-GRADING LANDSLIDE HAZARD

Date:	15-Jan-2021
CMT No.:	12188

PLATE 6B



LEGEND

- Areas not expected to be impacted by geologic hazards.
- Mapped landslide deposits (Plate 2). This area is buildable assuming the landslide deposits are completely removed. See Section 6.0 of the report.
- Surface fault rupture setback zone (Plate 4). No structures intended for human occupation may be located in this area. See Section 4.3.5 of the report.
- Seismic stability setback zone (Plate 6B). This area is non-buildable due to low psuedostatic factors of safety. See Section 7.0 of the report.
- Seismic stability deformation zone 3 (Plate 6B). This area is buildable, but structures may experience **3 to 4 inches** of deformation in a large-magnitude seismic event. This deformation should be addressed in the structural design.
- Seismic stability deformation zone 2 (Plate 6B). This area is buildable, but structures may experience **2 to 3 inches** of deformation in a large-magnitude seismic event. This deformation should be addressed in the structural design.
- Seismic stability deformation zone 1 (Plate 6B). This area is buildable, but structures may experience **0 to 2 inches** of deformation in a large-magnitude seismic event. This deformation should be addressed in the structural design.
- Lots subject to debris flow deposition (Plate 5). This area is buildable, but requires mitigation per Section 4.6 of the report. Mitigation parameters and criteria should be provided by the Project civil engineer prior to final design.

CMT ENGINEERING LABORATORIES

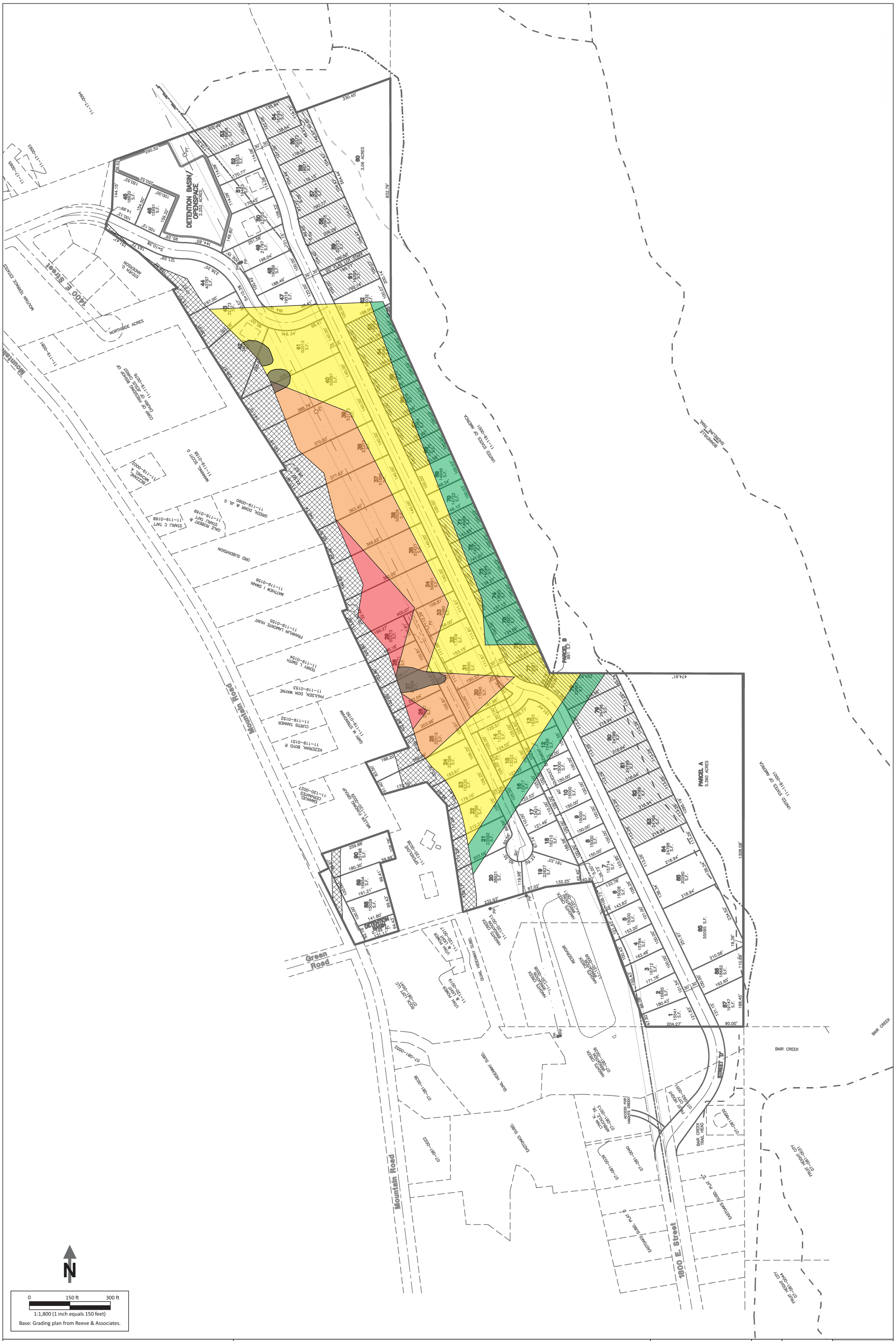
Rock Loft Ridge Subdivision

About 250 North 1450 East, Fruit Heights, Utah

HAZARDS SUMMARY

Date:	29-Jan-2021
CMT No.:	12188

PLATE 7



CMT ENGINEERING LABORATORIES

Rock Loft Ridge Subdivision

About 250 North 1450 East, Fruit Heights, Utah

HAZARDS SUMMARY

Date:	29-Jan-2021
CMT No.:	12188

PLATE 7A

HAZARDS SUMMARY LEGEND

Areas not expected to be impacted by geologic hazards.

Mapped landslide deposits (Plate 2). These areas are buildable assuming the landslide deposits are completely removed. See Section 6.0 of the report.

Surface fault rupture setback zone (Plate 4). No structures intended for human occupation may be located in these areas. See Section 4.3.5 of the report.

Seismic stability setback zone (Plate 6B). These areas are non-buildable due to low psuedostatic factors of safety. See Section 7.0 of the report.

Seismic stability deformation zone 3 (Plate 6B). These areas are buildable, but structures may experience **3 to 4 inches** of deformation in a large-magnitude seismic event. This deformation should be addressed in the structural design.

Seismic stability deformation zone 2 (Plate 6B). These areas are buildable, but structures may experience **2 to 3 inches** of deformation in a large-magnitude seismic event. This deformation should be addressed in the structural design.

Seismic stability deformation zone 1 (Plate 6B). These areas are buildable, but structures may experience **0 to 2 inches** of deformation in a large-magnitude seismic event. This deformation should be addressed in the structural design.

Lots subject to debris flow deposition (Plate 5). These areas are buildable, but require mitigation per Section 4.6 of the report. Mitigation parameters and criteria should be provided by the Project civil engineer prior to final design.

APPENDIX B

PREVIOUS
REPORTS

Great Basin Earth Science, Inc.



GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE FRUIT HEIGHTS, UTAH

Prepared For:

Cherry Ridge, LLC.
510 South 200 West, Suite 200
Salt Lake City, Utah 84101

Prepared By:

Great Basin Earth Science, Inc.
2241 East Bendemere Circle
Salt Lake City, Utah 84109

July 21, 2008

Great Basin Earth Science Job No. 07-013

Great Basin Earth Science, Inc.

2241 East Bendemere Circle
Salt Lake City, Utah 84109
Phone (801) 599-2189
Fax (801) 487-8473
elips@gbearthscience.com

July 21, 2008 GBES Job No. 07-013

Nicole Cottle
Cherry Ridge, LLC.
510 South 200 West, Suite 200
Salt Lake City, Utah 84101

RE: Report - Geologic Hazards Evaluation
Cherry Ridge
Fruit Height, Utah

Dear Ms. Cottle:

1.0 INTRODUCTION

This report presents the results of an evaluation of the geologic hazards conducted by Great Basin Earth Science, Inc. (GBES) for the proposed development of the Cherry Ridge property in Fruit Heights, Utah.

1.1 Objectives and Scope

The objectives of this investigation were to evaluate the hazards associated with surface fault rupture and tectonic deformation, landslides, debris flows, rock falls, and seismic seiche and storm surge. Other hazards possibly present at the site were not evaluated and are beyond the scope of this investigation.

A preliminary geologic hazards investigation was conducted by GBES (2008) for this property. This investigation concluded that surface fault rupture and tectonic deformation, landslide, debris flow, and rock fall hazards potentially existed at the site, and recommended that additional studies be conducted prior to site development. Where applicable, the results of the preliminary geologic hazards report (GBES, 2008) are incorporated into this report.

In order to evaluate the geologic hazards listed above, our scope of work has included the following:

1. Review available geologic maps and published reports, and inspect aerial photographs.
2. Conduct field inspections of surface topographic, geologic, and hydrologic features.

3. Excavation and logging of twelve trenches in order to identify the presence or absence of active faults, assess the zone of deformation, and recommend appropriate fault setback distances should active faults exist.
4. Excavate and log twelve test pits in order to identify the presence of, and characterize, recent debris flow deposits on alluvial fans in order to estimate thickness of flows, maximum boulder size, and approximate ages of flows.
5. Conduct analysis of potential sediment volume for future debris flows.
6. Conduct surface observations and assessment of previously identified landslides on property.
7. Evaluation and correlation of available data and preparation of a summary report.

The objectives and scope of our study were presented in our April 21, 2008 proposal to Cherry Ridge, LLC.

1.2 Project Description

The property consists of an irregular shaped parcel approximately 120 acres in size located in Fruit Heights. The general location of the project site with respect to major topographic features and existing facilities, as of 1998, is presented on Figure 1, Vicinity Map. The project site is located in Section 36 of Township 4 North, Range 1 West, Salt Lake Base and Meridian.

We understand that the proposed development at this site will consist of roads and construction of single-family residential structures.

2.0 SITE DESCRIPTION

The property is located at the base of the Wasatch Range, just north of Bair Canyon. The property boundaries and topographic and cultural features are shown on Figure 2, Site Plan. Elevations range between approximately 4,690 feet in the south west corner of the property to about 5,100 feet along the eastern edge of the property. Surface drainage is to the west ultimately towards the Great Salt Lake. Vegetation consists of native species (scrub oak, brush, and grasses) and fruit orchards. Slope gradients vary greatly across the site and in some locations are in excess of 30 percent. The property generally consists of two gently-sloping surfaces (slope to the west) separated by a steep 40-60 foot west-dipping escarpment.

3.0 METHODOLOGY

3.1 Review of Literature, Aerial Photographs, and Maps

The U.S. Geological Survey (Nelson and Personius, 1993) has published a surficial geologic map of the area. This report includes a map at a scale of 1:50,000, brief descriptions of the surficial geologic units, and a discussion of the Wasatch fault zone. The Utah Geological Survey

(Solomon, 2007) has published a more detailed surficial geologic map of part of the Kaysville Quadrangle. This map, at a scale of 1:24,000, also contains a description of the surficial geologic units and a general discussion of geologic hazards. This map shows the location of the Wasatch fault and a few landslide deposits, but otherwise does not delineate or discuss specific geologic hazards.

Geologic hazards have been mapped on a regional scale (1:24,000) by Davis County (Lowe, 1989). This unpublished series of maps delineate existing landslides, landslide hazard special study areas, debris flow hazard special study zones, and surface-fault rupture sensitive area zones.

GBES (2008) prepared an engineering geologic map of the site based on the descriptions and contacts of the geologic units from Solomon (2007). The specific engineering geologic units were modified from Keaton (1984) in order to provide more information on surficial materials and their origin, and thus be of more utility for engineering analysis.

Vertical aerial photographs of the site and vicinity were examined in order to assess the landslide and surface fault rupture hazard. Photographs were examined in stereo pairs at a scale of 1:20,000 and 1:40,000. In order to assess topographic features that existed prior to site disturbance, we examined photographs from 1958, 1965, and 1971.

3.2 Surface Investigations

As part of this study, field investigations were conducted in order to assess topographic features, surficial deposits, and bedrock exposures at the site. Observations were made of engineering geologic units and previously mapped contacts between geologic units were verified. In some cases, contacts were adjusted based on features observed in the field.

Landslides that had been identified from previous published reports and maps were field inspected. In addition, one area of potential landsliding was observed and mapped during the field investigations that had not been identified by previous reports and maps. At mapped landslides, observations were made of geologic materials, and evidence for factors that could affect slope stability and landslide susceptibility was recorded.

Locations where faults have been mapped were inspected to observe the presence or absence of surface features associated with recent fault movement.

Alluvial fans were inspected in order to verify the mapping and interpretation of deposits described by Lowe (1989), Solomon (2007), and GBES (2008). Rock fall hazards were assessed in the field by observing steep slopes where rocks are available and/or areas with evidence of previous rock falls.

3.3 Subsurface Investigations

In order to evaluate the subsurface geologic conditions and assess the fault rupture hazard, twelve trenches were excavated with track hoe. The trenches were oriented roughly in an east-west direction in order to intercept faults that are generally mapped trending in a north-south direction. Six trenches were located on the lower (western) bench of the property (Figure 3) and six trenches were located on the upper (eastern) bench. The trenches extended for distances between 114 and 408 feet and extended to a maximum depth of 18 feet below the ground surface. Figures 4 through 15 present detailed logs of the geologic materials exposed in the walls of the trenches at a scale of 1 inch to 5 feet.

In order to assess the debris-flow hazard twelve test pits were excavated with a rubber-tire back-hoe. The test pits were excavated near the eastern property boundary on the previously mapped alluvial fans (Figure 3) to a maximum depth of 8.3 feet below the ground surface. Figure 16 presents the logs of the geologic materials exposed in the test pits.

4.0 GEOLOGY

4.1 Tectonic Setting

The property is located along the Wasatch Front, in a broad north-trending valley at the foot of the western slope of the Wasatch Range. Within the vicinity of the project site, ongoing displacement along the Wasatch fault zone has resulted in the mountains rising steeply above the adjacent valley floors by as much as 5,000 feet. The Wasatch fault separates the Basin and Range Physiographic Province, a region characterized by a series of generally north-trending mountain ranges separated by similarly trending valleys from the Middle Rocky Mountains, a region characterized by Precambrian and Paleozoic sedimentary rocks deformed by compressional tectonics during the Sevier orogeny (roughly 75 to 40 million years ago).

The major topographic features along the Wasatch Front are a result of east-west crustal extension that began about 15 to 17 million years ago, and continues today (Hecker, 1993). The crustal extension has resulted in range-front normal faults that separate the mountains from valleys. Erosional processes have removed material from the mountains and partially filled the valleys with alluvial and lacustrine sediment. For the past 15 million years, this area has been a region of internal drainage, and lakes of varying sizes have occupied the basin throughout most of this time (Currey et al., 1984).

Much of the present topographic and geomorphic features are the result of faulting that has taken place on the Wasatch fault during the late-Pleistocene and Holocene. The Wasatch fault extends for about 213 miles along the base of the Wasatch Range between Malad City, Idaho and Fayette, Utah (Machette and others, 1992). Surface features associated with faulting have created a zone of deformation characterized by roughly north-trending, west-facing main fault scarps and smaller east-facing antithetic fault scarps. The Wasatch fault has been divided into 10

sections which rupture independently and are capable of generating large-magnitude surface-rupturing earthquakes (Machette and others, 1992).

The subject property lies within the Weber section of the Wasatch fault, which extends southward from North Ogden for about 35 miles to just south of Bountiful. Based on investigations conducted just south of Bair Canyon, McCalpin and others (1994) concluded that there have been five or six surface-faulting earthquakes in the past 13,000 years. The mean recurrence interval (long-term average time between events) is about 1,700 years (Black and others, 2003). Along the Weber section of Wasatch fault, net vertical ground displacement has been documented to vary between 3 and 12 feet per event (Black, et. al., 2003).

4.2 Lake Bonneville

The subject property lies with the area inundated by Lake Bonneville, a large late-Pleistocene pluvial lake that occupied the eastern Great Basin from about 28,000 to 10,000 years B.P. (Gilbert, 1890; Currey, 1990). The lake began to form about 28,000 years ago when colder and/or wetter climate conditions during the last ice age caused the lake to rise. This transgression continued until about 23,000 years ago when changing climate conditions caused the lake to undergo a slight regression. For nearly 3000 years, the level of the lake oscillated at an elevation of about 4,500 feet, forming the Stansbury shoreline, and then began to rise again.

By approximately 16,000 years before present, the lake reached the level of the topographic divide near Zenda, Idaho and began to overflow. This marks the beginning of the open-basin phase of the lake and the formation of the Bonneville shoreline (Burr and Currey, 1988). At the project site, the elevation of the Bonneville shoreline has been isostatically rebounded to an elevation of approximately 5,200 feet. At the time of the Bonneville highstand, the lake was over 1,000 feet deep and covered an area of approximately 20,000 mi^2 (Oviatt and Miller, 1997). Rivers emanating from the high mountains to the east provided large amounts of clastic material that was deposited in deltas along the mountain fronts. Spits, barriers bars, and beaches were supplied by alluvium and weathered bedrock in regions with little input from rivers. Muds and marls were deposited in more distal, offshore locations (Oviatt and Miller, 1997).

About 14,500 years ago, catastrophic failure of the alluvium at the threshold caused a massive flooding of lake waters into the Snake River drainage. This event, called the Bonneville flood, caused a drop in lake level of about 340 feet and is believed to have occurred in less than one year's time (O'Connor, 1993). The lake restabilized at a new bedrock threshold and the Provo shoreline began to form. At the project site, the Provo shoreline is near the eastern property boundary, at an elevation of approximately 4,860 feet.

The lake oscillated at or near the level of the Provo shoreline until a change in climate conditions permanently drove the lake below the threshold at Red Rock Pass. By about 11,500 years ago, the lake had reached levels comparable to modern Great Salt Lake (Godsey et al., 2005). The lake rose again briefly between about 11,000 and 10,000 years ago to form the Gilbert shoreline

(Oviatt et al., 2005) and then again declined. This marks the end of Lake Bonneville and the beginning of its successor, Great Salt Lake (Currey et al., 1984).

4.3 Engineering Geologic Units

Engineering geologic units were defined at the project site based on the lithology (as observed in both surface exposure and in trenches and test pits) and origin of the materials in general agreement with the procedure described by Keaton (1984). This system of classification is intended to provide information not contained in standard geologic maps. Specifically, additional detail is given to unconsolidated and surficial deposits. This system has the added benefit of providing information not only on materials, but on the processes responsible for transporting and depositing the materials.

We have delineated one bedrock engineering geologic unit, and five unconsolidated engineering geologic units. These units are shown on Figure 3, Engineering Geology Map and described below.

Bedrock Unit

MM Metamorphic This unit consists of undifferentiated metamorphic rocks assigned to the Farmington Canyon Complex.

Unconsolidated Units

Ac-b(f) Alluvial fan deposits This unit consists of poorly- to moderately-sorted, non-stratified, clay- to boulder-size sediment deposited primarily by debris flows at the mouths of active drainages and have distinct fan morphology.

A/Cs-k Alluvial-Colluvial mixed deposits This unit consists of poorly- to moderately-sorted, silt- to cobble-size sediment deposited by both alluvial and colluvial processes. This unit is present on the slopes below the Provo shoreline, and below the Wasatch fault. No distinct fan morphology exists.

Ls Lacustrine sand deposits This unit consists of fine- to coarse-grained sand with minor silt and gravel deposited in shallow water and beaches of Lake Bonneville. Exposed along scarp of Wasatch fault, or covered with thin layer of alluvial fan and/or alluvial/colluvial deposits.

Lg Lacustrine gravel deposits This unit consists of gravel with minor sand to cobble deposited in shorezones of Lake Bonneville. Located between the Provo and Bonneville shorelines.

Ss(r) Landslide deposits This unit consists of material displaced by rotational slides in unit Ls along scarp of Wasatch fault.

Ss-k Landslide deposits This unit consists of sandy to cobbly material displaced by slides originating in the lacustrine gravel deposit above the Provo shoreline.

5.0 GEOLOGIC HAZARDS

The geologic hazards that were evaluated as part of this investigation are surface fault rupture and associated tectonic deformation, landslides, debris flows, rock falls, and seismic seiche and storm surge. Each of these potential hazards are discussed below and, where applicable, shown on Figure 17, Geologic Hazards Map.

Geologic hazards have been evaluated on a regional scale (1:24,000) for portions of Davis County by Lowe (1989). This unpublished series of maps delineate existing landslides, landslide hazard special study areas, debris flow hazard special study zones, and surface-fault rupture sensitive area zones. Additional, site specific information on geologic hazards is provided by Solomon (2007), and on a regional scale by Solomon and others (2005).

Davis County does not currently have geologic hazards ordinances, and thus, there is no requirement to adhere to the recommended geologic hazard special study zones delineated by Lowe (1989). However, for the purpose of this investigation (unless where noted below), we consider the recommendations contained in Lowe (1989) to be scientifically valid and in accordance with generally accepted engineering geologic principles and practices in this area at this time. As such, we have generally accepted the delineation of the geologic hazards and recommendations for special study provided by Lowe (1989).

The Utah Geological Survey (UGS), although not a regulatory agency, does have the responsibility to provide for public safety in Utah by assisting cities, towns, and counties in dealing with geologic hazards. Among other tasks, they assist in planning, zoning, and permit approval issues by reviewing developer's geologic hazards reports for subdivisions or other proposed development. Our evaluations and recommendations with regard to the geologic hazards are in general agreement with guidelines published by the UGS.

5.1 Surface Fault Rupture Hazards

During a large earthquake, fault rupture at depth may propagate upward and displace the ground surface creating a fault scarp. Faults that show evidence of repeated surface displacement in the recent geologic past are considered to be active and represent a potential hazard to development. For the purposes of this study, a fault is considered active if there had been more than 4 inches of displacement during the Holocene (approximately the past 10,000 years).

Surface fault rupture hazards (including tectonic deformation of the ground surface) were investigated by first conducting a review of maps and reports of Quaternary faults in the area (Nelson and Personius, 1993; Hecker, 1993; Black, et. al., 2003), as well as inspection of the map by Lowe (1989) and Solomon (2007). The locations of known faults in the project area

were mapped on the engineering geologic map and inspected during the field investigations of this study. Additionally, stereo pairs of aerial photographs were examined for evidence of surface features associated with recent faulting.

Lowe (1989), Nelson and Personius (1993), and Solomon (2007) map a single north-trending, west-dipping trace of the Weber section of the Wasatch fault through, or adjacent to the project site (Figure 3). This fault trace is the represented by the prominent west-facing escarpment that runs the entire north-south length of the property. The scale of the existing geologic maps is not sufficient to determine the exact location of the fault with respect to the escarpment, but we have assumed that the fault is within the lower half of the escarpment. In addition, the previous geologic maps show one shorter, east-dipping, antithetic fault trace bisecting the lower portion of the property. The published reports have concluded that there has been movement on the Weber section of the Wasatch fault during the Holocene. This is based on the fact that the fault has displaced Provo-aged sediments, and on radiocarbon dating of faulted sediments in nearby trenches.

Christenson and others (2003) recommend special studies for all structures designed for human occupancy and all critical facilities where Holocene faults are present. The purpose of the studies are to determine the locations of active faults, zones of deformation, and amounts and direction of displacement in order to determine appropriate avoidance strategies such as building setbacks. Lowe (1989) has delineated a surface-fault rupture special study zone that encompasses the known mapped faults and an area that extends approximately 500 feet on either side of the faults. The width of the special study zone is based on the acknowledgement that the location of the fault is not known with certainty because of cover and incomplete mapping at sufficient scales. Based on the available mapping, and our experience with faults in similar settings, we infer that the fault is located within the lower half of the prominent escarpment on the property. The inferred locations of the faults and the surface-rupture special study zone boundary are shown on Figure 3.

In order to confirm the presence or absence of the antithetic fault that has been mapped on the lower (western) bench of the property, we excavated 6 trenches (Trenches A through F, Figure 3). The location, orientation, and length of these trenches were determined so as to intercept the fault, if it were present, in the area where future site development will take place.

The material exposed in the trenches consisted of three primary stratigraphic units (See Figure 4b for explanation of geologic units). Unit I consists of thin to thick beds of poorly sorted silty gravels with sand and cobbles and a few lenses of sandy gravel. We interpret this unit to represent Holocene-aged debris-flows and/or colluvial deposits. Unit II consists of thinly bedded poorly sorted, silty sand and gravel with a few lenses of clean sand and sandy gravel. We interpret this unit to represent late-Pleistocene lacustrine shorezone deposits that were reworked while Lake Bonneville was at or near the Provo stage. Unit III consists of thin to thick bedded, sub-horizontal planar beds of loose, rounded, sand and gravel. We interpret this unit to represent sediments deposited in beaches and near shore environments during the transgression of Lake Bonneville, and thus to be late-Pleistocene in age.

Detailed examination of the sediments in Trenches A through F revealed evidence of no faulting (such as back tilted or displaced beds) in any of the stratigraphic units (Figures 4 - 9). Specifically, we observed that while some individual beds naturally thinned and were thus discontinuous, there was evidence that the sediments were not offset as a result of faulting. Other than discontinuities which were either sedimentary or stratigraphic in nature, the sediments exposed in the trenches were continuous. Because the sediments of Unit II and Unit III were deposited and/or reworked by shorezone processes associated with Lake Bonneville, we conclude that there has not been surface faulting on the lower (western) bench of the property at the location of our trenches within the past 11,500 years (the last time Lake Bonneville was at this elevation).

The results of our detailed subsurface investigations are in conflict with previous published reports (Nelson and Personius, 1993; Solomon, 2007) which have mapped an antithetic fault in this area. First, we note that neither Nelson and Personius (1993) or Solomon (2007) conducted subsurface investigations. They identified the fault from aerial photographic examination and limited field inspections of surface features. We are puzzled by the fact that they mapped this suspected fault as an antithetic fault, which would, if it existed, be manifested by a fault scarp facing east (towards the main trace of the Wasatch fault) and possibly a down-dropped surface between the antithetic fault and the main fault. No such features exist at this site. In fact, the ground slopes towards the west across the entire lower bench. In addition, we observed no indication of an antithetic fault in our examination of aerial photographs. We are at odds to explain the basis for Nelson and Personius having mapped this fault. However, the property owner reports that a house and septic drain field did exist near the southwest corner of the property. It is possible that prior to regrading in this area, a depression in the ground existed where the foundation and drain field were located. Perhaps this depression was interpreted by Nelson and Personius (1993) as a fault-related feature. Regardless of the basis, we are confident, based on our detailed subsurface investigation, that the previously mapped antithetic fault does not exist on the lower bench of the property where we trenched.

In order to confirm the presence or absence of faults on the upper (eastern) bench of the property, we excavated 6 trenches (Trenches G through L, Figure 3). The location, orientation, and length of these trenches were determined so as to intercept any faults that might exist which would trend approximately parallel to the main trace of the Wasatch fault. The western end of the trenches was near the upper part of the main fault scarp; the eastern end of the trenches corresponds to the eastern limit of the 500-foot fault special study zone identified by Lowe (1989). Trenches H, I, and J are discontinuous; no subsurface investigations were conducted within the aqueduct easement.

The material exposed in the trenches consisted of the same three primary stratigraphic units as encountered in the trenches on the lower bench (Figures 10 – 15). In addition, we identified an older unit (Unit IV) in Trench G that was distinctly bimodal, part consisting of angular boulders up to 6-feet in diameter and part consisting of well sorted sand and gravel in the spaces between the boulders. We interpret this unit to represent a bouldery landslide or debris flow from Bair

Canyon that contains beach sand and gravel that washed in between the boulders as Lake Bonneville rose to this elevation. Thus, this unit would be late-Pleistocene in age.

Detailed examination of the sediments in Trenches G through L revealed evidence of no faulting (such as back tilted or displaced beds) in any of the stratigraphic units. Specifically, we observed that while some individual beds naturally thinned and were thus discontinuous, there was evidence that the sediments were not offset as a result of faulting. Other than discontinuities which were either sedimentary or stratigraphic in nature, the sediments exposed in the trenches were continuous. Because the sediments of Unit II, III, and IV were deposited and/or reworked by shorezone processes associated with Lake Bonneville, we conclude that there has not been surface faulting on the upper (eastern) bench of the property at the location of our trenches within the past 11,500 years (the last time Lake Bonneville was at this elevation). Our conclusions are consistent with previous publications which indicate that there are no faults in this area.

There was no trenching as part of this study to identify the precise location of the main trace of the Wasatch fault at this site. However, based on the available mapping, published reports, and our experience with faults in similar settings, we infer that the fault is located within the lower half of the prominent escarpment on the property. Because the scarp most likely represents multiple surface fault rupture events, and because of the uncertainty of the exact location of the recent ruptures on the scarp, we recommend that no critical facilities or structures designed for human occupancy be built on the scarp or within setback zones on either side of the scarp discussed below and shown on Figure 17a.

Fault setback zones are delineated based on the current understanding that surface fault rupture events follow patterns of the recent past. We have calculated the fault setback zone according to the procedures described by Batatian (2002) and Christenson and others (2003). The fault setback for the downthrown block (west side of the Wasatch fault at this site) is calculated as:

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

where:

S = Setback distance from active faults,

U = Criticality factor (1.5 for Residential, IBC occupancy class R-3)

D = Expected fault displacement per event (assumed to be 12.0 feet)

F = maximum depth of footing or subgrade portion of building (8.0 feet)

θ = Dip of fault (assumed 60°)

The fault setback for the upthrown block (east side of the Wasatch fault at this site) is calculated as:

$$S = U(2D)$$

Based on the above formulas, we recommend a setback distance of 45 feet on the west side of the base of the scarp and 40 feet on the east side of the top of the scarp. These setback distances are considered conservative because we have assumed that the fault trace is located at the base and top of the scarp.

5.2 Tectonic Deformation Hazards

During a surface rupturing earthquake, deformation of the ground will occur. This deformation is generally manifested at two scales, local and regional. Local deformation will be associated with displacement of the ground along scarps in a zone where the fault reaches the ground surface. In a normal fault, such as the Wasatch fault, displacement typically produces a steeply dipping scarp with the downthrown block on the west side of the scarp. In addition, it is common to have a zone of deformation on the downthrown block where additional normal scarps are present as well as antithetic scarps, where displacement is down to the east. In complex zones of deformation, the displacement of the ground surface can create narrow valleys (called grabens) which are bounded on the east by a normal fault and on the west by an antithetic fault.

We conducted extensive subsurface investigations in the area west of the main trace of the Wasatch fault and found evidence for no antithetic faults, or normal faults that diverge from the main fault. We observed that there were no displaced or deformed sediments, and thus conclude that tectonic deformation has not occurred within the past 11,500 in the area of our trenches.

Tectonic deformation can also occur on a regional scale and is generally manifested by a gentle tilting of the crust of the downthrown block towards the fault. This deformation can extend for tens of miles, but is generally small in magnitude compared with the ground displacement associated with surface rupture along the fault itself. The most likely hazards associated with regional tectonic deformation would be backtilting of sewer lines or flooding from the Great Salt Lake; the latter is discussed below in the section on seismic seiche and storm surge.

5.3 Landslide Hazards

Landslide hazards were investigated by evaluating information from previous published reports, aerial photographs, topographic maps, and site inspections. Existing landslides were initially mapped from information contained in Solomon (2007) and Lowe (1989); these landslides were inspected in the field in order to confirm their existence. Landslides were delineated based on the presence of features associated with slope movement, *inter alia*, head scarps, hummocky topography, internal scarps, and bulging toes. Observations of these features were made from examination of stereo pairs of vertical aerial photographs and during the field investigations part of this study.

Two small landslides exist in the fault scarp east of Trenches A and B (See Figure 3). These are rotational slides involving the sandy lacustrine sediments. At the main scarp of the southern slide, we observed a broken cast iron pipe approximately 10 inches in diameter. We do not know if this pipe carried water for irrigation, or was part of a drainage system, but we suspect that leakage from this pipe may have saturated the sediments and contributed to the movement. Discussions with the property owner confirmed that this slide occurred as a result of over watering on the upper bench. We understand that GSH Geotechnical Consultants, Inc. has conducted stability analysis of this landslide. The northern of these slides is located on the escarpment west of a gentle topographic swale on the upper bench. We suspect that surface-water runoff has been concentrated on the upper bench and subsequently contributed to the slope instability in this area.

An area of possible landsliding was observed adjacent to the southern landslide discussed above (see Figure 3). We interpret this as a possible slide based on a near-vertical main scarp, hummocky topography, and an apparent bulging of material at the toe of the slope. However, one, or more, traces of the Wasatch fault cut across the slope in this area and could be responsible for the hummocky slope. We did observe a broken pipe in the main scarp similar to the one in the landslide to the north, and believe that leakage may have contributed to movement in this are as well. Additional investigation, including analysis of subsurface materials, may determine that the topographic features in this area are not a result of landslide process. However, based on the evaluation conducted to date, we consider this area as a landslide.

A slide has been mapped by Solomon (2007) in the southeast corner of the property (Figure 3). This slide was mapped solely on the basis of aerial photographic interpretation [Solomon, 2008, personal communications], presumably on the basis of a topographic bulge on the otherwise uniform slope. Our examination of this area indicates that the bulge is not displaced sandy and cobbly material associated with the shorezone sediments upslope, but is an outcrop of metamorphic rock. Because of its greater resistance to erosion, this outcrop stands out in relief. We observed two other similar outcrops of metamorphic rock at similar positions on the slope north of this area, indicating that this rock is likely in place bedrock and not displaced. Furthermore, we observed that there is no topographic swale upslope of this area that would be indicative of a landslide scarp. Based on detailed examination of aerial photographs and site inspections, we conclude that this is not a landslide as mapped by Solomon (2007).

Solomon (2007) also maps a slide upslope of the north east corner of the subject property approximately 300-500 feet east of the property boundary (east of Alluvial Fan A, but not in the area covered by Figure 3). This slide was mapped solely on the basis of aerial photographic interpretation [Solomon, 2008, personal communications], presumably on the basis of a topographic swale on the otherwise uniform slope which was interpreted to be a landslide scarp. However, based on our examination of the slope in question, we observed that the swale is the result of differential erosion where two ridges of more resistant bedrock stand out in relief. We observed no evidence for slope movement in aerial photographs or from our detailed site inspection. Therefore, we conclude that is not a landslide as mapped by Solomon (2007).

Other areas of possible old landslides exist on the steep scarp of the Wasatch fault, both north and south of the existing landslides discussed above. We observed several areas where the upper bench appears to be migrating eastward due, in part, to landslide processes. We suspect that concentration of surface and/or ground water on the upper bench contributes to sliding. We understand that based on the results of their slope stability analysis, GSH Geotechnical Consultants, Inc. will provide recommendations for setback distances from the top of the scarp.

Existing landslide hazard areas on the property are shown Figure 17a. We recommend that no critical facilities or structures designed for human occupancy be built in these areas.

5.4 Debris-Flow Hazards

Debris flows are a fast-moving, flow-type landslide composed of a slurry of rock, mud, organic debris, and water that move down drainage basin channels onto alluvial fans (Giraud, 2005). Debris flows generally initiate on steep slopes or in channels as a result of a rapid influx of water, either from an intense rainfall, or rapid snowmelt. As debris flow travel down channels, they often entrain material, and thus increase in volume. When debris flows reach an alluvial fan the gradient and channel confinement both decrease and flows begin to spread laterally and deposit their sediment (Lips, 1993; Giraud, 2005). Thus, alluvial fans are susceptible to debris flow hazards.

Lowe (1989), Nelson and Personius (1993), and Solomon (2007) all map alluvial fans on the subject property; however, the maps are not consistent with each other. Because of the scale, and based on discussion with the author (Solomon, 2008, personal communication), we believe that the mapping by Solomon (2007) is the most accurate representation of the alluvial fans at this site. However, Solomon (2007) maps the entire upper bench of the subject property (between the scarp of the Wasatch fault and the Provo shoreline) as alluvial fans formed predominately as the result of debris flow deposition. We believe that only portions of this upper bench are active alluvial fans, and therefore subject to debris-flow hazards. Based on the lack of fan morphology on most of the surface of the upper bench, we believe that some of the material that overlies the fine-grained lake sediments is derived from colluvial slope processes and/or small debris floods. Therefore, we have mapped alluvial fans only at the mouths of the drainages and/or where the surface has distinct fan morphology. On this basis, we have identified seven alluvial fans on, and east of the subject property (Alluvial Fans A through G, Figure 3).

The large alluvial fan mapped by Solomon (2007) on the southern end of the upper bench lacks fan morphology and is covered with lacustrine shorezone sediments that were reworked while Lake Bonneville was at the Provo shoreline (Unit IIa, Figure 10, Log of Trench G). Below the lake sediments are large boulders that were likely deposited by debris flows from Bair Canyon prior to the rise of Lake Bonneville. However, incision of the creek in the Holocene has isolated this old alluvial fan surface and it is no longer connected to Bair Canyon.

Although there are distinct alluvial fans on the property, not all fans are comprised of, or subject to debris flows. We consider a debris-flow hazard to exist on an alluvial fan if there is evidence

of one or more debris flows during the Holocene (past 10,000 years). In order to determine which fans are susceptible to inundation by future debris flows, we conducted detailed surface and subsurface investigations on Alluvial Fans B through G. Our evaluation of the debris-flow hazard is based on observations of debris-flow features at the surface and/or evidence of Holocene debris flows exposed in the subsurface sediments.

Surface investigations consisted of identifying source area size and evidence of debris-flow deposits such as levees or boulder snouts. In order to identify past debris-flow deposits, we excavated 12 test pits on Alluvial Fans B through G (Figure 3). The test pits were extended to the depths necessary to expose sediments associated with Lake Bonneville and thus be older than Holocene sediments. In each test pit, observations were made of debris-flow deposits (where present) and the maximum size of boulders in the deposits was recorded.

The results of the investigation reveal that only Alluvial Fans B and E have evidence for debris flows that have traveled to the property during the Holocene. Both of these fans have boulders at the surface from recent debris flows and evidence for at least one debris flow exposed in the subsurface. Sediments in Test Pits 1 and 2 on Fan B, and Test Pits 8 and 9 on Fan E (Figure 3) reveal at least one debris flow deposit in Unit I (Figure 16), which was deposited after the Lake Bonneville sediments (Unit II, Figure 16).

There was evidence for no Holocene debris flows at Alluvial Fans C, D, F, and G. Specifically, we noted no surface features indicative of debris flows, no boulders, and no sediments in the subsurface that resulted from deposition of debris flows. In addition, all of these fans have weak fan morphology and are small in size, both a result of the small drainage areas above the fans, none of which extend above the Bonneville shoreline. On the basis of the evidence for lack of Holocene debris flows, we believe that the debris-flow hazard for these fans is very low. However, it should be recognized that these fans could be inundated by surface water runoff, and sediment contained in the flow, as a result of snow melt and/or precipitation events. We understand that flood hazards for the entire property have been evaluated by Hydrologic Design.

Because the ground surface at Alluvial Fan A has been disturbed by prior earthwork activities, our evaluation of this fan could not be based on surface observations and subsurface investigations discussed above. However, given the size of the drainage basin(s) that contribute to this fan, and the similarities to the drainage above Alluvial Fan B, we assume that there is a debris-flow hazard.

Debris flows can be effectively mitigated with flow control structures on alluvial fans. These structures generally fall into two categories; open structures and closed structures (Hungr et. al., 1987; VanDine, 1996). Open structures, such as deflection berms, baffles, and lined channels are designed to constrain flow. Closed structures, such as debris racks and debris basins are designed to contain debris. In Utah, sediment storage basins (debris basins) are the most common type of control structure to reduce debris-flow risks (Giraud, 2005). The greatest advantage of debris basins is that they benefit the entire alluvial fan.

In order to assist in selecting and designing mitigation structures, we have evaluated the volume of sediment and the thickness and size of boulders that could be expected in future flows. We estimate the sediment volume of future debris flows at the mouths of the canyons based on evaluating the channel length and sediment storage. The rationale for this approach is discussed in detail by Giraud (2005) and is based on numerous observations that the majority of material deposited by debris flows on alluvial fans (80-90%) is derived from the channel that the debris flow travels down (Croft, 1967; Santi, 1988; Keaton and Lowe, 1988). Most estimates of sediment entrainment, or bulking, are based on a unit volume of erodible channel material, usually expressed in cubic yards per linear foot of channel (Hungr et. al., 1984; VanDine, 1985; Williams and Lowe, 1990). Estimated bulking rates for recent debris flows in Utah vary between 1.5 yd^3/ft and 12 yd^3/ft (Giraud, 2005). Channel length and channel sediment storage have been found to be the most important factors in estimating volumes of future debris flows (Hungr et. al., 1984; VanDine, 1985, 1986; Williams and Lowe, 1990).

Sediment bulking factors for this evaluation were derived by comparing the channels above the alluvial fans with the channel characteristics and sediment bulking factors reported by Hungr and others (1984). The channels above Alluvial Fans A, B, and E generally have nonerodible side slopes and channel beds of thin debris or loose soil over bedrock. These channels correspond to Channel Type B of Hunger and others (1984) which have sediment bulking factors of 2-4 yd^3/ft. Because the gradients of the channels at this site are at or greater than the gradient range of Channel Type B, we have selected a sediment bulking factor of 2yd^3/ft. Table 1 provides a summary of the channel and debris-flow characteristics for Alluvial Fans A, B, and E.

Channel lengths and gradients were measured from U.S.G.S. 1:24,000 topographic maps (Figure 17b). Because of the regrading at the head of Fan A, we are uncertain which of the channels currently contributes sediment to the site. For the purposes of evaluating the sediment volume, we have used the longest channel.

Estimated flow thickness and boulder size for future flows were based on observations of these characteristics of flows on the surfaces of the fans and/or as exposed in the test pits. Because of the similarities in the channel length, gradient, and geologic materials for the drainages above Fan A and Fan B, we assume that the flow thickness and maximum boulders observed on Fan B are representative of those that would be expected for a debris flow on Fan A.

Because of the proximity of the proposed development to the mouths of the canyons from which the debris flows emerge, and because of the uncertainty in the paths debris flow take on alluvial fans, we have not delineated areas of the fans with higher or lower risk from debris flows. Rather, we delineate the debris flow hazard at the mouth of the canyon, based on the estimated debris-flow characteristics listed in Table 1. Alluvial Fans A, B, and E are shown on Figure 17a; the channels that feed these fans are shown on Figure 17b.

Table 1. Channel and Alluvial Fan Characteristics

Alluvial Fan	Channel Length[1] (ft)	Channel Gradient (deg)	Sediment Bulking Factors[2] (yd^3/ft)	Estimated Debris Flow Volume (yd^3)	Estimated Debris-Flow Thickness[3] (ft)	Estimated Maximum Boulder Dia.[4] (ft)
A	2,800	21	2.0	5,600	3.0	4.0
B	3,000	22	2.0	6,000	3.0	4.0
E	700	16	2.0	1,400	2.0	3.0

[1] Channel lengths are measured from the property boundary to the head of the channel identifiable on the U.S.G.S. 1:24,000 topographic map.
[2] Sediment Bulking Factors correspond to Channel Type B from Hungr and others (1984).
[3] From observations of past debris flows exposed in test pits.
[4] From observations of boulders on the alluvial fan surfaces

5.5 Rock Fall Hazards

Rock fall involves the rapid movement of intact rocks by falling, rolling, or bouncing from steep to vertical slopes. Rock fall hazards have not been investigated or delineated in previous published reports. For this hazard evaluation, the identification of rock-fall hazard areas is based on the presence of steep to vertical slopes, availability of rocks, and evidence of previous rock falls. Based on these criteria, we have delineated two areas on the steep slopes comprising the walls of the gravel pits in the northeast portion of the property where rock fall hazards exist (Figure 17a).

Mitigative measures designed to reduce the risk from rock falls include engineering structures that either contain or deflect falling rocks. Effective mitigation also includes regrading the slope to gradients that are lower than those needed for the initiation of rock falls. We have assumed that regrading in the area of the gravel pits during the development of the property will reduce slopes and thus eliminate the rock fall hazard. However, if these steep slopes remain, and other mitigative measures are employed, additional geologic analysis would be required to determine characteristics of the falling rocks (travel distance, velocity, bounce height, and impact forces) necessary for the design of the engineered structures.

5.6 Seismic Seiche and Storm Surge Hazards

Seismic seiche and storm surge hazards were evaluated based on a review of published reports of characteristics of these flood events (Wang, 1978; Lowe, 1993; Atwood and Mabey, 1995; Chang and Smith, 1998; Atwood, 2002; and Solomon et. al., 2005).

Flooding hazards associated with the Great Salt Lake (GSL) can result from climate-driven lake-level rise, seiche, storm surge, and/or tectonic related tilting. In assessing each of these potential hazards, we consider the historic average elevation of the GSL (4,202 feet above sea level [asl]), with the lowest elevation on the subject property (4,690 feet asl).

Climate-driven lake level rise operates on several time scales. During the Holocene, the lake has risen to an elevation of 4,215 feet asl every few hundred years. Longer and more intense wet cycles have caused the lake to rise to an elevation of 4,217 feet, but these cycles probably occur at a frequency of several hundred years. Only once in the past 10,000 years has the lake risen to elevation of 4,220 feet asl (Atwood, 2002). In order for the lake to rise to the elevation of the subject property, it would require a climate change that operates on cycles of tens of thousands of years. For engineering geologic planning purposes, we do not consider this to be a hazard for the subject property.

Seiche is a free or standing-wave oscillation of the water level in an enclosed basin. Wind-driven lake set up, landslides, earthquake induced ground shaking, or surface faulting on the lake floor may create seiche (Solomon et al., 2005). The maximum height of a wind-driven seiche recorded for the GSL is approximately 2 feet (Wang, 1978). Lowe (1993) suggests that an earthquake-induced seiche 12 feet in height resulted from a M 6 earthquake in Hansel Valley. There are no estimates of the height of a seiche that could result from a M 7 earthquake on the Wasatch fault, or from landsliding, but they are likely to be on the same order of magnitude as the one generated by the Hansel Valley earthquake. For engineering geologic planning purposes, we do not consider this to be a hazard for the subject property.

Storm surge and wave runup (together referred to as shoreline superelevation) can cause waves to reach elevations in shorezones that are above the stillwater elevation of the lake. Measurements of debris deposited in the shorezone on Antelope Island during the high lake levels of the 1980's indicated that the shoreline superelevation reached heights of 11 feet (Atwood, 2002). For engineering geologic planning purposes, we do not consider this to be a hazard for the subject property.

Earthquake-induced tectonic deformation can cause tilting of the ground surface on the downthrown block along the Wasatch fault zone. This backtilting would cause a subsidence of the ground surface and would result in an eastward migration of the shore of the Great Sale Lake and thus cause flooding in low lying areas adjacent to the eastern shore of the lake. Based on a worst case scenario (a M 7.5 earthquake on the Wasatch fault, and the Great Salt Lake at an elevation of its historic highstand of 4,212 feet asl), Chang and Smith (1998) estimate a 3.5 mile southeast displacement of the shoreline. However, this would only change the location of the shoreline, the elevation of the lake would still be at 4,212 feet asl. For engineering geologic planning purposes, we do not consider this to be a hazard for the subject property. A secondary hazard would be the adverse effects of backtilting on gravity-driven water and sewer lines near the fault zone.

6.0 DISCUSSION AND RECOMMENDATIONS

Portions of the subject property either are, or may be susceptible to geologic hazards. For each of the geologic hazard included in this evaluation we provide the following discussion and recommendations.

Surface Fault Rupture Hazards

Based on the results of our detailed subsurface investigations we conclude that there are no active faults on either the upper or lower benches of the property. The main trace of the Wasatch fault forms the prominent scarp separating the upper and lower benches. The exact location of the Wasatch fault was not determined from trenching conducted as part of this hazard evaluation. However, based on mapping by others, and our experience in similar settings, we infer that the fault is located within the lower half of the scarp. We have delineated a setback zone on either side of the scarp (Figure 17a) and recommend that no critical facilities or structures designed for human occupancy are built within this zone. Because of the uncertainty of the exact location of the Wasatch fault, our setback zone has been established from the toe and crest of the scarp.

Tectonic Deformation Hazards

We observed that there were no antithetic faults or zones of deformation on the lower bench of the property in an area where tectonic deformation associated with active faulting could occur. Thus, we conclude that other than the surface rupture hazard discussed above, there is no evidence for a local tectonic deformation hazards.

Movement on the Wasatch fault could result in a regional-scale deformation of the ground surface causing the ground west of the fault to become tilted slightly to the west. If this happens, gradients of sewer and gravity-driven water lines could be reduced.

Landslide Hazards

We identified three areas on the property where landslides have (or likely) occurred. Each of these is in the unconsolidated sediments exposed in the scarp of the Wasatch fault. Water saturation likely contributed to the movement of these slides. We recommend that no critical facilities or structures designed for human occupancy be built in these landslide hazard areas (shown on Figure 17a). In addition, because of evidence of possible older landslides and the potential for surface and/or ground water to be concentrated in the sediments on the upper bench near the scarp of the Wasatch fault, we recommend that no structures designed for human occupancy are built near the scarp (toe or crest). We understand that setback recommendations for these areas have been provided by others.

Debris-Flow Hazards

We consider a debris-flow hazard to exist on an alluvial fan if there has been at least one debris flow during the Holocene. Based on the results of our detailed investigations of the surface and subsurface of the alluvial fans on the property, we conclude that there is a debris-flow hazard on only three of the fans (Alluvial Fan A, B, and E, Figure 17a). The remaining fans contained evidence that there have not been debris flows during the Holocene. We have estimated the volume of sediment, flow thickness, and maximum boulder that could be expected from future flows in the channels above Alluvial Fans A, B, and E (Figure 17b). These characteristics (Table 1) can be used to assist in selecting and designing risk-reducing mitigation measures on, or near, the alluvial fans.

Rock Fall Hazards

We have identified two areas in the northeast portion of the property where rock fall hazards exist (Figure 17a). Our delineation of these areas is based primarily on the basis of the steep slopes. However, we understand that these areas will be regarded to lower gradients during site development and thus the rock fall hazard will be eliminated. If there will be no regarding of the steep slopes in this area, we recommend that prior to site development additional studies are conducted for the rock fall hazard runout zones. These studies should determine characteristics of rock fall behavior such as travel distance, velocity, bounce height, and impact forces for various sized rocks such that appropriate risk-reducing mitigative measures can be employed.

Seismic Seiche and Storm Surge Hazards

Because of the elevation difference between the Great Salt Lake (GSL) and the lowest elevation on the subject property (approximately 490 feet), we do not consider there to be a hazard resulting from climate-driven lake-level rise, seiche, storm surge, and/or tectonic related tilting.

7.0 LIMITATIONS

The analysis, conclusions, and recommendations submitted in this report are based upon the data obtained from site-specific observations and compilation of known geologic information. Specifically, our conclusions and recommendations are governed by the physical properties of the surface and subsurface materials encountered during the field inspections. As additional information on site geologic conditions becomes available, GBES must be informed so that our conclusions and recommendations can be reviewed and amended, if necessary.

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

Great Basin Earth Science, Inc.

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.

Sincerely,
Great Basin Earth Science, Inc.



Elliott W. Lips, P.G.
Senior Engineering Geologist
and President
Utah Professional Geologist # 5529142-2250

Copies: Addressee (4)

Attachments:
Figure 1, Vicinity Map
Figure 2, Site Plan
Figure 3, Engineering Geology Map
Figure 4a-b, Log of Trench A
Figure 5, Log of Trench B
Figure 6a-b, Log of Trench C
Figure 7, Log of Trench D
Figure 8a-b, Log of Trench E
Figure 9a-b, Log of Trench F
Figure 10, Log of Trench G
Figure 11a-c, Log of Trench H
Figure 12a-c, Log of Trench I
Figure 13a-c, Log of Trench J
Figure 14a-c, Log of Trench K
Figure 15, Log of Trench L
Figure 16, Log of Test Pits
Figure 17a-b, Geologic Hazards Map

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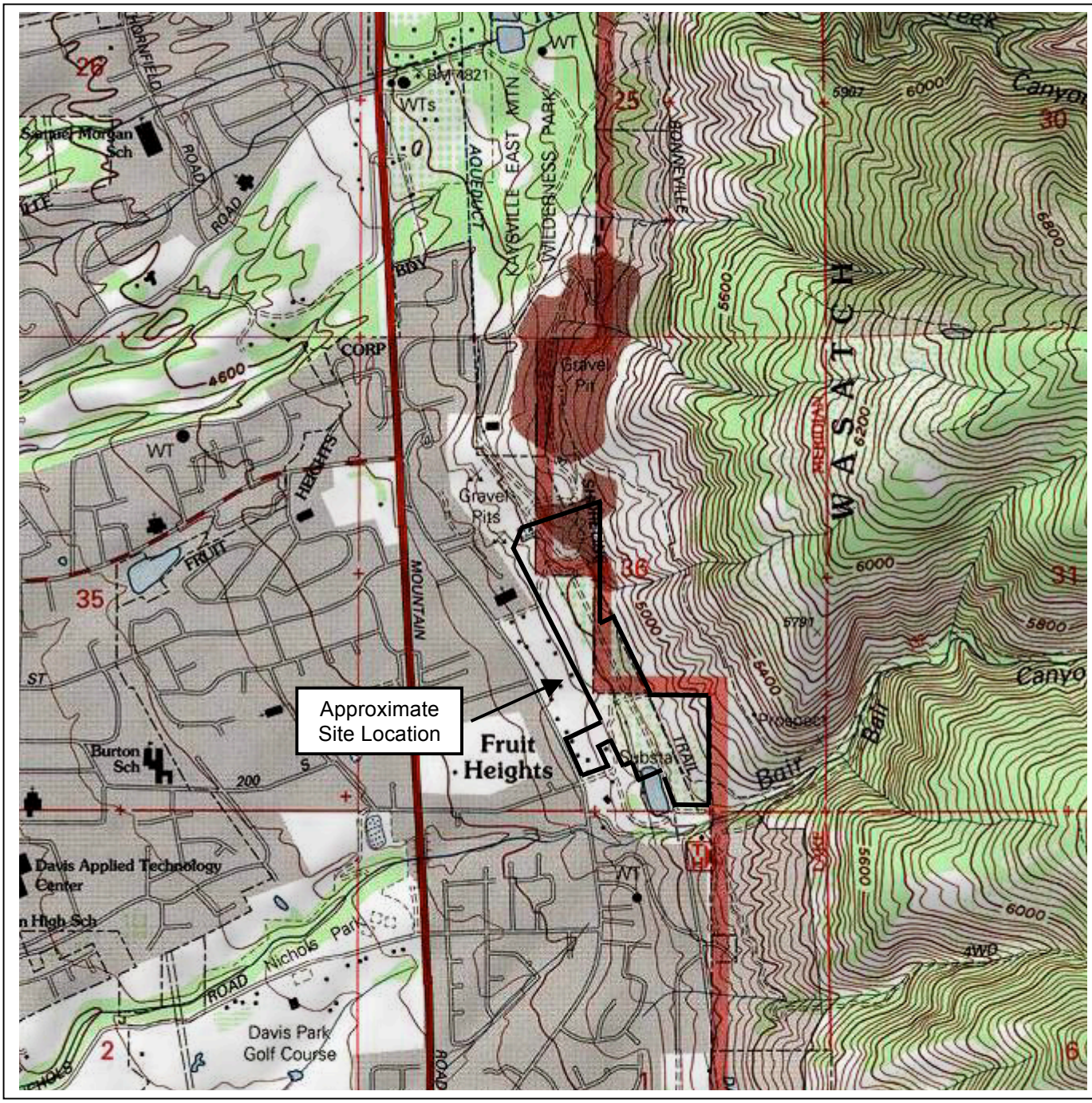
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N

Scale: 1 inch = 2000 feet

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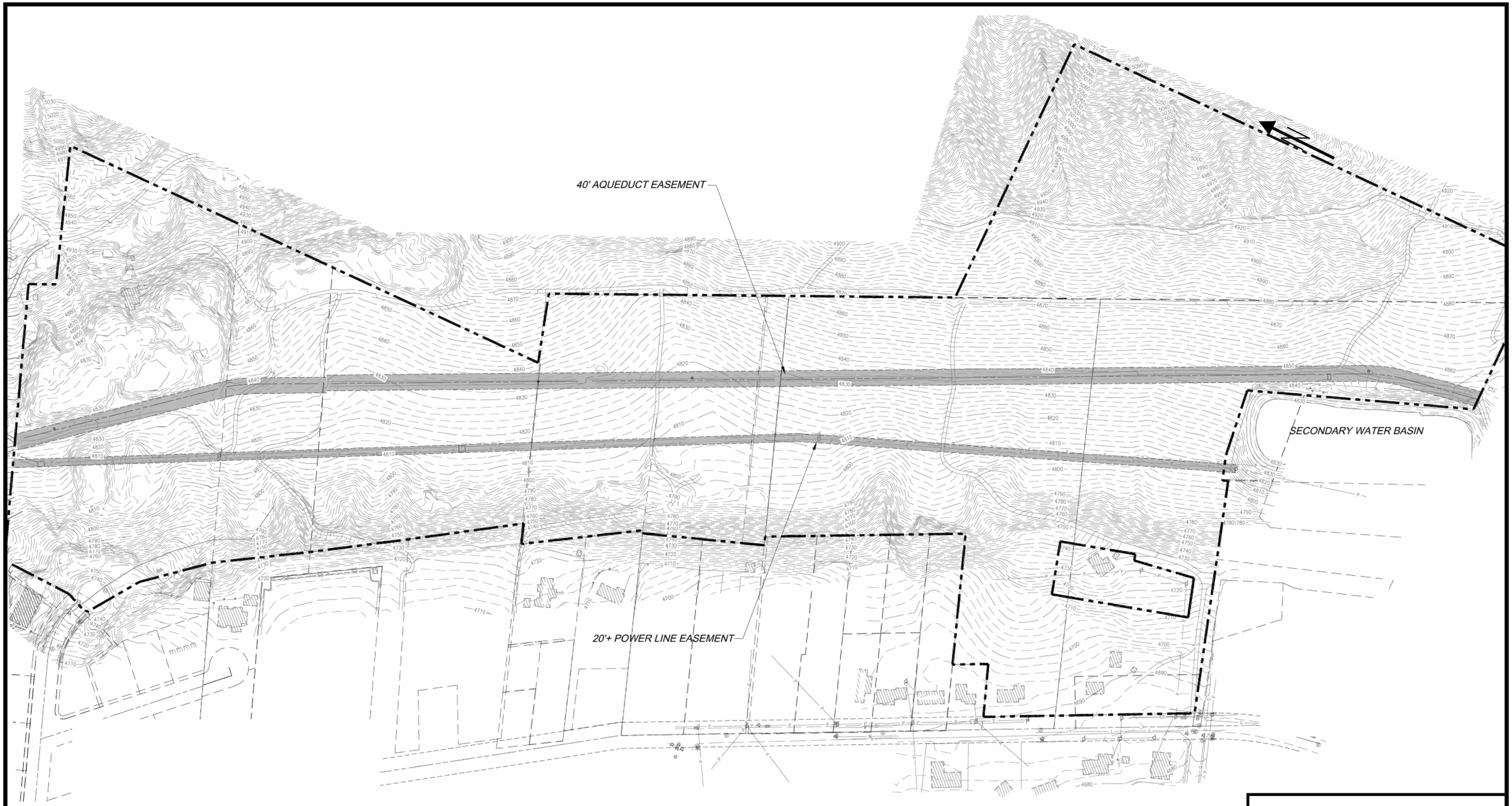
FIGURE 1 VICINITY MAP

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE

Fruit Heights, Utah

July 21, 2008



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Scale: 1 inch = 250 feet

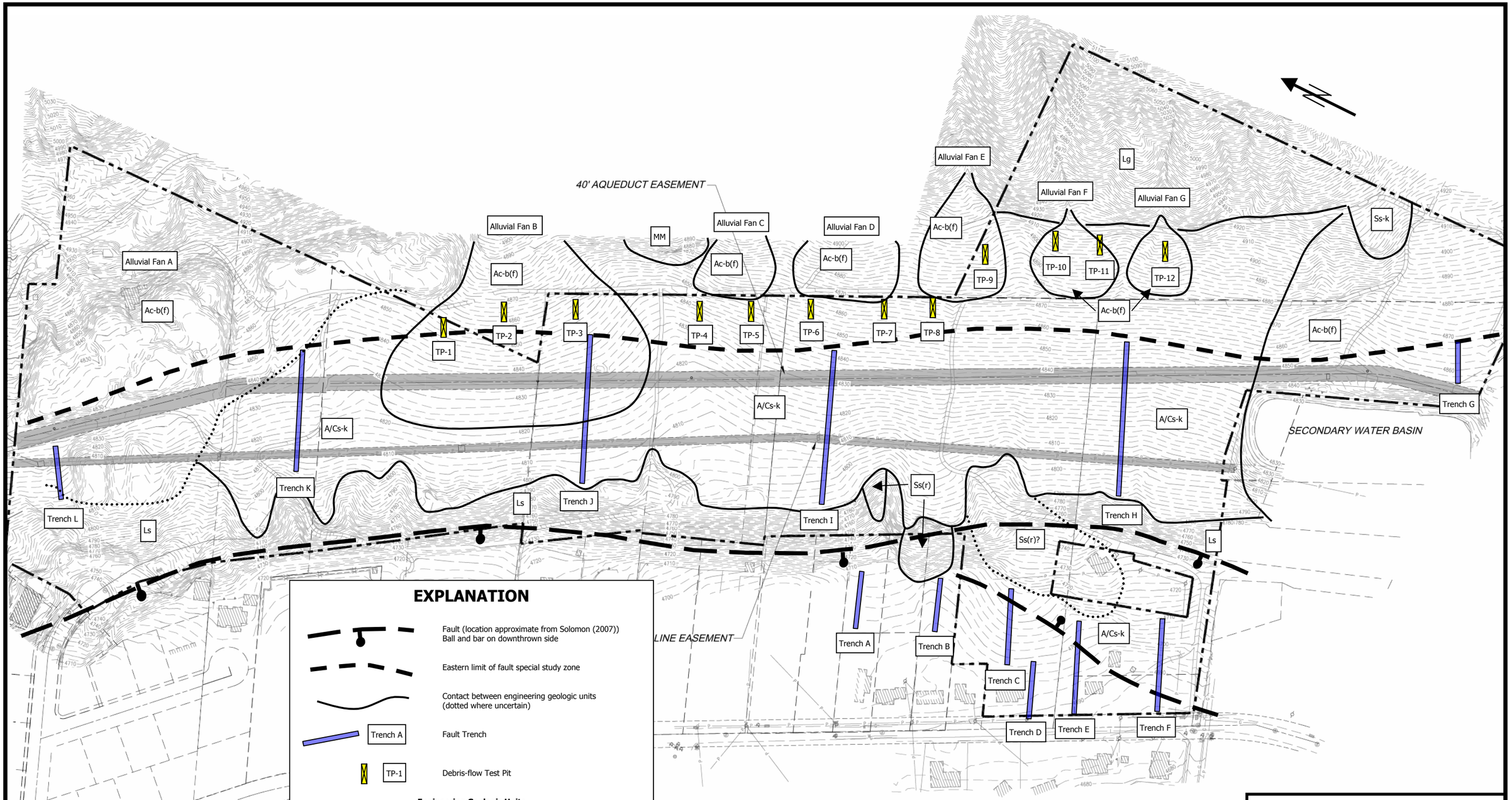
125 0 125 250

FIGURE 2 SITE MAP

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008



EXPLANATION

Fault (location approximate from Solomon (2007)) Ball and bar on downthrown side

Eastern limit of fault special study zone

Contact between engineering geologic units (dotted where uncertain)

Trench A — Fault Trench

TP-1 — Debris-flow Test Pit

Engineering Geologic Units
(see text for complete descriptions)

MM	Metamorphic rock, undifferentiated
Ac-b(f)	Alluvial fan deposits, clay to boulder size
A/Cs-k	Alluvial-Colluvial mixed deposits, sand to cobble
Ls	Lacustrine sand deposits
Lg	Lacustrine gravel deposits
Ss(r)	Landslide deposits, sandy material, rotational
Ss-k	Landslide deposits, sandy to cobbly material

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Scale: 1 inch = 250 feet

125 0 125 250

FIGURE 3 ENGINEERING GEOLOGY MAP

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008

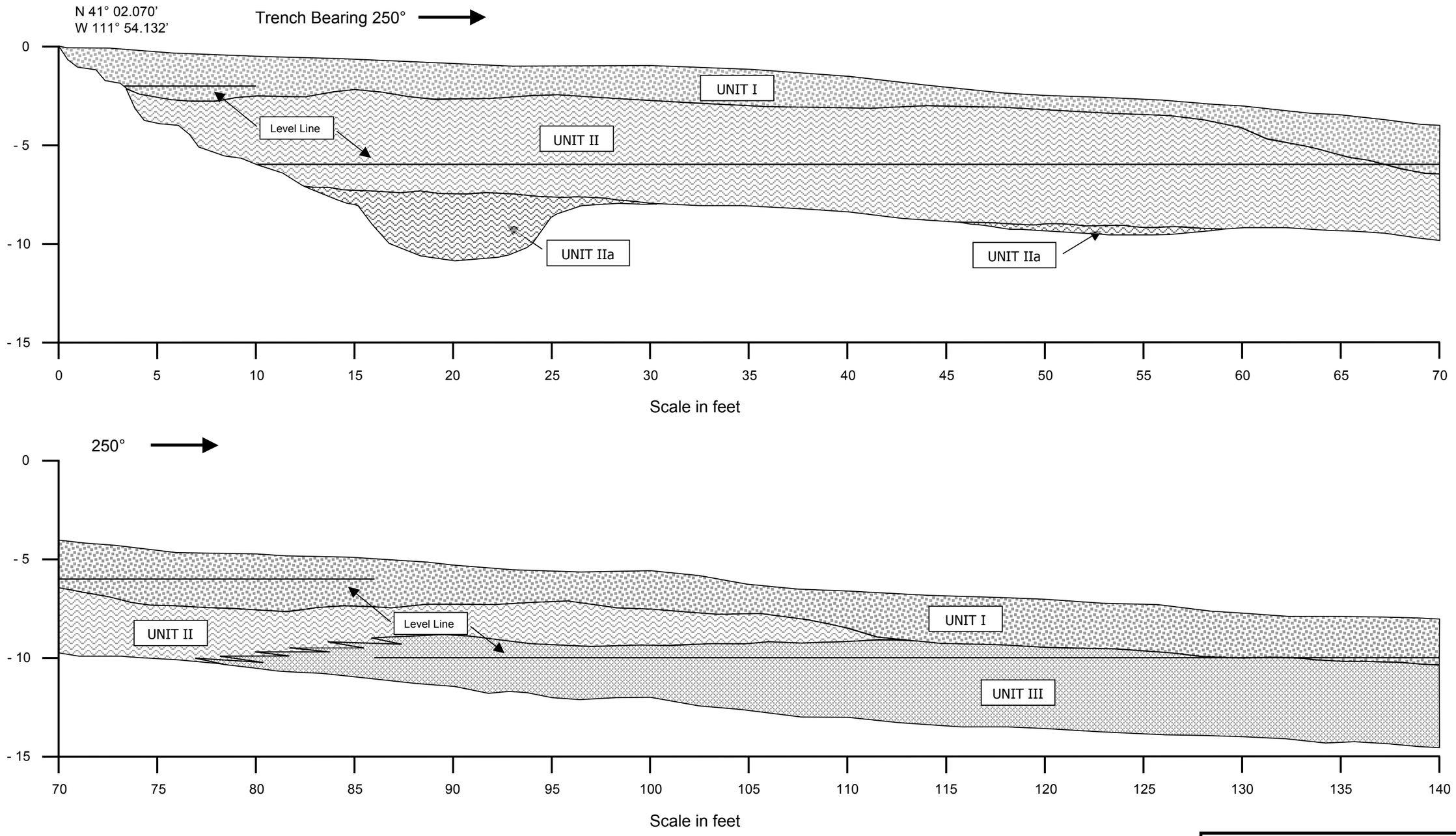


FIGURE 4a LOG OF TRENCH A

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

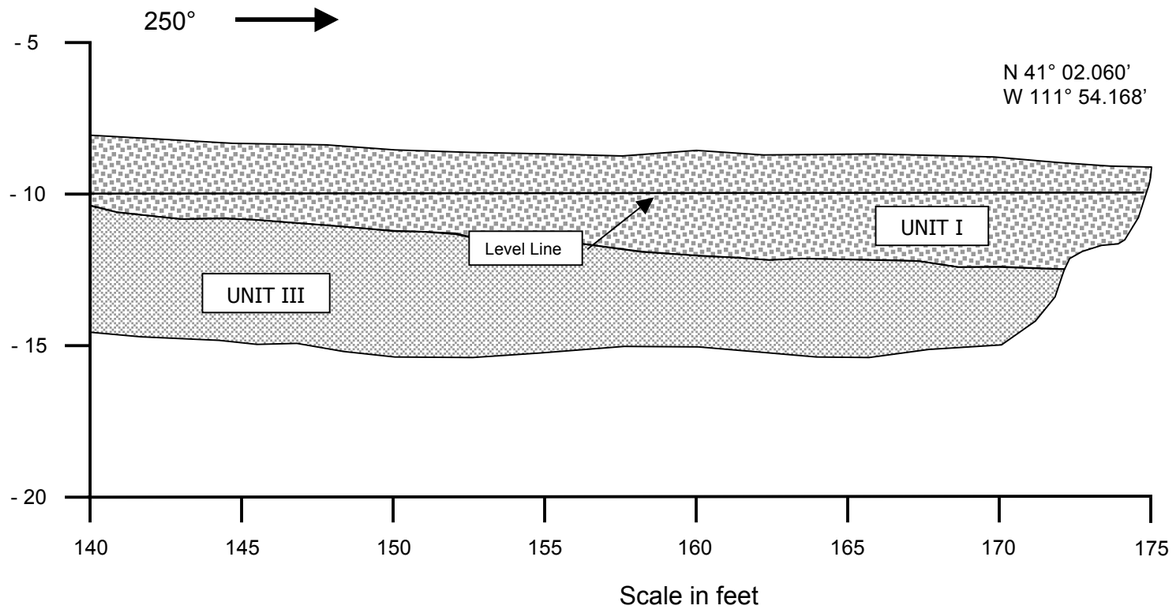
July 21, 2008

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See Figure 4b for Explanation of Geologic Units

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

Trench logged by Elliott W. Lips, P.G.
on May 7, 2008



EXPLANATION OF GEOLOGIC UNITS

Fill	Poorly sorted clay to cobbles, occasional construction debris.
Unit I	Silty gravels with sand and cobbles, poorly sorted to very poorly sorted, cobbles to 6", sub-rounded to sub-angular, occ. cobble to 12", thin to thick bedded, dark brown (7.5YR 3/3) to very dark grayish brown (10YR 3/2), few lenses of sandy gravel, gvl to 2". Debris-flow and/or colluvial deposits, Holocene.
Unit II	Silty sand with gravel, poorly sorted, gvl to 2", sub-rounded, weak cementation, loose, thin bedded, brown (7.5 YR 4/4) to dark brown (7.5YR 3/3), few lenses of clean sand and sandy gravel. Reworked lacustrine shorezone deposits, late-Pleistocene (Lake Bonneville at Provo stage).
Unit IIa	Silty sand as above, less gravel, better sorted.
Unit IIb	Silty sand as above, cleaner sand and gravel, better sorted
Unit III	Sand and gravel, well sorted, loose, med to csc sand, gvl to 2" occ. to 4", sub-rounded to well-rounded, thin to thick bedded, sub-horizontal (dip to west) planar beds, brown (7.5YR 4/3) to dark brown (7.5YR 3/4). Late-Pleistocene (Bonneville transgressive beach).
Unit IIIa	Sand and gravel as above, very minor gravel. Late-Pleistocene (Bonneville transgressive near-shore).
Unit IV	Bimodal: boulders, 1 to 6 feet diameter, angular, non-equidimensional, boulder to boulder contact, and sand and gravel matrix, well sorted, csc, brown (7.5YR 4/3) washed between boulders. Late-Pleistocene landslide or debris flow from Baer Canyon with Bonneville transgressive beach sand and gravel (Unit III) washed in between boulders.

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

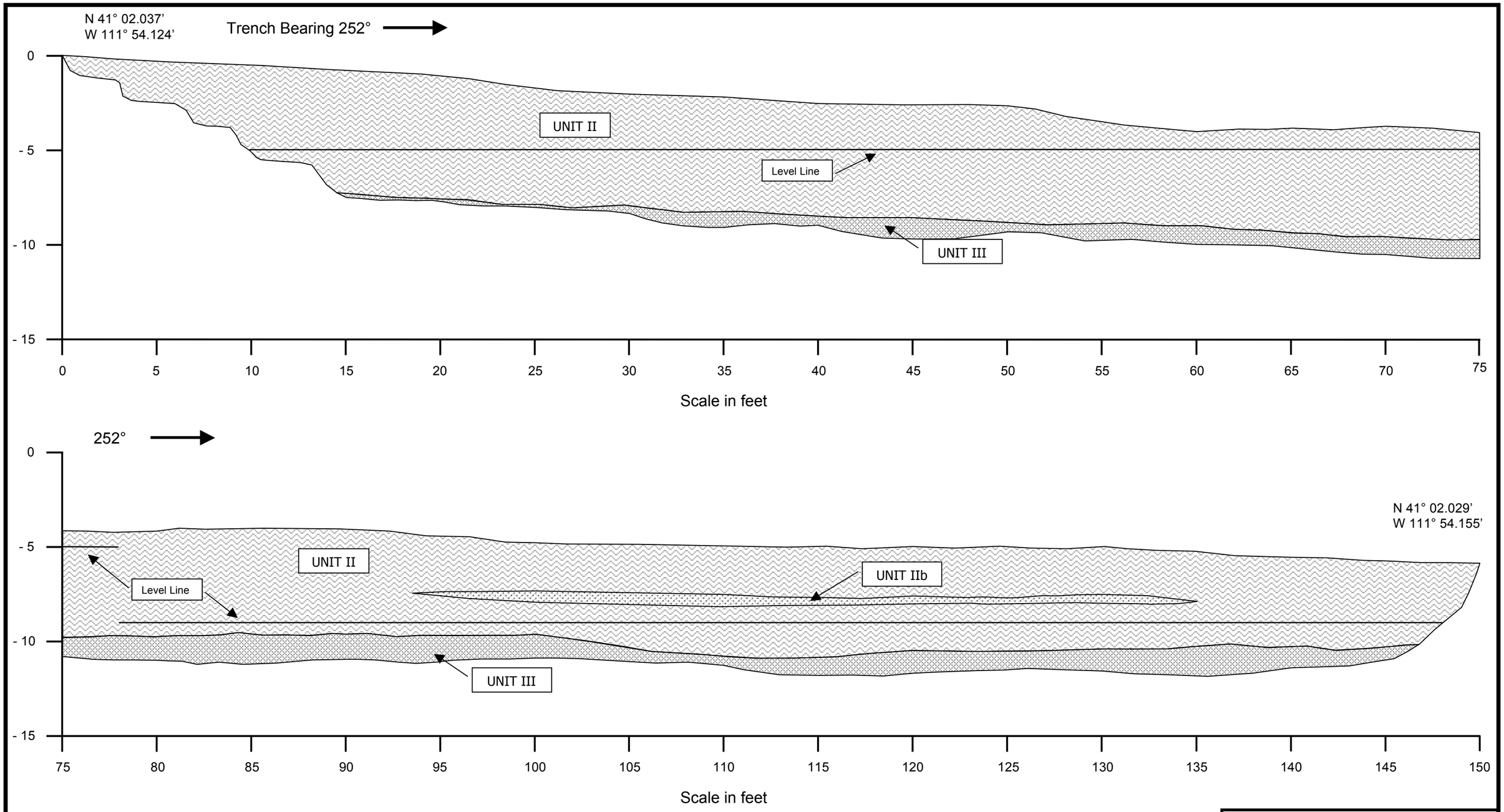
Trench logged by Elliott W. Lips, P.G.
on May 7, 2008

FIGURE 4b LOG OF TRENCH A

GEOLOGIC HAZARDS
EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008



Great Basin Earth Science, Inc.

See Figure 4b for Explanation of Geologic Units

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

Trench logged by Elliott W. Lips, P.G.
on May 7, 2008

FIGURE 5 LOG OF TRENCH B

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008

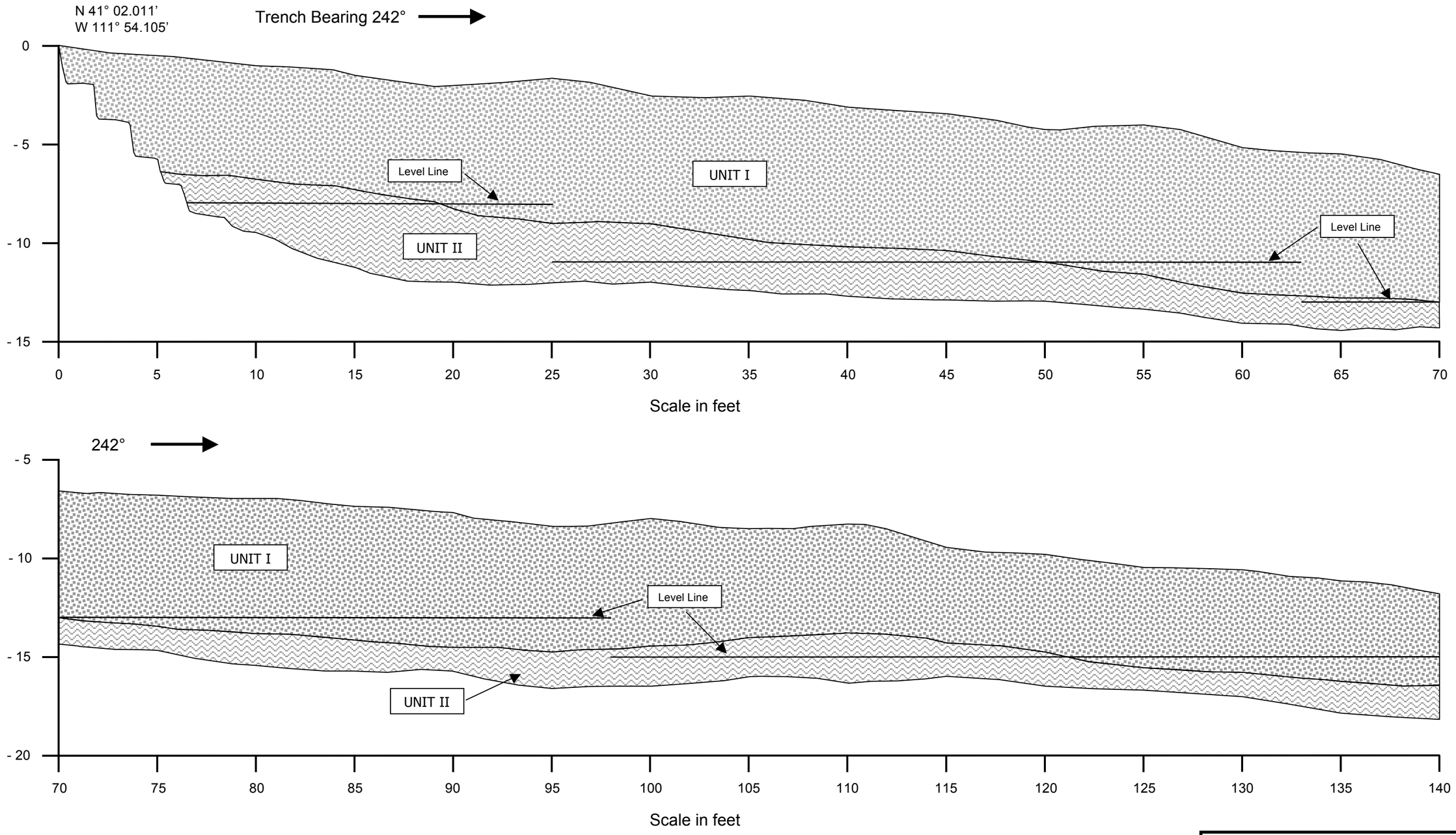


FIGURE 6a LOG OF TRENCH C

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

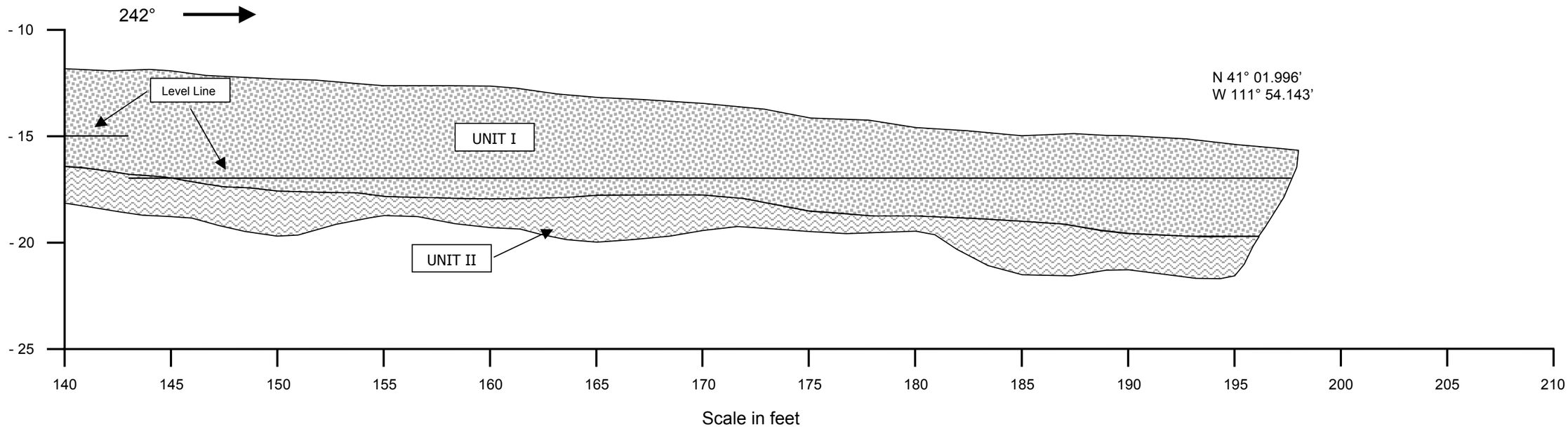
July 21, 2008

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See Figure 4b for Explanation of Geologic Units

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

Trench logged by Elliott W. Lips, P.G.
on May 8, 2008



Great Basin Earth Science, Inc.

See Figure 4b for Explanation of Geologic Units

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

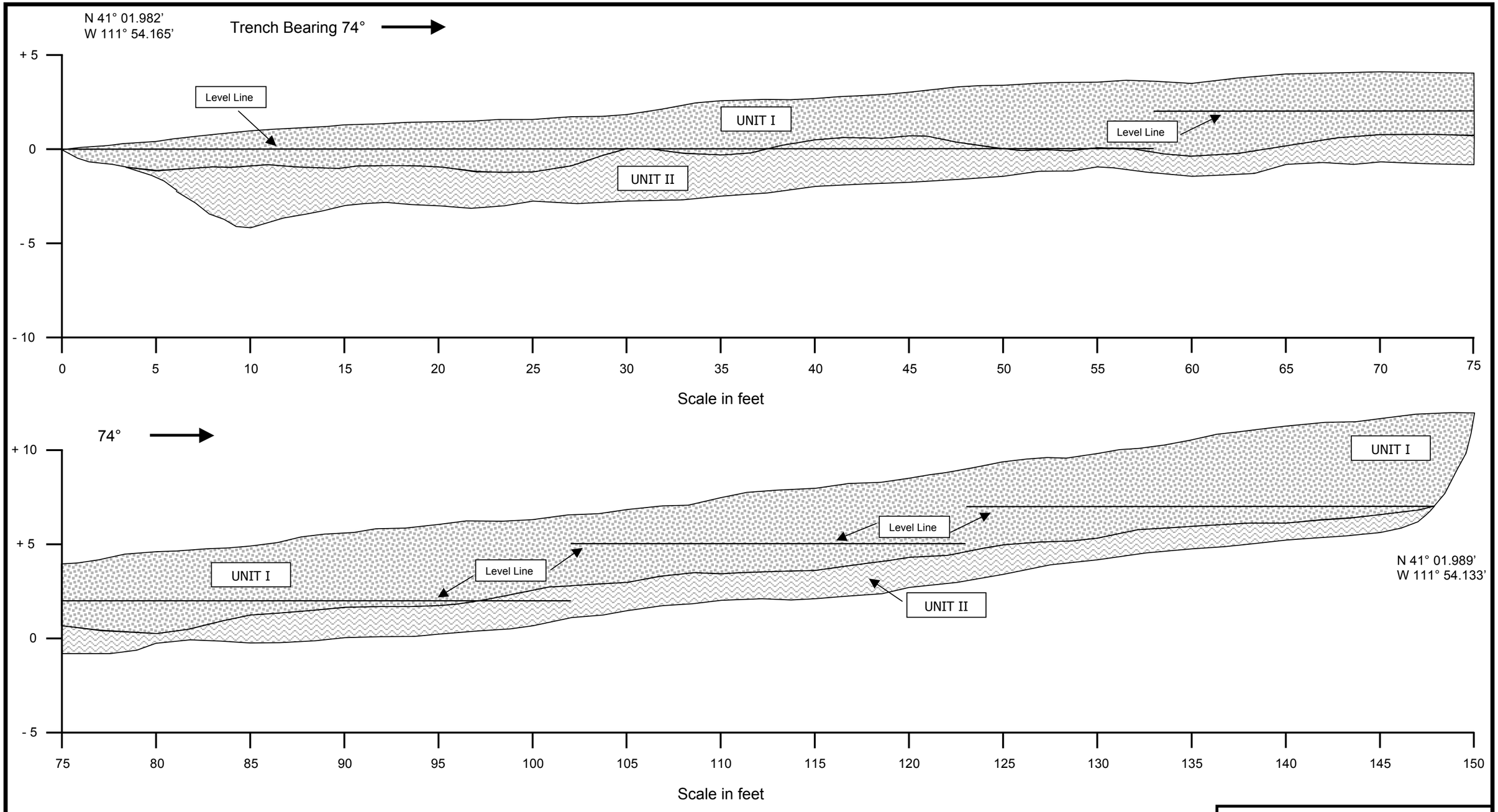
Trench logged by Elliott W. Lips, P.G.
on May 8, 2008

FIGURE 6b LOG OF TRENCH C

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008



Great Basin Earth Science, Inc.

See Figure 4b for Explanation of Geologic Units

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

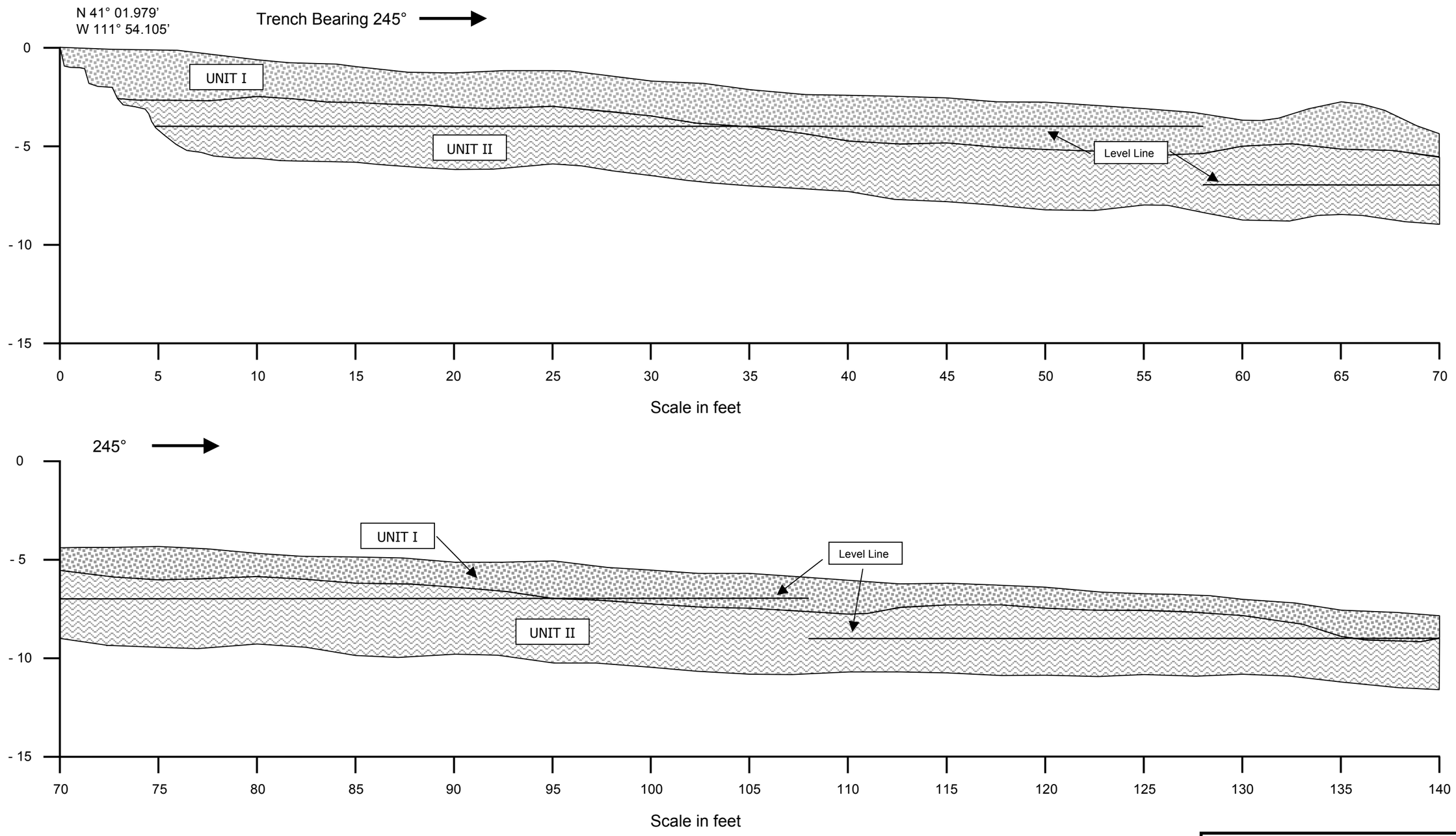
Trench logged by Elliott W. Lips, P.G.
on May 8, 2008

FIGURE 7 LOG OF TRENCH D

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008



Great Basin Earth Science, Inc.

See Figure 4b for Explanation of Geologic Units

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

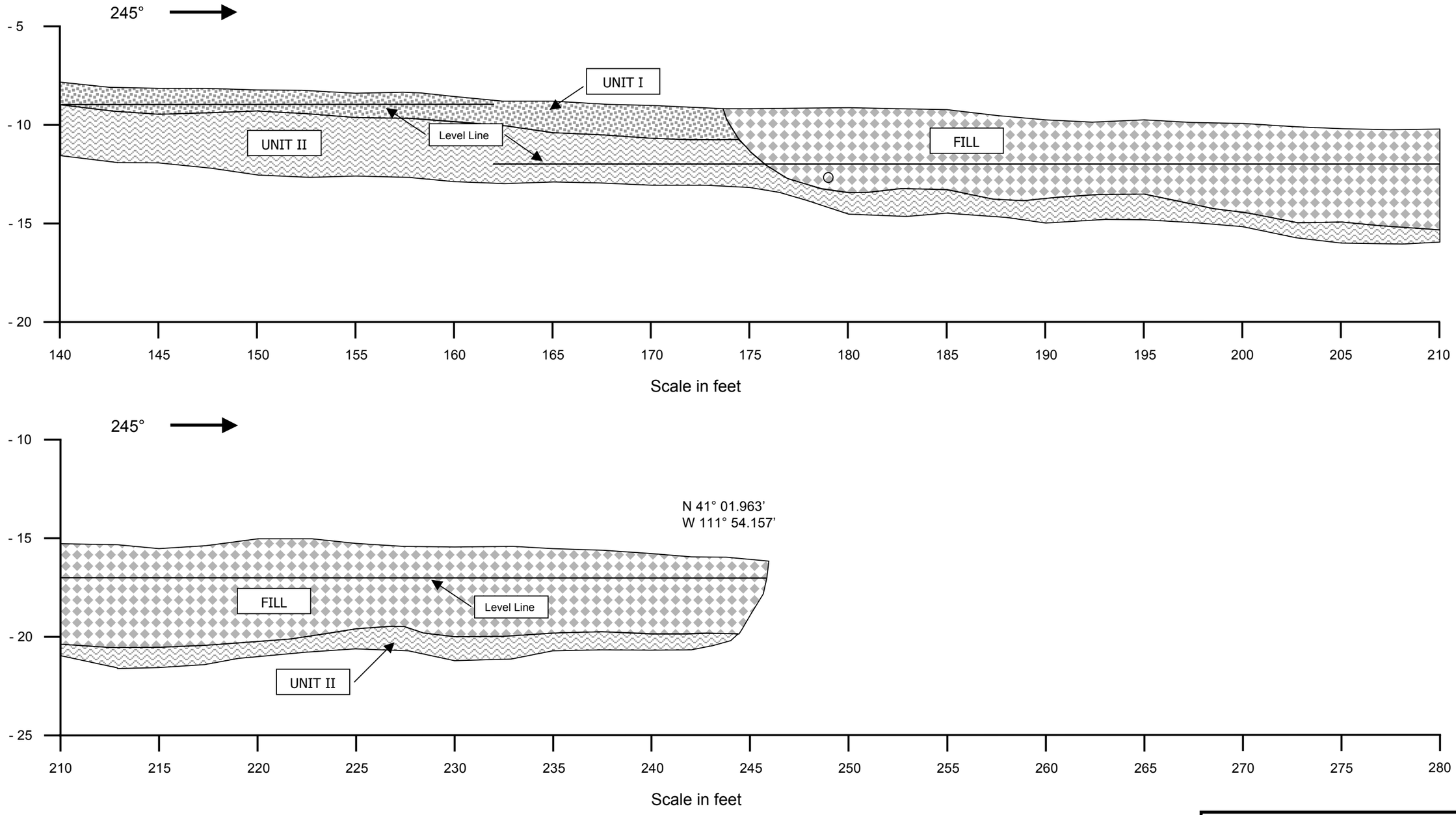
Trench logged by Elliott W. Lips, P.G.
on May 9, 2008

FIGURE 8a LOG OF TRENCH E

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008



Great Basin Earth Science, Inc.

See Figure 4b for Explanation of Geologic Units

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

Trench logged by Elliott W. Lips, P.G.
on May 9, 2008

FIGURE 8b LOG OF TRENCH E

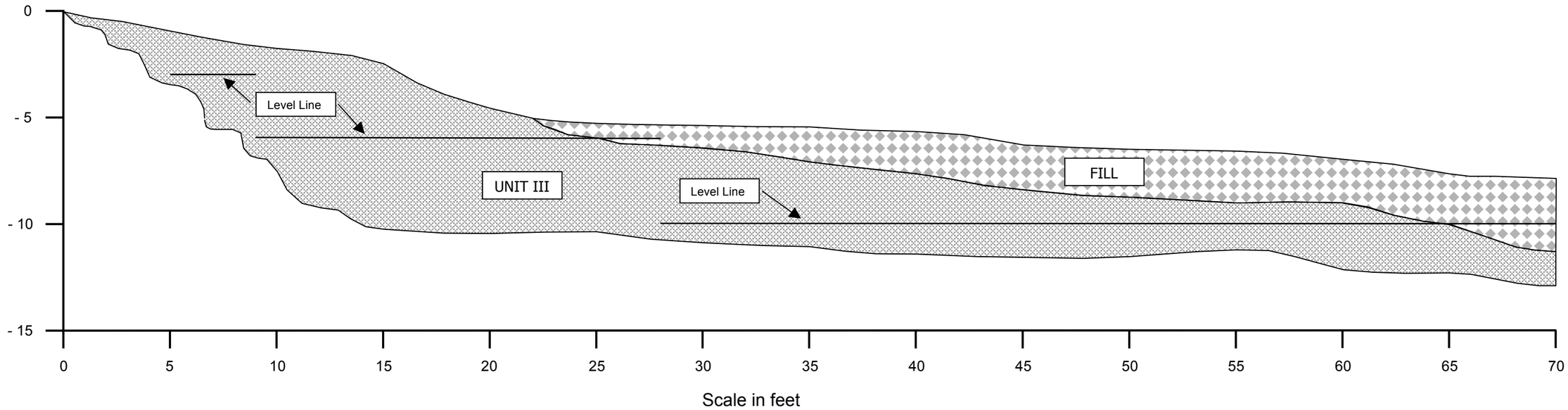
GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

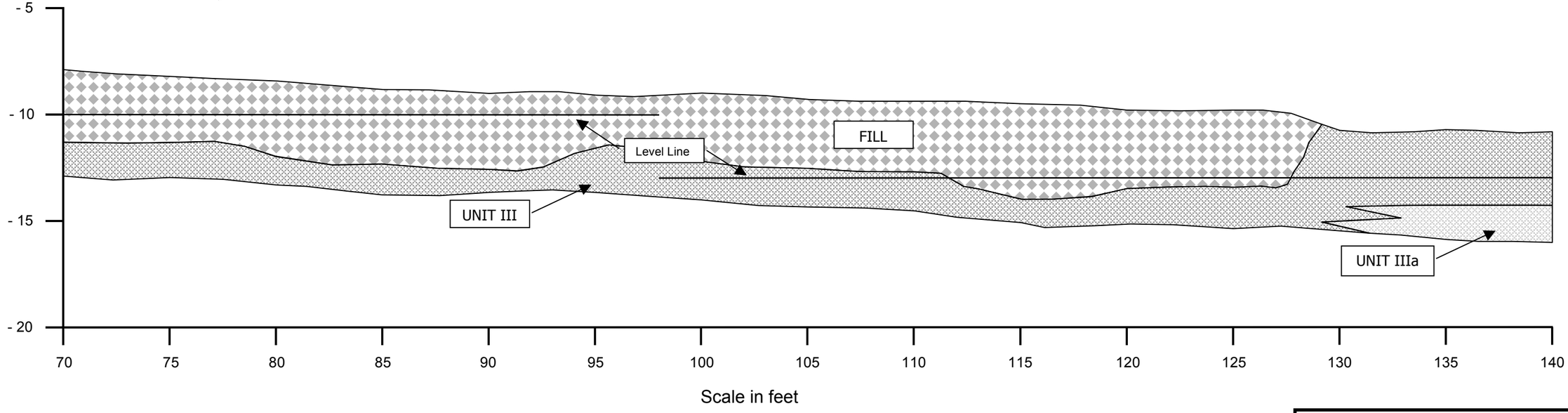
July 21, 2008

N 41° 01.939'
W 111° 54.097'

Trench Bearing 239° →



239° →



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See Figure 4b for Explanation of Geologic Units

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

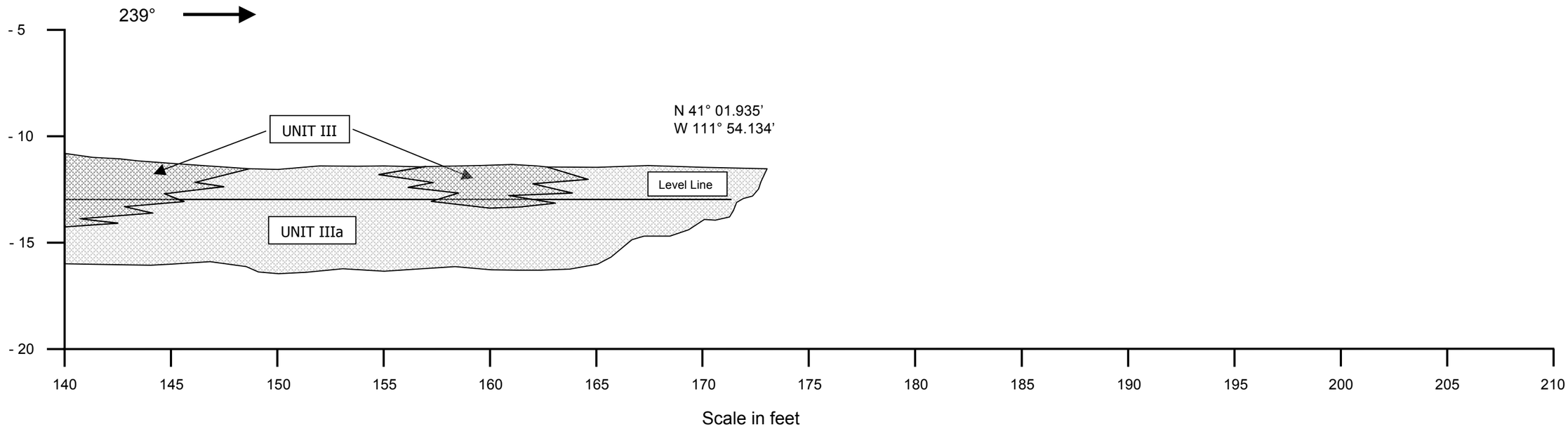
Trench logged by Elliott W. Lips, P.G.
on May 12, 2008

FIGURE 9a LOG OF TRENCH F

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008



Great Basin Earth Science, Inc.

See Figure 4b for Explanation of Geologic Units

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

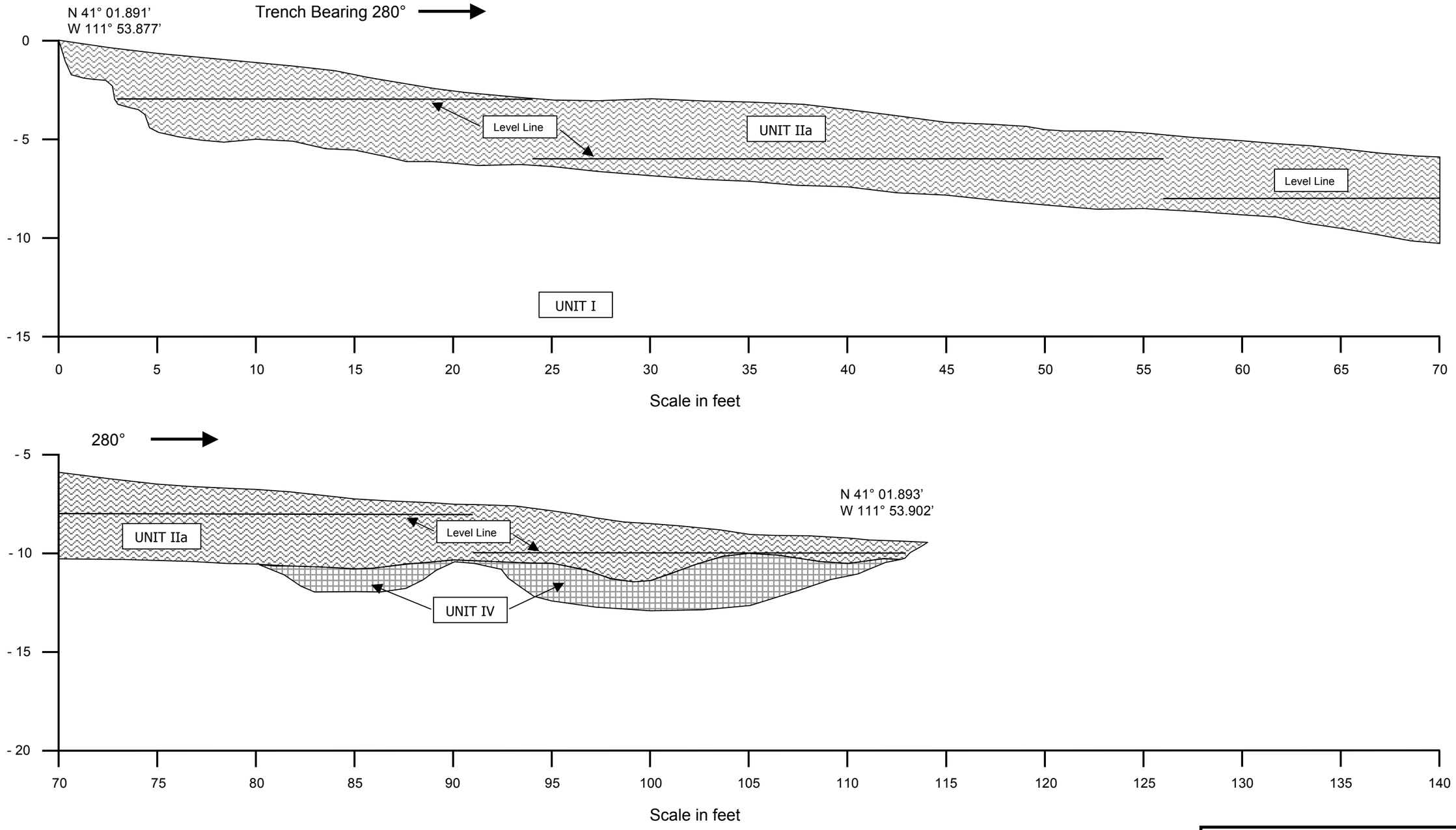
Trench logged by Elliott W. Lips, P.G.
on May 12, 2008

FIGURE 9b LOG OF TRENCH F

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008



Great Basin Earth Science, Inc.

See Figure 4b for Explanation of Geologic Units

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

Trench logged by Elliott W. Lips, P.G.
on May 12, 2008

FIGURE 10 LOG OF TRENCH G

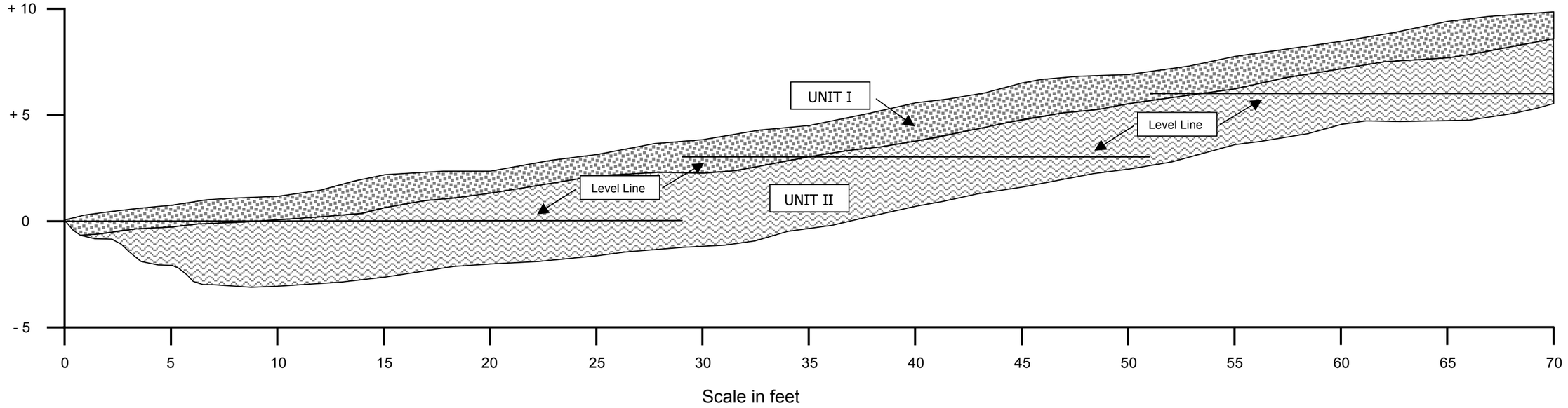
GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008

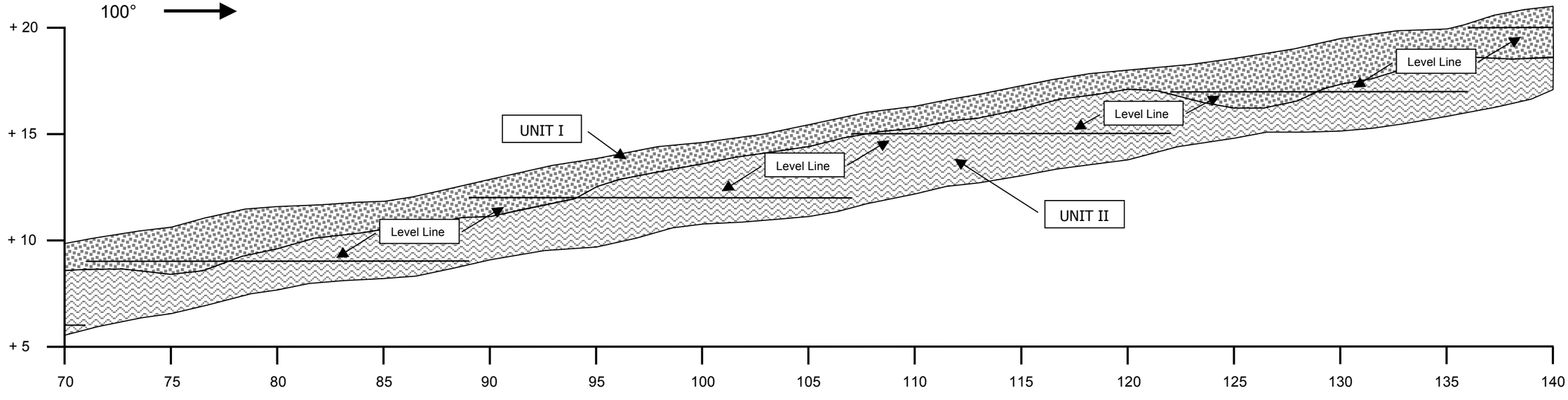
N 41° 02.001’
W 111° 54.040’

Trench Bearing 100° →



Scale in feet

100° →



Scale in feet

Great Basin Earth Science, Inc.

See Figure 4b for Explanation of Geologic Units

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

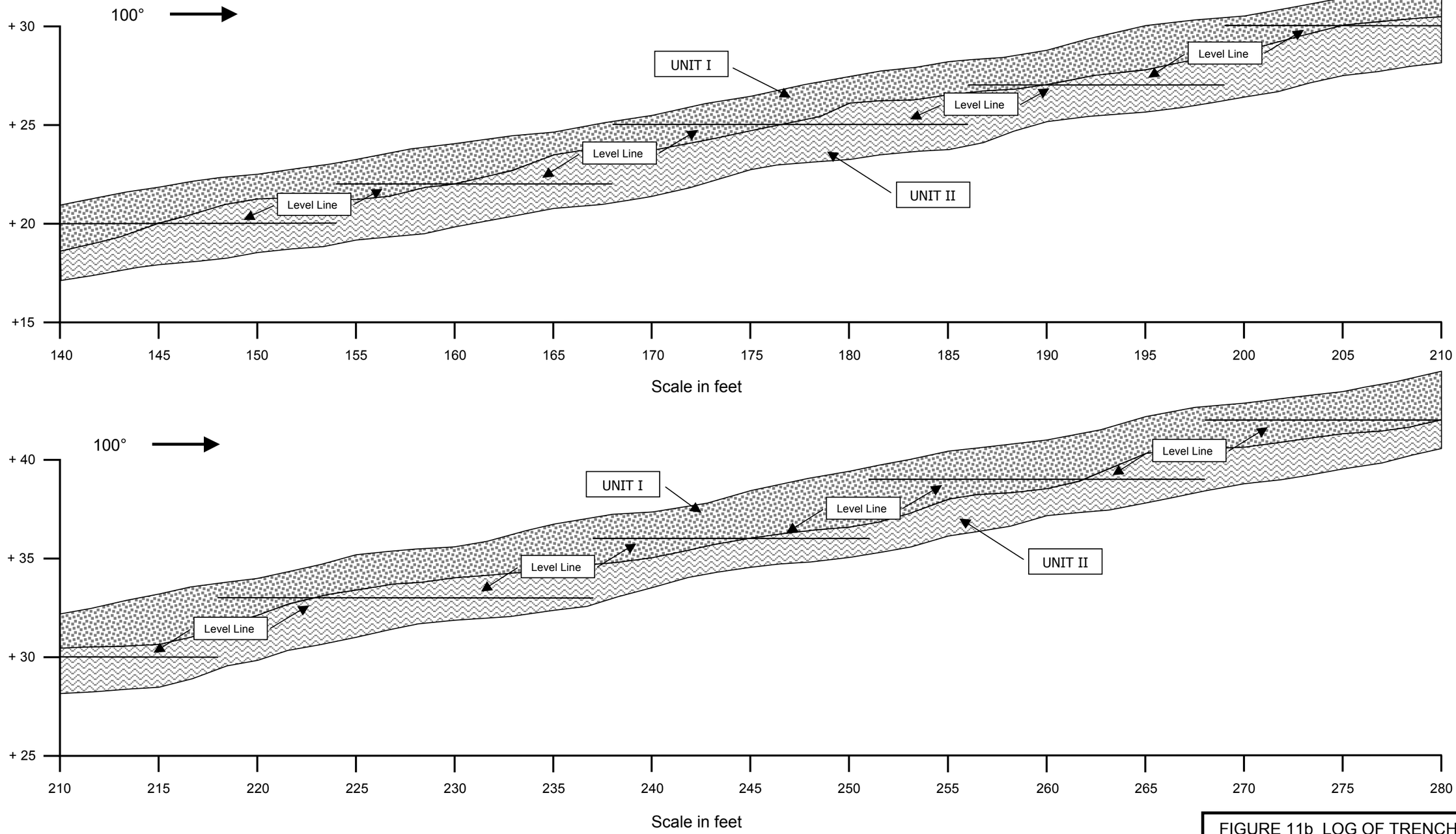
Trench logged by Elliott W. Lips, P.G.
on May 13, 2008

FIGURE 11a LOG OF TRENCH H

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008



Great Basin Earth Science, Inc.

See Figure 4b for Explanation of Geologic Units

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

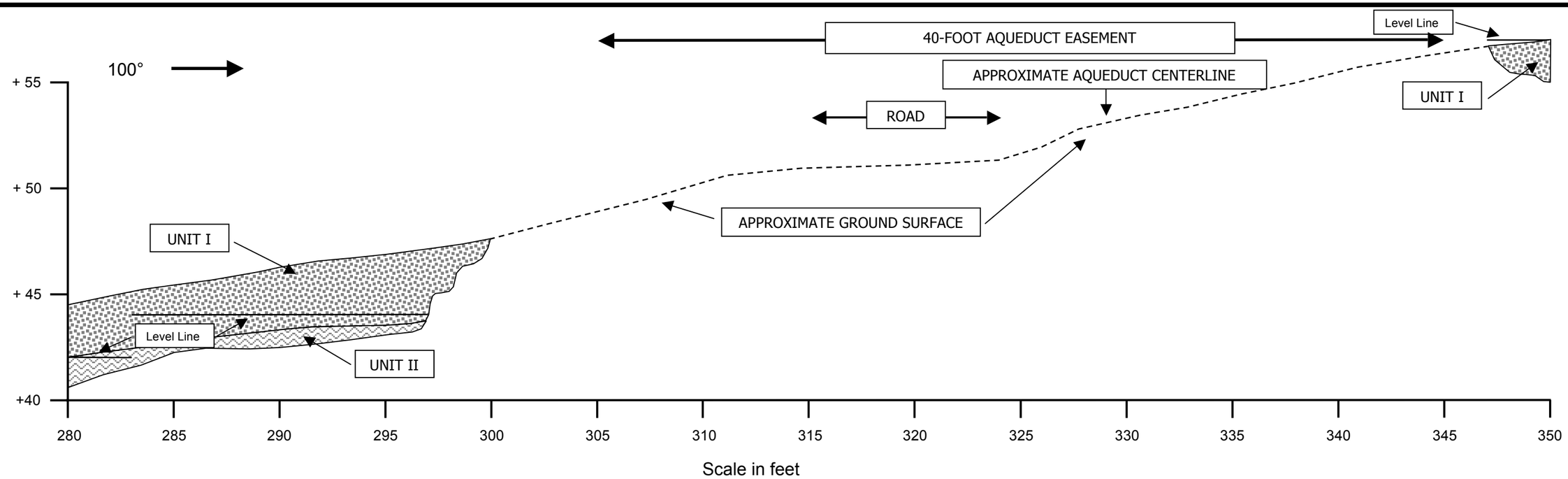
Trench logged by Elliott W. Lips, P.G.
on May 13, 2008

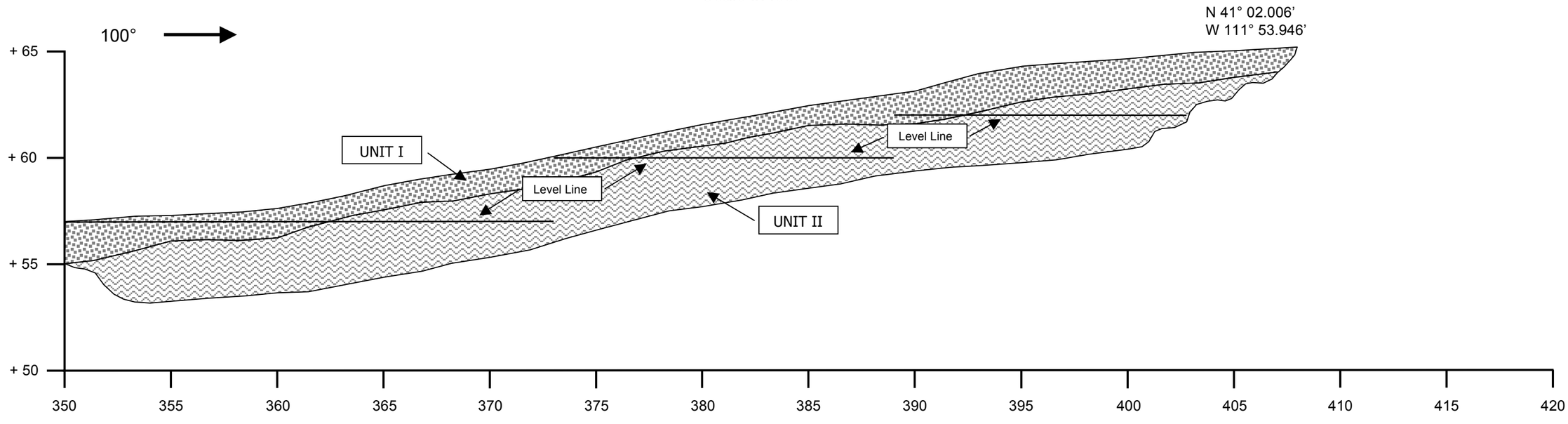
FIGURE 11b LOG OF TRENCH H

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008





Great Basin Earth Science, Inc.

See Figure 4b for Explanation of Geologic Units

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

Trench logged by Elliott W. Lips, P.G.
on May 13, 2008

FIGURE 11c LOG OF TRENCH H

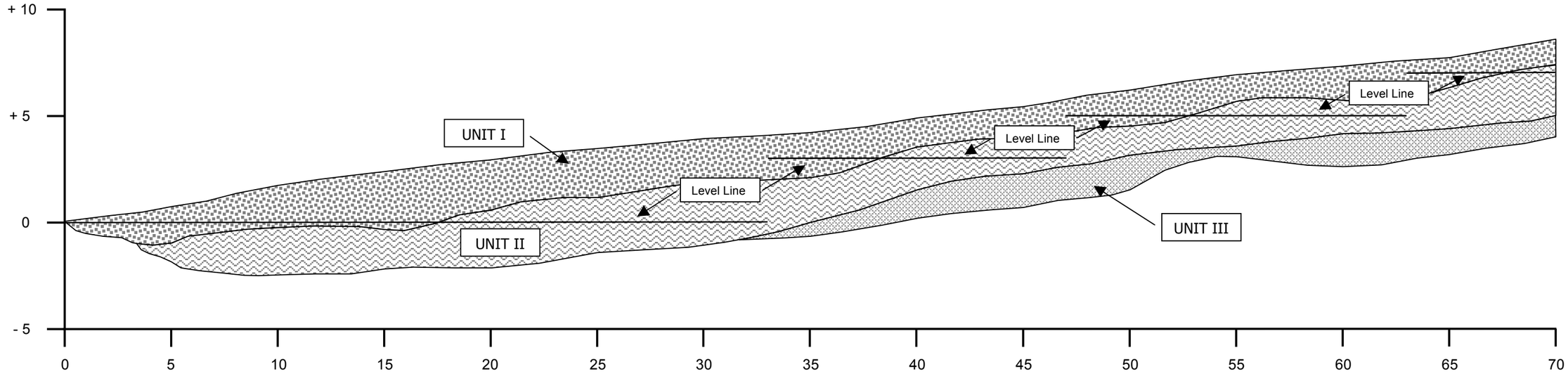
GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008

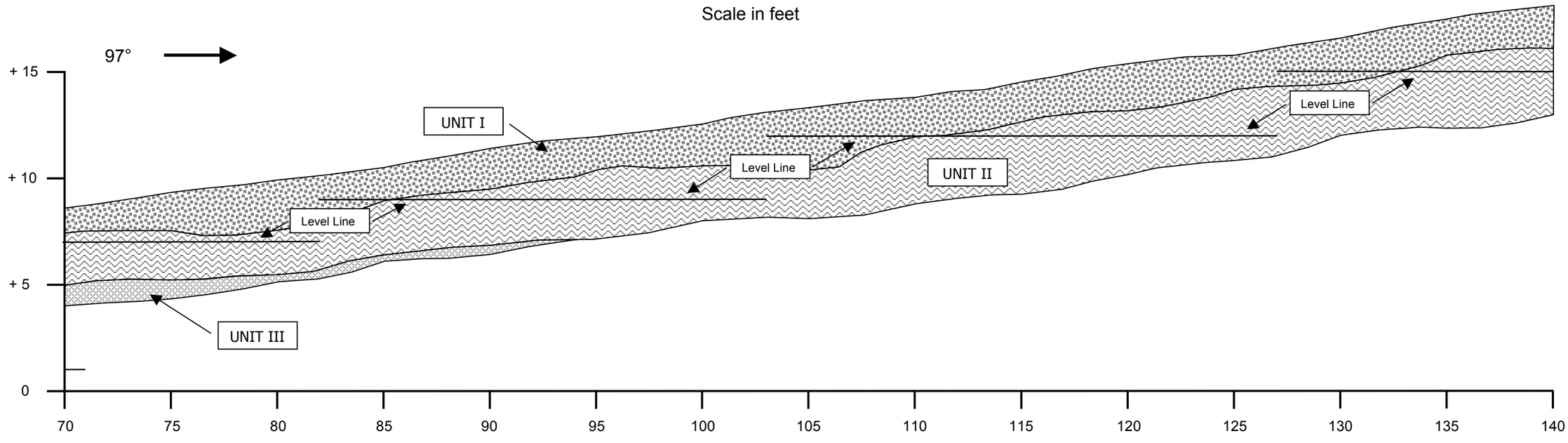
N 41° 02.098’
W 111° 54.108’

Trench Bearing 97° →



Scale in feet

97° →



Scale in feet

Great Basin Earth Science, Inc.

See Figure 4b for Explanation of Geologic Units

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

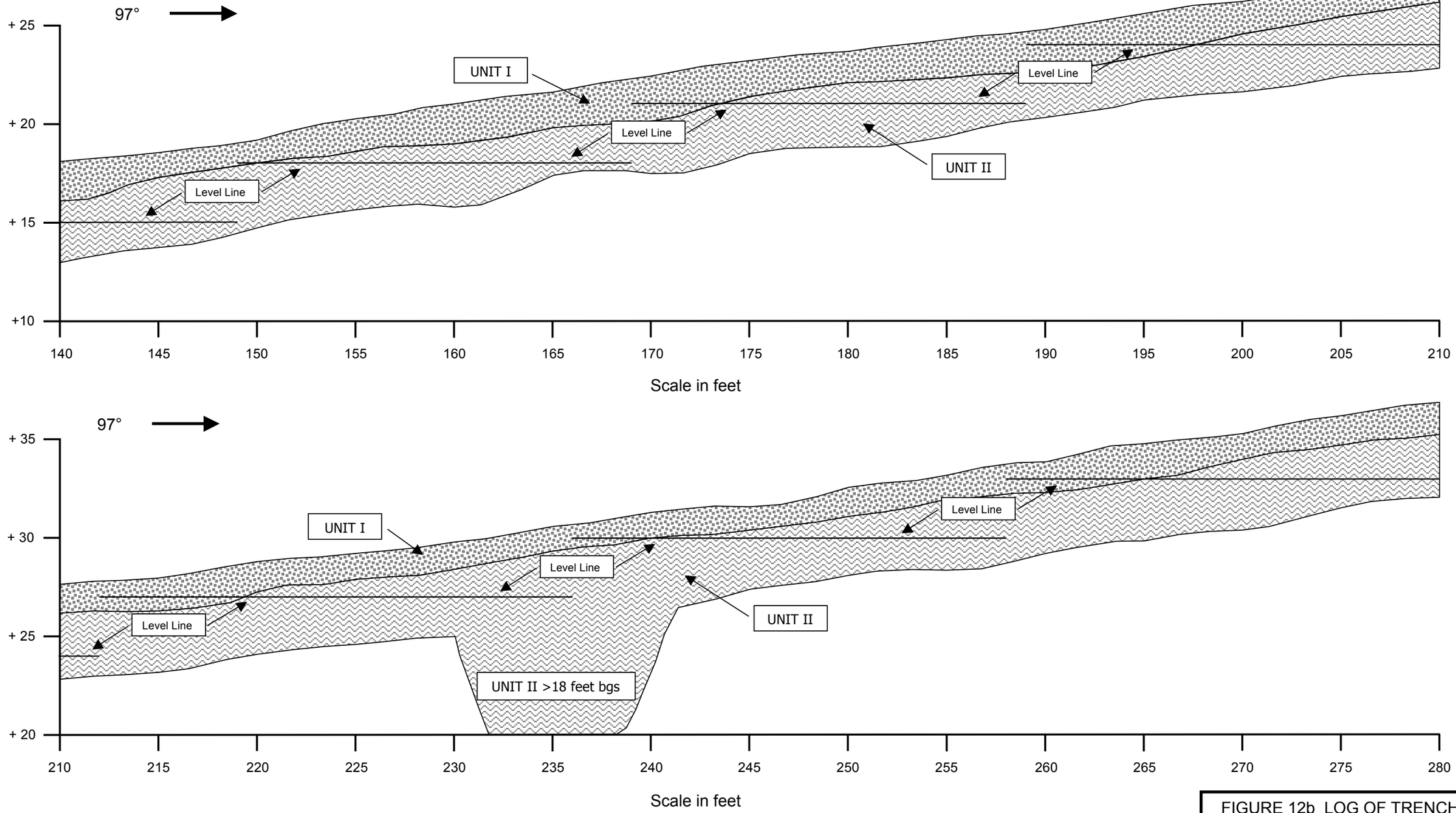
Trench logged by Elliott W. Lips, P.G.
on May 13, 2008

FIGURE 12a LOG OF TRENCH I

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008



Great Basin Earth Science, Inc.

See Figure 4b for Explanation of Geologic Units

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

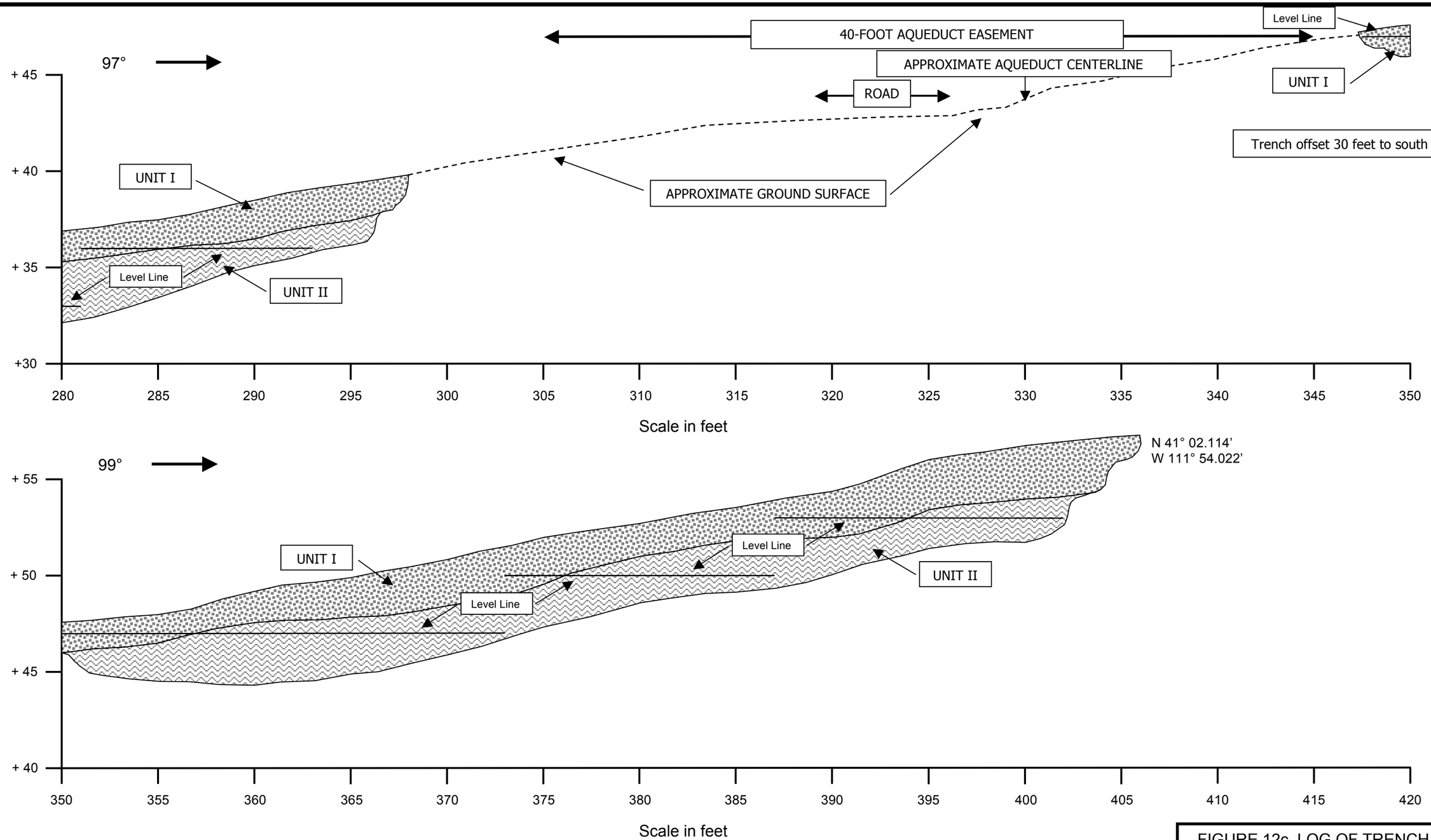
Trench logged by Elliott W. Lips, P.G.
on May 13, 2008

FIGURE 12b LOG OF TRENCH I

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008



Great Basin Earth Science, Inc.

See Figure 4b for Explanation of Geologic Units

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

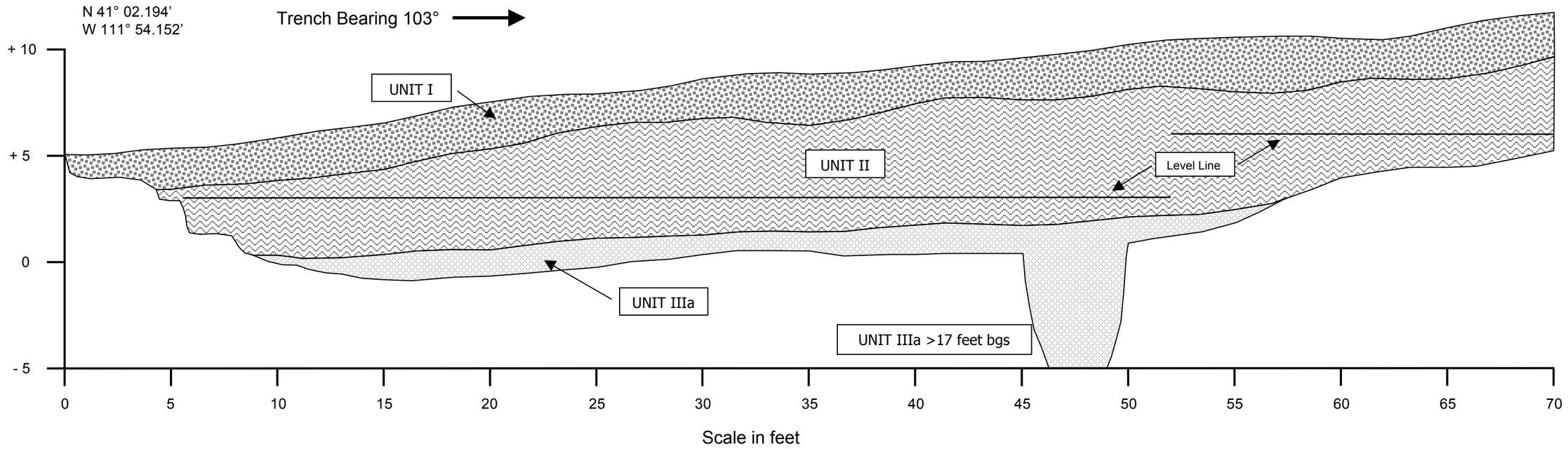
Trench logged by Elliott W. Lips, P.G.
on May 13, 2008

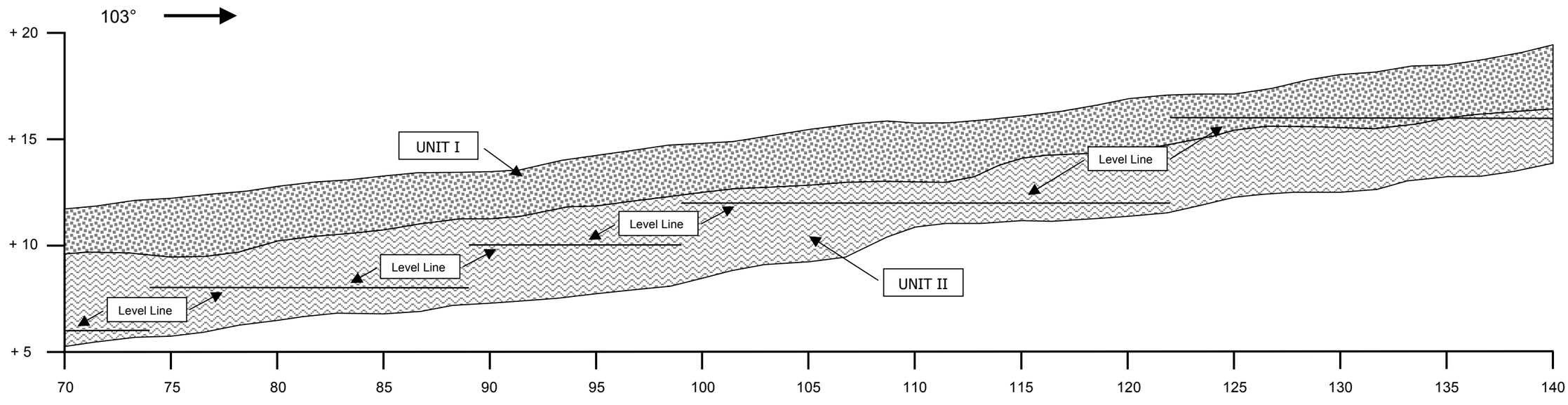
FIGURE 12c LOG OF TRENCH I

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008





Great Basin Earth Science, Inc.

See Figure 4b for Explanation of Geologic Units

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

Trench logged by Elliott W. Lips, P.G.
on May 14, 2008

FIGURE 13a LOG OF TRENCH J

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008

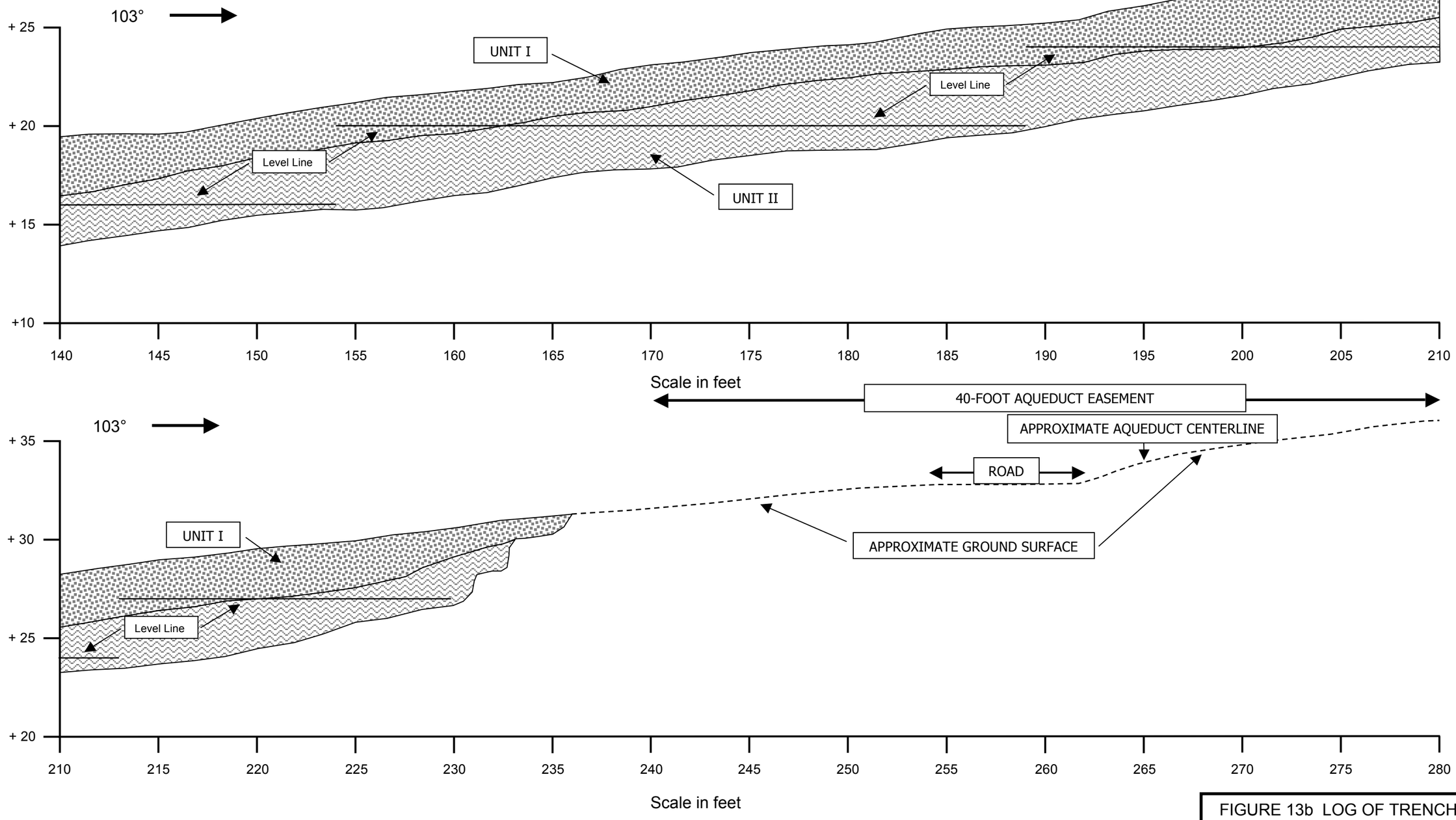


FIGURE 13b LOG OF TRENCH J

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

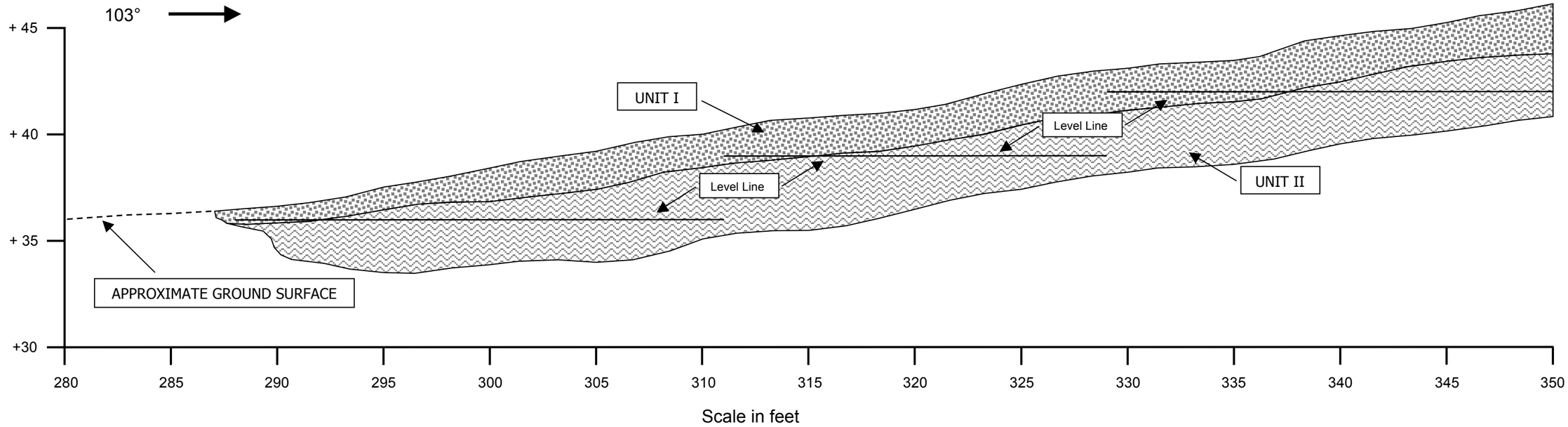
July 21, 2008

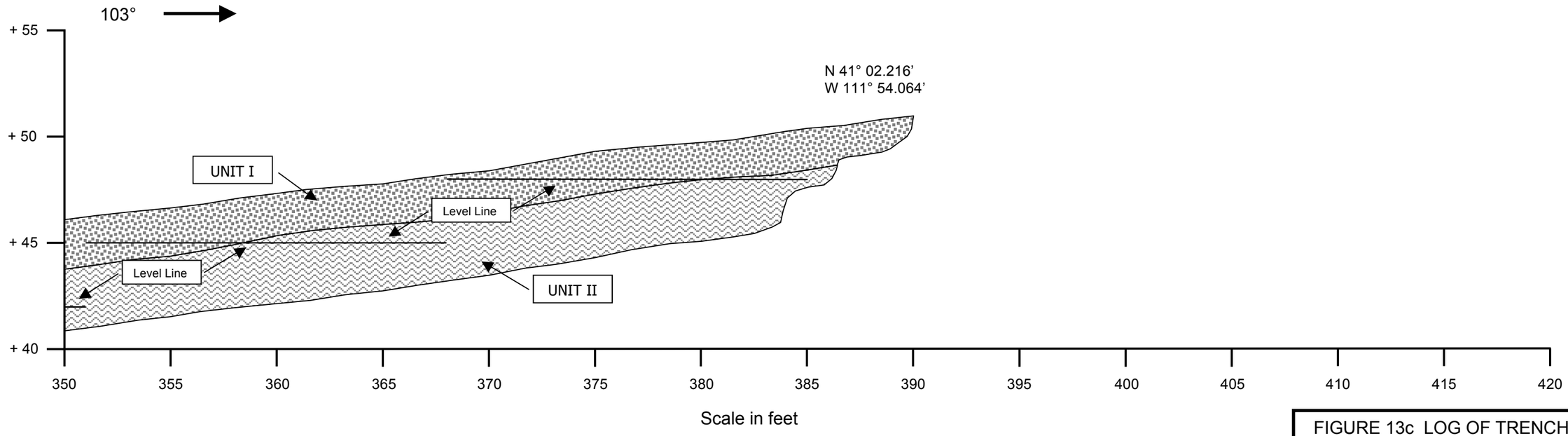
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
North Trench Wall Logged

Trench logged by Elliott W. Lips, P.G.
on May 14, 2008

See Figure 4b for Explanation of Geologic Units

Great Basin Earth Science, Inc.





Great Basin Earth Science, Inc.

See Figure 4b for Explanation of Geologic Units

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

Trench logged by Elliott W. Lips, P.G.
on May 14, 2008

FIGURE 13c LOG OF TRENCH J

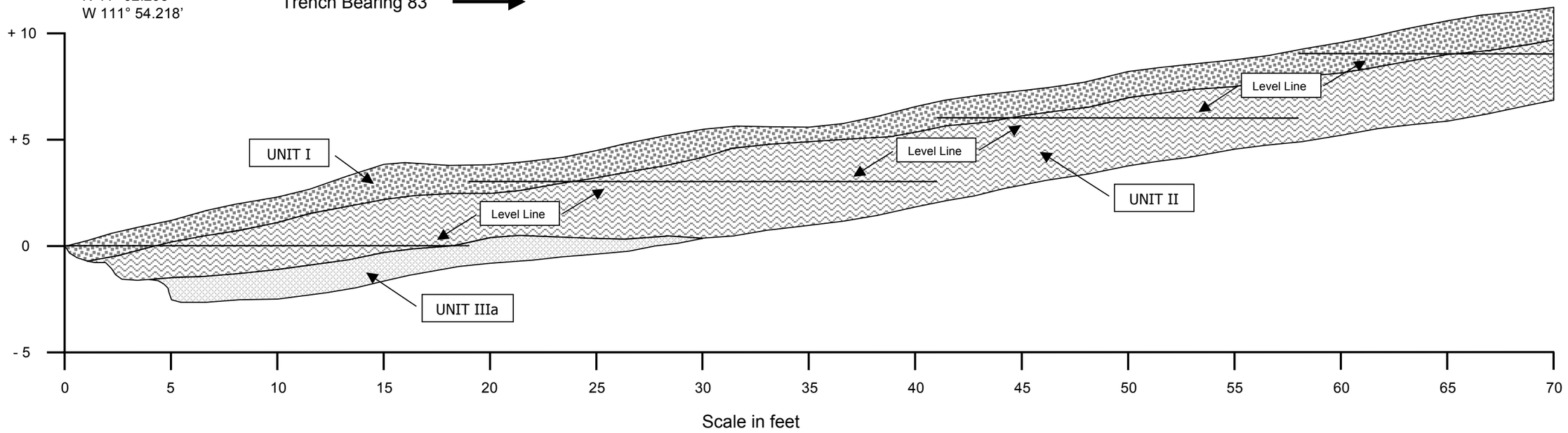
GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

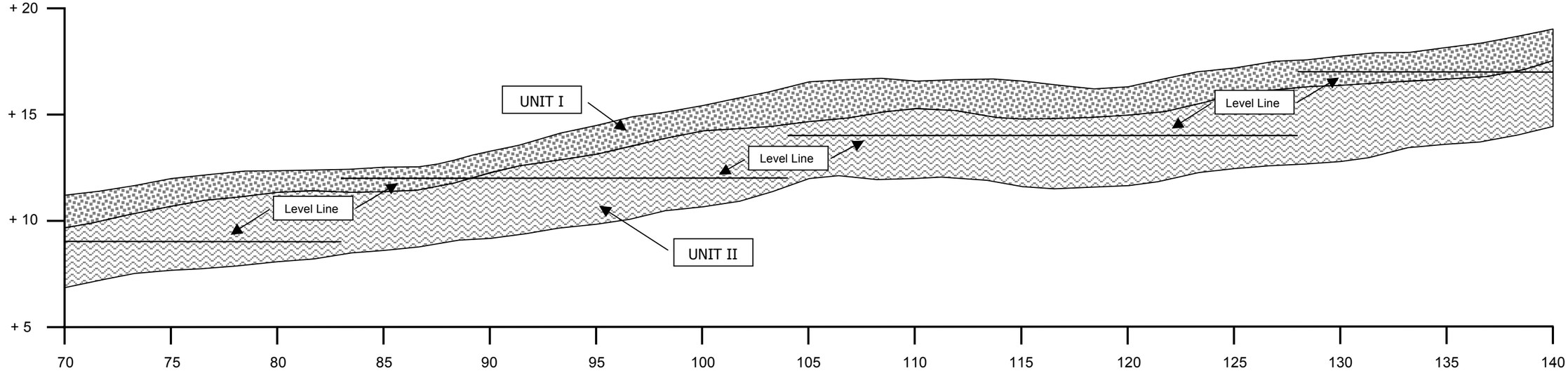
July 21, 2008

N 41° 02.295'
W 111° 54.218'

Trench Bearing 83° →



83° →



Great Basin Earth Science, Inc.

See Figure 4b for Explanation of Geologic Units

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

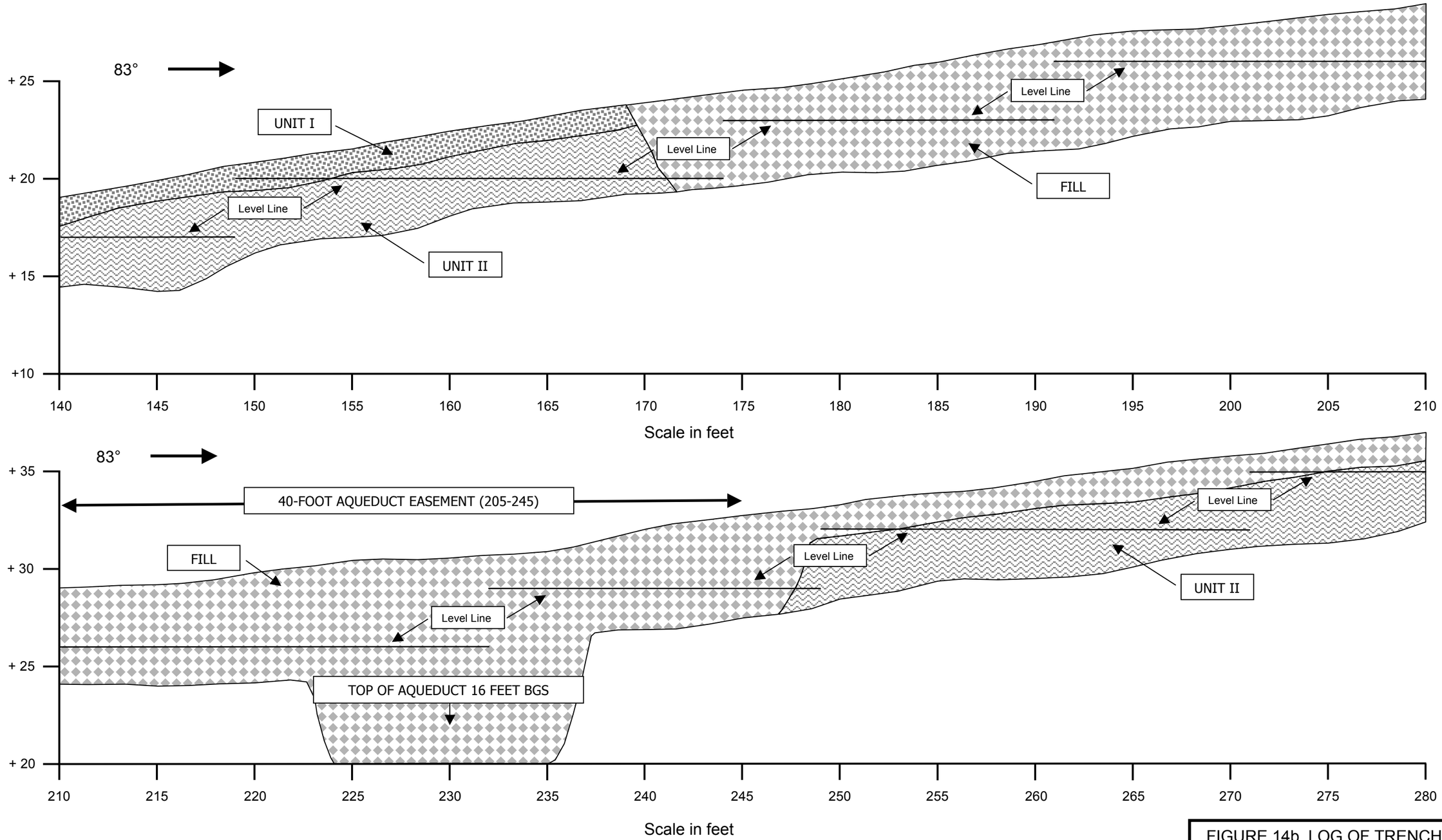
Trench logged by Elliott W. Lips, P.G.
on May 15, 2008

FIGURE 14a LOG OF TRENCH K

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008



Great Basin Earth Science, Inc.

See Figure 4b for Explanation of Geologic Units

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

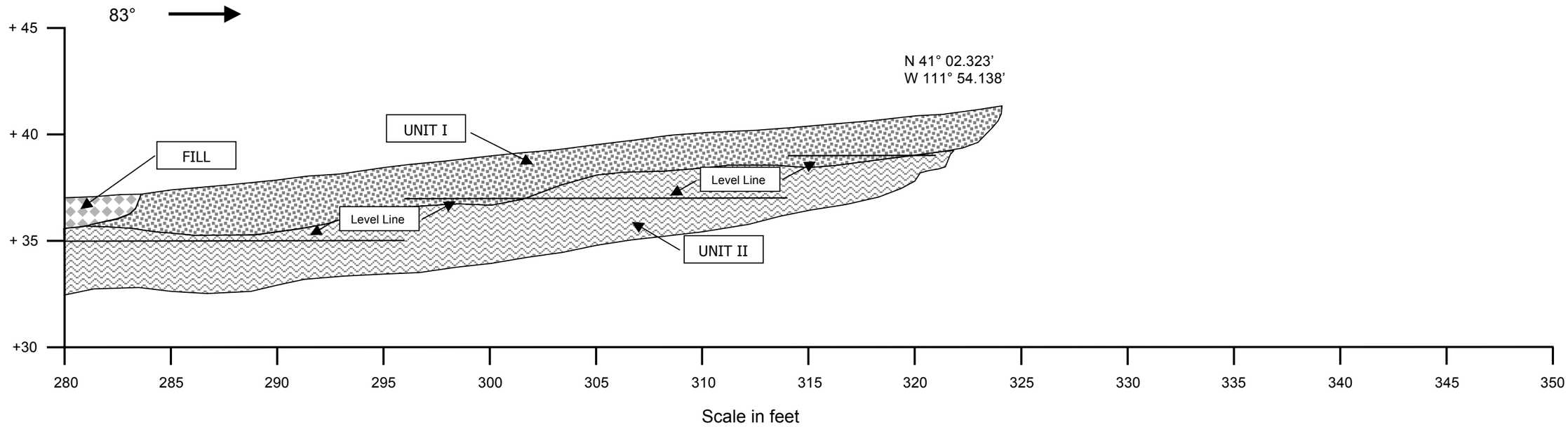
Trench logged by Elliott W. Lips, P.G.
on May 15, 2008

FIGURE 14b LOG OF TRENCH K

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008



Great Basin Earth Science, Inc.

See Figure 4b for Explanation of Geologic Units

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

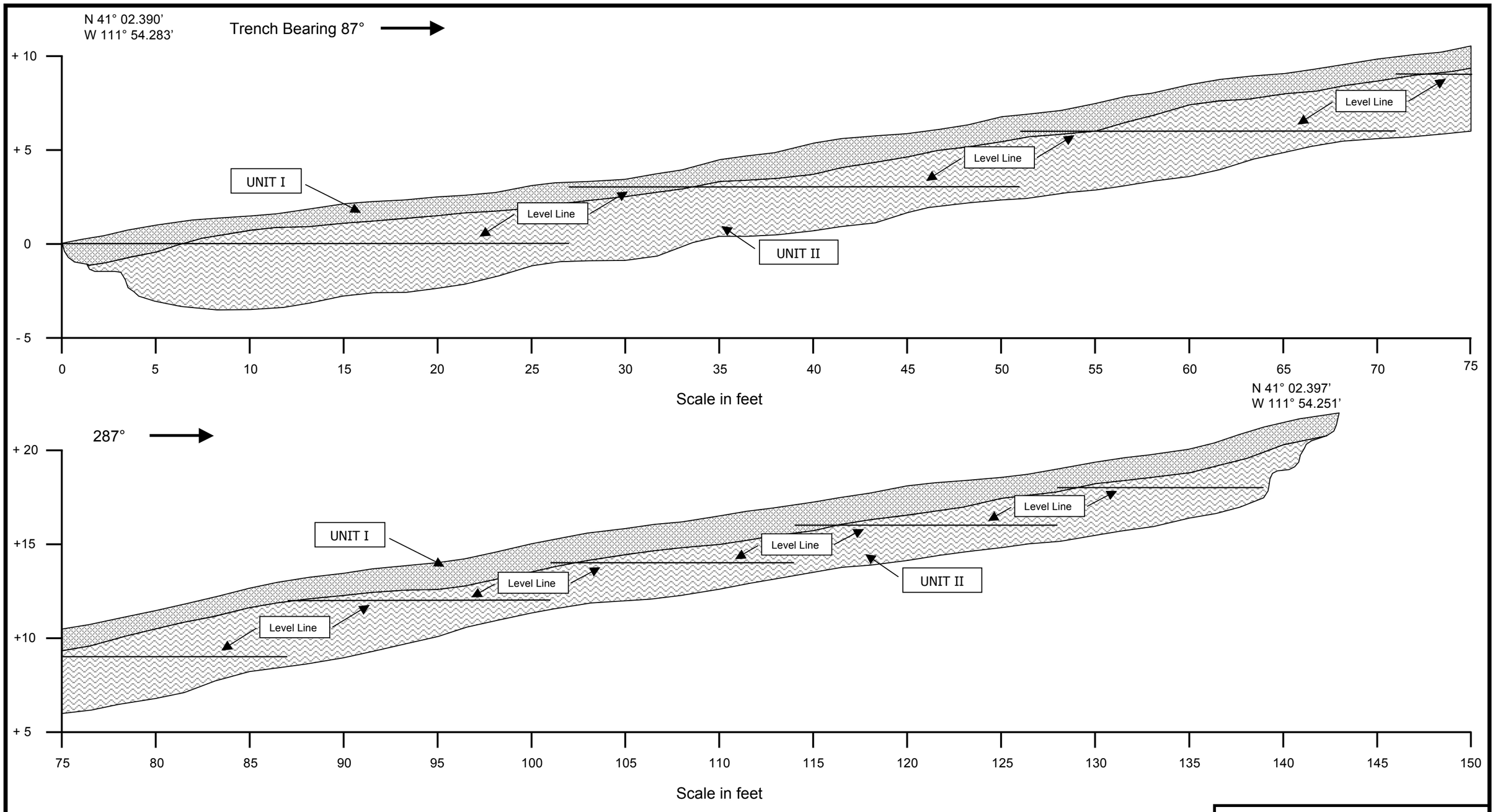
Trench logged by Elliott W. Lips, P.G.
on May 15, 2008

FIGURE 14c LOG OF TRENCH K

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008



Great Basin Earth Science, Inc.

See Figure 4b for Explanation of Geologic Units

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

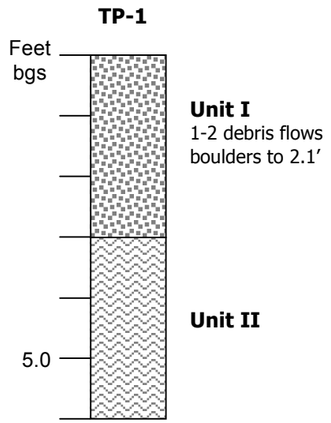
Trench logged by Elliott W. Lips, P.G.
on May 15, 2008

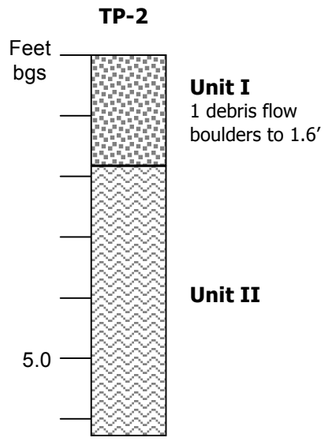
FIGURE 15 LOG OF TRENCH L

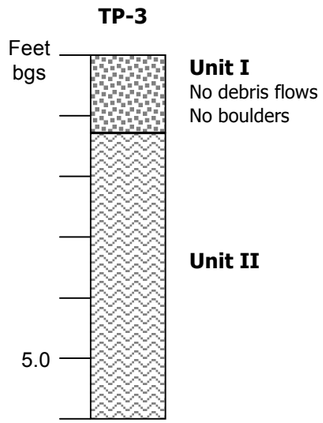
GEOLOGIC HAZARDS EVALUATION

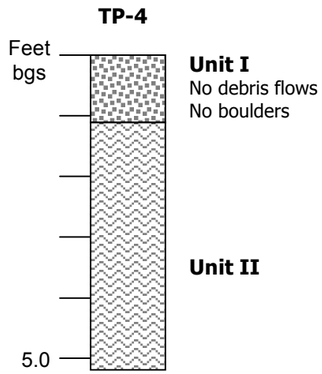
CHERRY RIDGE
Fruit Heights, Utah

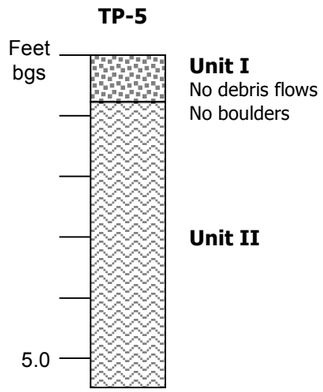
July 21, 2008

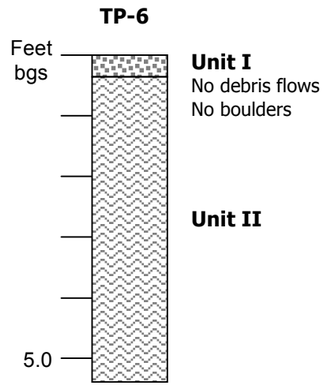


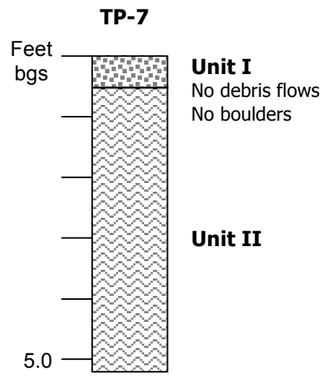


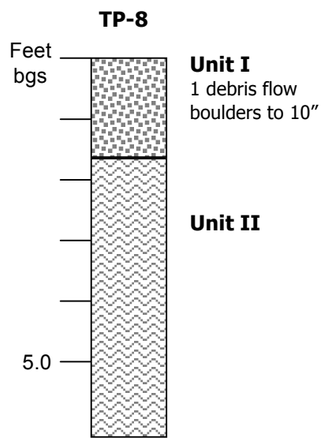


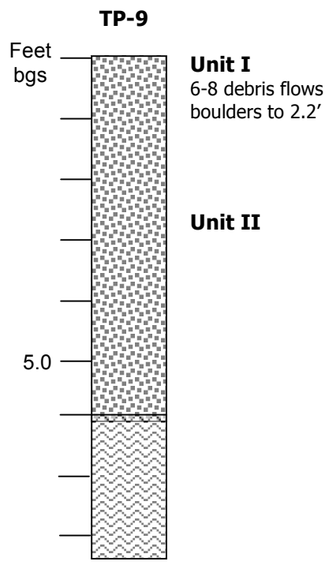


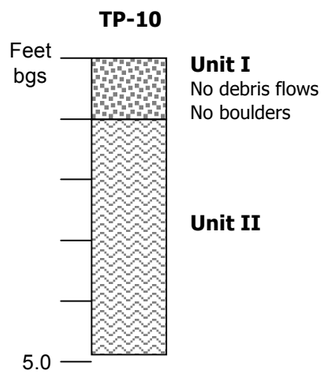


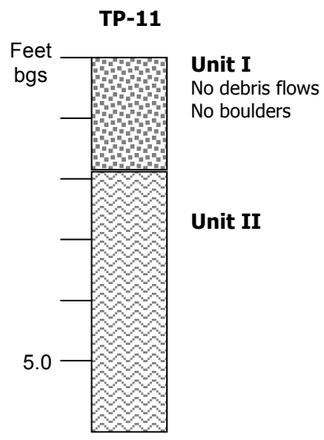


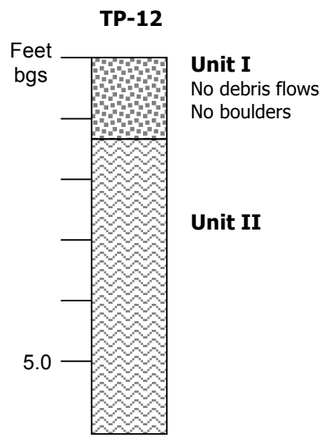












Great Basin Earth Science, Inc.

See Figure 4b for Explanation of Geologic Units

SCALE: 1 inch = 10 feet

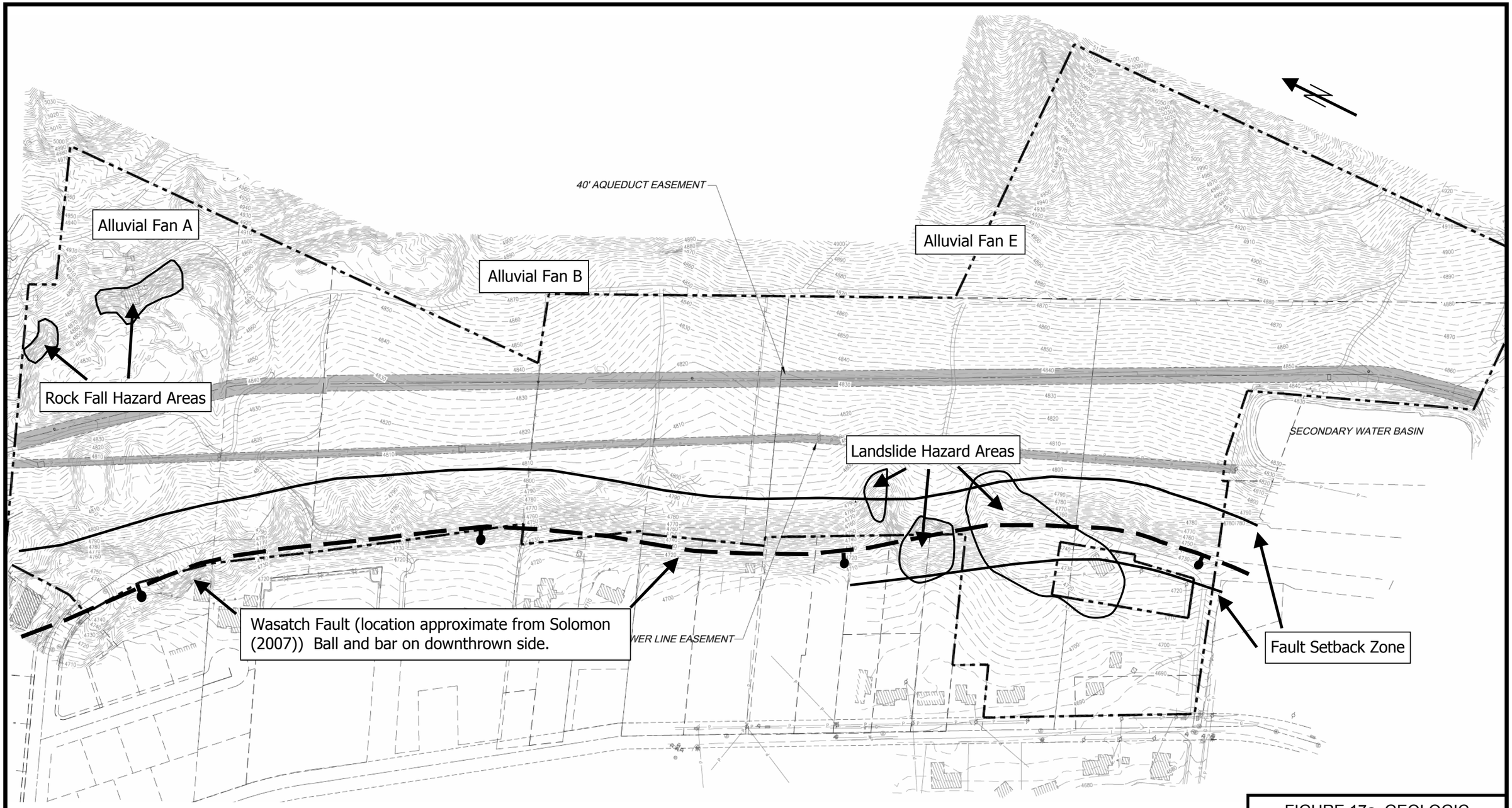
Test Pits logged by Elliott W. Lips, P.G. on May 16, 2008

FIGURE 16 LOG OF TEST PITS

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008



Great Basin Earth Science, Inc.

Scale: 1 inch = 250 feet

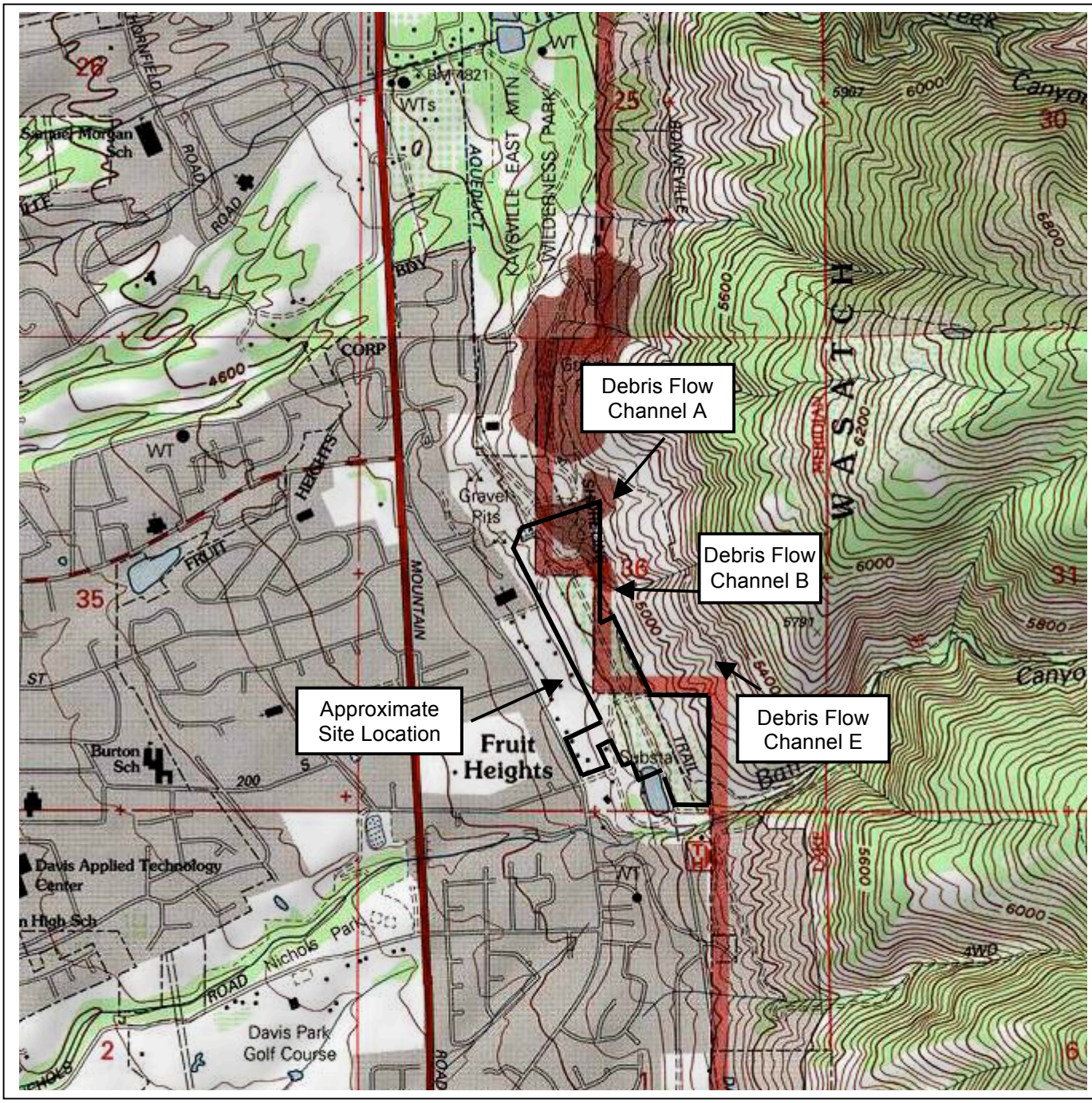
125 0 125 250

FIGURE 17a GEOLOGIC HAZARDS MAP

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008



Reference: U.S. Geologic Survey 7.5 Minute Series Topographic Maps – Kaysville, Utah, 1998

Great Basin Earth Science, Inc.

FIGURE 17b GEOLOGIC HAZARDS MAP

GEOLOGIC HAZARDS EVALUATION

CHERRY RIDGE
Fruit Heights, Utah

July 21, 2008

REPORT
GEOTECHNICAL AND SLOPE STABILITY ANALYSIS PROPOSED RAYMOND PROPERTY RESIDENTIAL DEVELOPMENT EAST OF THE INTERSECTION OF MOUNTAIN ROAD AND GREEN ROAD FRUIT HEIGHTS, UTAH

Submitted To:

Lowe Companies
Attention: Mr. Matt Lowe
6028 South Ridge Drive
South Ogden, Utah

Submitted By:

GSH Geotechnical, Inc.

September 24, 2014

Job No. 1696-001-14

September 24, 2014
Job No. 1696-001-14

Mr. Matt Lowe
Lowe Companies
6028 South Ridge Drive
South Ogden, Utah 84403

Mr. Lowe:

Re: Report
Geotechnical Study
Proposed Raymond Property Residential Development
East of the Intersection of Mountain Road and Green Road
Fruit Heights, Utah
(41.0329 N, 111.8996 W)

1. INTRODUCTION

1.1 GENERAL

This report presents the results of our geotechnical study performed at the site of the proposed Raymond Property Residential Development, to be located east of the intersection of Mountain Road and Green Road in Fruit Heights, Utah. The general location of the site with respect to major topographic features, as of 1998, is presented on Figure 1, Vicinity Map. A more detailed layout of the site showing existing and proposed facilities and roadways is presented on Figure 2, Site Plan. The locations of the test pits excavated in conjunction with this study are also presented on Figure 2.

1.2 OBJECTIVES AND SCOPE

The objectives and scope of our study were planned in discussions between Mr. Marr Lowe of the Lowe Companies and Mr. Alan Spilker of GSH Geotechnical, Inc. (GSH).

In general, the objectives of this study were to:

1. Define and evaluate the subsurface soil and groundwater conditions at the site.

2. Provide appropriate foundation, earthwork, slope stability, and pavement recommendations and geoseismic information to be utilized in the design and construction of the proposed development.

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

1. A field program consisting of the excavating, logging, and sampling of 15 exploration test pits.

2. A laboratory testing program.

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

1.3 AUTHORIZATION

Authorization was provided by returning a signed copy of our Proposal No. 14-0807 dated August 5, 2014.

1.4 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 test pits, 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, GSH 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

The proposed project consists of subdividing and constructing a 46 lot residential subdivision on the parcel located east of the intersection of Mountain Road and Green Road in Fruit Heights, Utah. The development will include single-family residences, installation of utilities to service the proposed residences, and associated roadways and pavements.

Construction will likely consist of reinforced concrete footings and basement foundation walls supporting 2 to 3 wood-framed levels above grade with some stone, brick, or stucco veneer. Projected maximum column and wall loads are on the order of 10 to 20 kips and 1 to 3 kips per lineal foot, respectively.

New residential roadways will be part of the development. It is anticipated that the residential streets will be constructed of asphalt pavement with relatively light projected traffic that includes

primarily passenger vehicles, daily delivery trucks, daily buses, and an occasional semi-tractor/trailer combination.

Site development will require a moderate amount of earthwork in the form of site grading. We estimate in general that maximum cuts and fills to achieve design grades will be on the order of 2 to 5 feet. Larger fills and cuts may be required at isolated areas and should be engineered accordingly to maintain stability of the slopes at the site.

3. SITE INVESTIGATIONS

3.1 FIELD PROGRAM

In order to define and evaluate the subsurface soil and groundwater conditions at the site, 15 test pits were excavated to depths of about 13.0 to 16.0 feet below existing grade. The test pits were excavated using a rubber tire-mounted backhoe. Test pit locations are presented on Figure 2.

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 excavating 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 3A through 3O, Test Pit Log. Soils were classified in accordance with the nomenclature described on Figure 4, Key to Test Pit Log (USCS).

A 2.42-inch inside diameter thin-wall drive sampler was utilized in the subsurface sampling at the site.

Following completion of excavating and logging, each test pit 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 LABORATORY TESTING

3.2.1 General

In order to provide data necessary for our engineering analyses, a laboratory testing program was performed. The program included performing moisture, density, partial gradation, direct shear, and chemical tests on representative subsurface soil samples. The following paragraphs describe the tests and summarize the test data.

3.2.2 Moisture and Density

To provide index parameters and to correlate other test data, moisture and density tests were performed on selected samples. The results of these tests are presented on the test pit logs, Figures 3A through 3O.

3.2.3 Partial Gradation Test

To aid in classifying the granular soils, partial gradation tests were performed. Results of the tests are tabulated below:

Test Pit No.	Depth (feet)	Percent Passing No. 200 Sieve	Soil Classification
TP-1	12.0	4	GP
TP-2	4.0	6	SP
TP-3	2.0	10	SP-SM
TP-3	10.5	13	SM
TP-4	8.0	13	SM
TP-5	7.5	55	ML
TP-5	15.0	33	SM
TP-6	9.0	13	SM
TP-6	14.0	7	SP-SM
TP-7	9.0	16	SM
TP-8	4.0	15	SM
TP-8	8.0	31	SM
TP-9	14.0	69	ML
TP-10	10.5	7	SP-SM
TP-11	2.0	14	SM
TP-11	12.0	11	SM
TP-12	3.0	9	SP-SM
TP-12	15.5	6	SP
TP-13	3.0	27	GM
TP-14	8.0	6	SP

TP-15	10.0	31	SM

3.2.4 Laboratory Direct Shear Tests

To determine the shear strength of the soils encountered at the site, laboratory direct shear tests were performed. The results of the tests are tabulated below:

Test Pit No.	Depth (feet)	Soil Type	In-Situ Moisture Content (percent)	Dry Density (pcf)	Internal Friction Angle (degrees)	Apparent Cohesion (psf)
TP-4	3.5	SM	4	104	32	325
TP-8	8.0	SM	6	88	32	20

3.2.5 Chemical Tests

In order to determine if the site soils will react detrimentally with concrete, chemical tests were performed on representative samples of the onsite soils. The results of the chemical tests are tabulated below:

Boring No.	Depth (feet)	Soil Classification	pH	Total Water Soluble Sulfate SO_4 (mg/kg-dry)
TP-3	5.0	SP-SM	6.9	None Detected
TP-12	3.0	SP-SM	7.0	None Detected

4. SITE CONDITIONS

4.1 SURFACE

The subject property consists of two parcels east of the intersection of Green Road and Mountain Road in Fruit Heights, Utah. The first parcel is a small rectangular parcel at the northeast corner of the intersection of Mountain Road and Green Road. It is anticipated that three lots will be developed on this parcel. The parcel contains an existing home and outbuilding along the north edge of the property. Vegetation at the site consists primarily of native weeds, grasses, and a number of mature trees. The subject property is relatively flat and generally slopes to the west-southwest. The parcel is bordered by Mountain Road to the west, Green Road to the south, and residential development to the north and east.

The second parcel is a large, irregularly shaped parcel east of the end of Green Road. It is anticipated that 43 lots will be developed within this parcel. The subject property is currently utilized as an orchard and contains a number of small, unpaved roads as well as an extensive irrigation system. Vegetation at the site consists primarily of native weeds, grasses, and many mature trees. The subject property generally slopes to the west-southwest at grades of approximately 10% to 35% with a change in elevation of about 350 feet across the subject property. The subject property is bordered by a residential development to the northwest, by an irrigation reservoir to the southwest, and by undeveloped property to the north, south, and east.

4.2 SUBSURFACE SOIL

The soil conditions encountered vary slightly across the site. At the test pit locations, topsoil and disturbed soils were encountered at the surface of the site to about 2 to 6 inches below existing grades. Natural soils were encountered beneath the topsoil and disturbed soils within the test pits to the full depth penetrated, about 13.0 to 16.0 feet, and consisted of fine to coarse sand with varying amounts of silt and fine and coarse gravel, silty fine and coarse gravel with fine to coarse sand, fine sandy silt, and occasional mixtures of these soils. Occasional cobbles and boulders were encountered within the sands and gravels at the site. Major soil layering encountered ranged from a thickness of roughly 1.0 feet up to about 16.0 feet.

The natural fine-grained silt soils encountered were medium stiff to stiff, moist, light brown in color, and will exhibit moderate strength and moderate compressibility characteristics under the anticipated loading range.

The natural sand and gravel soils encountered were loose to dense, slightly moist to moist, light brown to brown in color, and will exhibit high strength and low compressibility characteristics under the anticipated loading range.

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

4.3 GROUNDWATER

During excavating, groundwater was not encountered to depths of 16 feet. Groundwater is anticipated to be at depths greater than 16 feet below ground surface in the area. It has been our experience in the area that “perched” groundwater conditions can occur during the year, most typically during the late spring and early summer months. Discussions concerning “perched” groundwater are included within section 5.8, Foundation Subdrains.

Seasonal and longer-term groundwater fluctuations on the order of one to two feet should be anticipated with the highest levels occurring during the late spring and summer months.

5. DISCUSSIONS AND RECOMMENDATIONS

5.1 SUMMARY OF FINDINGS

The results of our analyses indicate that the proposed structures may be supported upon conventional spread and/or continuous wall foundations established upon suitable natural soils or granular structural fill extending to suitable natural soils. The most significant geotechnical aspects of the site are the moderate strength characteristics of the near surface sandy silt soils encountered at some areas of the site, the potential for "perched" groundwater conditions, and the stability of the slopes at the property.

Because of the possibility of "perched" groundwater conditions, perimeter foundation/chimney subdrains will be required along the perimeter of any proposed structures with subgrade levels.

The on-site soils may be re-utilized as structural site grading fill if they meet the requirements for such, as stated herein. It should be noted that from a handling and compaction standpoint, soils containing high amounts of fines (silts and clays) are very sensitive to changes in moisture content and will require very close moisture control during placement and compaction. This will be very difficult, if not impossible, during wet and cold periods of the year.

A qualified geotechnical engineer from GSH should verify that all topsoil and disturbed soils have been completely removed prior to the placement of structural site grading fills, floor slabs, footings, foundations, or rigid pavements.

In the following sections, detailed discussions pertaining to earthwork, foundations, at-grade concrete slabs, slope stability, and the geoseismic setting of the site are provided.

5.2 EARTHWORK

5.2.1 Site Preparation

Initial site preparation will consist of the removal of surface vegetation, topsoil, and other deleterious materials from beneath an area extending out at least 3 feet from the perimeter of the proposed building, pavements, and exterior flatwork areas.

Additional site preparation will consist of the removal of topsoil/disturbed soils from an area extending out at least 3 feet from the perimeter of all residential structures. Based on our observations at the test pit locations, the topsoil/disturbed soils are likely to extend about 2 to 6 inches in depth below existing site grades; however, variation in the depth of topsoil/disturbed soils is likely.

Subsequent to stripping and prior to the placement of structural site grading fill, pavements, driveway and garage slabs on grade, the prepared subgrade must be proofrolled by passing moderate-weight rubber tire-mounted construction equipment over the surface at least twice. If excessively soft or loose soils are encountered, they must be removed to a maximum depth of

2 feet and replaced with structural fill. Beneath footings, all loose and disturbed soils must be totally removed

Surface vegetation and other deleterious materials should generally be removed from the site. Topsoil, although unsuitable for utilization as structural fill, may be stockpiled for subsequent landscaping purposes.

5.2.2 Excavations

Temporary excavations up to 8 feet deep in fine-grained cohesive soils or bedrock, above or below the water table, may be constructed with sideslopes no steeper than one-half horizontal to one vertical (½H:1V). Excavations deeper than 8 feet are not anticipated at the site.

For granular (cohesionless) soils, construction excavations above the water table, not exceeding 4 feet, should be no steeper than one-half horizontal to one vertical (½H:1V). For excavations up to 8 feet, in granular soils and above the water table, the slopes should be no steeper than one horizontal to one vertical (1H:1V). Excavations encountering saturated cohesionless soils will be very difficult and will require very flat sideslopes and/or shoring, bracing and dewatering.

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.

5.2.3 Structural Fill

Structural fill will be required as site grading fill, as backfill over foundations and utilities, and possibly as replacement fill beneath some footings. All structural fill must be free of sod, rubbish, construction debris, 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. The maximum particle size within structural site grading fill should generally not exceed 4 inches; although, occasional particles up to 6 to 8 inches may be incorporated provided that they do not result in “honeycombing” or preclude the obtainment of the desired degree of compaction. In confined areas, the maximum particle size should generally be restricted to 2.5 inches.

On-site soils may be re-utilized as structural site grading fill if they do not contain construction debris or deleterious material and meet the requirements of structural fill. Fine-grained soils will require very close moisture control and may be very difficult, if not impossible, to properly place and compact during wet and cold periods of the year.

Only granular soils are recommended in confined areas such as utility trenches etc. Generally, we recommend that all imported granular structural fill consist of a well-graded mixture of sands and gravels with no more than 30 percent retaining on the 0.75-inch sieve and no more than 20 percent fines (material passing the No. 200 sieve). The plasticity index of import fine-grained soil should not exceed 18 percent.

To stabilize soft subgrade conditions (if encountered) or where structural fill is required to be placed closer than 1.0 foot above the water table at the time of construction, a mixture of coarse gravels and cobbles and/or 1.5- to 2.0-inch gravel (stabilizing fill) should be utilized. It may also help to utilize a stabilization fabric, such as Mirafi 600X or equivalent, placed on the native ground if 1.5- to 2.0-inch gravel is used as stabilizing fill.

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.

5.2.4 Fill Placement and Compaction

All structural fill shall be placed in lifts not exceeding 8 inches in loose thickness. Structural fills shall be compacted in accordance with the percent of the maximum dry density as determined by the ASTM[1] D-1557 (AASHTO[2] T-180) 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
Outside area defined above	0 to 5	90
Outside area defined above	5 to 8	95
Roadbase	---	96

Structural fills greater than 8 feet thick are not anticipated at the site.

Subsequent to stripping and prior to the placement of structural site grading fill, the subgrade should be prepared as discussed in Section 5.2.1, Site Preparation, of this report. In confined areas, subgrade preparation should consist of the removal of all loose or disturbed soils.

If utilized for stabilizing fill, coarse gravel and cobble mixtures should 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 at least twice. Subsequent fill material placed over the coarse gravels and cobbles should be adequately

[1] American Society for Testing and Materials
[2] American Association of State Highway and Transportation Officials

compacted so that the "fines" are "worked into" the voids in the underlying coarser gravels and cobbles.

5.2.5 Utility Trenches

All utility trench backfill material below structurally loaded facilities (flatwork, floor slabs, roads, etc.) shall 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 shall be proofrolled and/or properly compacted prior to the construction of any exterior flatwork over a backfilled trench. Proofrolling shall 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 shall be removed to a maximum depth of 2 feet below design finish grade and replaced with structural fill.

Most utility companies and City-County governments are now requiring that Type A-1a or A-1b (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.

5.3 SLOPE STABILTY

5.3.1 Stability Analyses

The properties of the natural sand soils and gravels observed at the test pit locations were estimated using laboratory testing, published correlations, and our experience with similar soils. Based on tests performed by the Bureau of Reclamation[1], silty sands have an internal friction angle ranging from 30 to 36 degrees and a cohesion varying from 180 to 420 pounds per square foot. Accordingly, we estimated the following parameters for use in the stability analyses:

Material	Internal Friction Angle (degrees)	Apparent Cohesion (psf)	Saturated Unit Weight (pcf)
On-Site Natural Sands	32	50	115

For the seismic (pseudostatic) analysis, a peak horizontal ground acceleration of 0.35*g* was obtained after adjusting for Site Class D (see Section 5.11.4) for site (grid) locations of 41.0329 degrees latitude (north) and 111.8996 degrees longitude (west). To model sustained accelerations at the site, one-half of this value is typically employed. Accordingly, a value of 0.18 was used as the pseudostatic coefficient for the stability analysis.

[1] U.S. Bureau of Reclamation, 1987, "Design Standards No. 13, Embankment Dams," Denver, Colorado.

We evaluated the global stability of the slope using the computer program *SLIDE*. This program uses a limit equilibrium (Simplified Bishop) method for calculating factors of safety against sliding on an assumed failure surface and evaluates numerous potential failure surfaces, with the most critical failure surface identified as the one yielding the lowest factor of safety of those evaluated. Two slope configurations were analyzed. The first configuration we analyzed consisted of a 70-foot high slope inclined at approximately 1V:2H (Vertical:Horizontal) grading to 10-foot high slope inclined at approximately 1V:3H followed by a building pad created for the home site on Lot 39. To simulate the load imposed on the slope by the proposed residential construction, a load of 1,500 psf was modeled at the building pad location. The second configuration analyzed was on Lot 13 and consisted of a relatively level home pad grading to 40-foot high slope inclined at about 1V:4H followed by a relatively flat 10-foot wide trail followed by a 200-foot tall slope inclined at about 1V:3.75H grading to a relatively flat 20-foot wide trail. Typically, the required minimum factors of safety are 1.5 for static conditions and 1.0 for seismic (pseudostatic) conditions. The results of our analyses indicate that the existing slope configurations will meet both these requirements provided our recommendations are followed. The slope stability data are included as Figures 5 through 8, attached.

Changes to the grading at the site and any retaining walls must be properly engineered to maintain stability of the slopes. GSH should be retained to review the final grading plans for the project prior to construction.

5.4 SPREAD AND CONTINUOUS WALL FOUNDATIONS

5.4.1 Design Data

The proposed structure may be supported upon conventional spread and continuous wall foundations established upon suitable natural soils/bedrock and/or structural fill extending to suitable natural soils/bedrock. For design, the following parameters are provided:

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	- 16 inches
Minimum Recommended Width for Isolated Spread Footings	- 24 inches

Recommended Net Bearing Pressure for Real Load Conditions	- 2,500 pounds per square foot
Bearing Pressure Increase for Seismic Loading	- 50 percent

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.

5.4.2 Installation

Under no circumstances shall the footings be established upon non-engineered fills, loose or disturbed soils, topsoil, sod, 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. For instance if the footing width is 2 feet and the structural fill depth beneath the footing is 2 feet, the fill replacement width should be 4 feet, centered beneath the footing.

5.4.3 Settlements

Maximum settlements of foundations designed and installed in accordance with recommendations presented herein and supporting maximum anticipated loads as discussed in Section 2, Proposed Construction, are anticipated to be 1 inch or less.

Approximately 60 percent of the quoted settlement should occur during construction.

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. For estimated frictional resistance, a coefficient of friction of 0.35 should be utilized for foundations established on native soils. 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, are for backfills which will consist of drained granular soil placed and compacted in accordance with the recommendations presented herein. 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 walls (moderately yielding), generally not exceeding 8 feet in height, granular backfill may be considered equivalent to a fluid with a density of 45 pounds per cubic foot. The above values assume that the surface of the soils slope behind the wall is no steeper than 4 horizontal to 1 vertical and that the granular fill within 3 feet of the wall will be compacted with hand-operated compacting equipment.

For seismic loading, a uniform pressure should be added. The uniform pressures based on different wall heights are provided in the following table:

Wall Height (feet)	Seismic Loading Active Case (psf)	Seismic Loading Moderately Yielding (psf)
4	25	55
6	40	85
8	55	115

5.7 FLOOR SLABS

Lightly loaded at-grade floor slabs may be established upon suitable natural soils and/or upon structural fill extending to suitable natural soils. Under no circumstance should floor slabs be established upon loose or disturbed soils, sod, non-engineered fill, deleterious materials, expansive soils, frozen soils, or within ponded water. To facilitate construction and to provide a capillary moisture break, we recommend that at-grade slabs be immediately underlain by a minimum of 4 inches of “free-draining” granular material, such as “pea” gravel or three-quarter- to one-inch minus clean gap-graded gravel.

Settlement of lightly loaded floor slabs (average uniform pressure of 200 pounds per square foot or less) is anticipated to be less than 1/2 inch.

5.8 SUBDRAINS

5.8.1 General

Groundwater was not encountered at the site, however it is anticipated that “perched watertable” conditions may develop within the subsurface soils during spring and summer months, therefore

we recommend perimeter foundation subdrains around footings for structures with subgrade levels. In road cut areas, subdrains may also be required. We recommend that the perimeter foundation subdrains be installed as indicated below.

5.8.2 Foundation Subdrains

Foundation subdrains should consist of a 4-inch diameter perforated or slotted plastic or PVC pipe enclosed in clean gravel. The invert of a subdrain should be at least 2 feet below the top of the lowest adjacent floor slab. The gravel portion of the drain should extend 2 inches laterally and below the perforated pipe and at least 1 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 4-inch-wide zone of "free-draining" sand/gravel should be placed adjacent to the foundation walls and extend to within 2 feet of final grade. The upper 2 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/gravel, 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 drain pipe should be clean 0.75-inch to 1.0-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

5.9.1 Design Criteria

It is projected that proposed roadways will consist of primarily asphalt concrete. The existing natural sand soils encountered at the site will exhibit moderate pavement support characteristics when saturated or near saturated.

All pavement areas must be prepared as previously discussed (see Section 5.2.1, Site Preparation). With the subgrade soils and the projected traffic as discussed in Section 2, Proposed Construction, the pavement sections on the following pages are recommended.

Minor Streets/Cul-de-Sac Traffic
(Light to Moderate Volume of Automobiles and Light Trucks,
Light Volume of Medium-Weight Trucks,
and occasional Heavy-Weight Trucks)
[4 equivalent 18-kip axle loads per day]

Flexible Pavement:

3.0 inches	Asphalt concrete
8.0 inches	Aggregate base
Over	Suitable natural soils, properly prepared soils, and/or structural site grading fill extending to properly prepared/suitable natural soils.

Or

3.0 inches	Asphalt concrete
5.0 inches	Aggregate base
5.0 inches	Sub base
Over	Suitable natural soils, properly prepared soils, and/or structural site grading fill extending to properly prepared/suitable natural soils.

5.10 CEMENT TYPES

The laboratory tests indicate that the natural soils tested contain a negligible amount of water soluble sulfates. Based on our test results, concrete in contact with the on-site soil will have a low potential for sulfate reaction (ACI 318, Table 4.3.1). Therefore, all concrete which will be in contact with the site soils may be prepared using Type I or IA cement.

5.11 GEOSEISMIC SETTING

5.11.1 General

Utah municipalities adopted the International Building Code (IBC) 2012 and International Residential Code (IRC) for One- to Two-Family Dwellings 2012 on July 1, 2013. The IBC and IRC 2012 codes determine 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).

5.11.2 Site Class

For dynamic structural analysis, the Site Class D – Stiff Soil Profile as defined in Chapter 20 of ASCE 7 (per Section 1613.3.2, Site Class Definitions, of IBC 2012) can be utilized.

5.11.3 Faulting

Based upon our review of available literature, the Wasatch Fault passes along the eastern edge of the proposed development. Due to the proximity of the proposed development to the Wasatch Fault, a site specific geologic hazards study including a surface fault rupture study is recommended. Such a study is beyond the scope of work associated with our geotechnical study.

5.11.4 Ground Motions

The IBC 2012 code is based on 2008 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 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 D soil profile in the second column. Based on the site latitude and longitude (40.5718 degrees north and -111.4241 degrees west, respectively), the values for this site are tabulated below.

Spectral Acceleration Value, T	Site Class B Boundary [mapped values] (% g)	Site Coefficient	Site Class D [adjusted for site class effects] (% g)	Design Values (% g)
Peak Ground Acceleration	51.7	F_a = 1.000	51.7	34.5
0.2 Seconds (Short Period Acceleration)	S_S = 129.3	F_a = 1.000	S_{MS} = 129.3	S_{DS} = 86.2
1.0 Second (Long Period Acceleration)	S_1 = 50.2	F_v = 1.500	S_{M1} = 75.3	S_{D1} = 50.2

5.11.5 Liquefaction

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.

Liquefaction of near-surface soils at the site is not anticipated during the design seismic event due to the unsaturated nature of the site soils. However the liquefaction susceptibility of the underlying soils (deeper than our explorations) is unknown and would require further investigation to quantify.

5.12 SITE VISITS

As stated previously, prior to placement of foundations, floor slabs, pavements, and site grading fills, a geotechnical engineer from GSH must verify that all topsoil and weathered/disturbed soils have been removed and suitable subgrade conditions encountered.

If you have any questions or would like to discuss these items further, please feel free to contact us at (801) 393-2012.

Respectfully submitted,

GSH Geotechnical, Inc.



Andrew M. Harris, P.E.
State of Utah No. 7420456
Senior Geotechnical Engineer

Reviewed by:

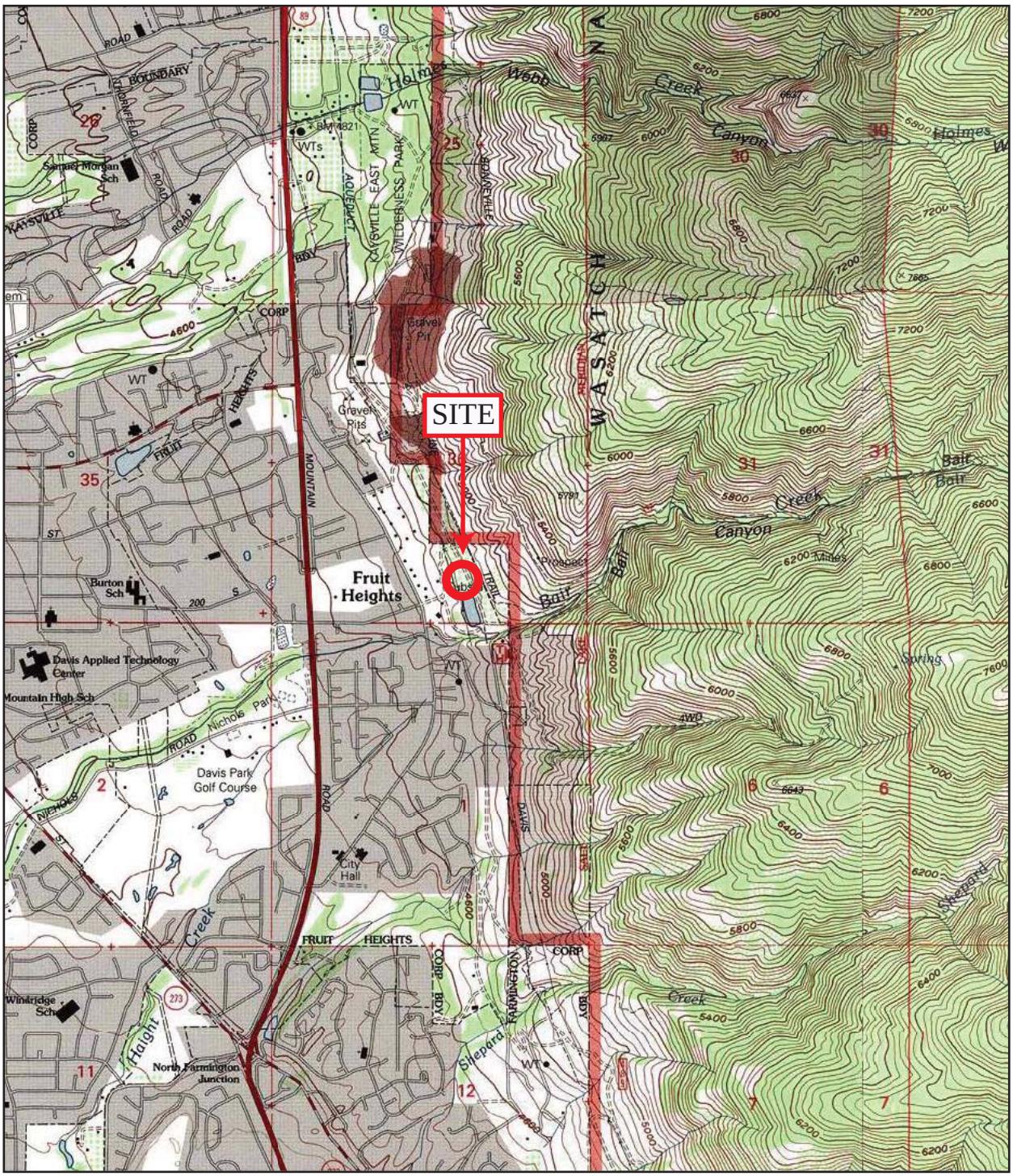
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Senior Geotechnical Engineer

AMH/WGT:mmh

Encl. Figure 1, Vicinity Map
Figure 2, Site Plan
Figures 3A through 3O, Test Pit Logs
Figure 4, Key to Test Pit Log (USCS)
Figures 5, through 8, Stability Results

Addressee (email)

LOWE COMPANIES
JOB NO. 1696-001-14



SCALE IN FEET
1000 0 1000 2000

FIGURE 1
VICINITY MAP

REFERENCE:
USGS 7.5 MINUTE TOPOGRAPHIC QUADRANGLE MAP(S)
ENTITLED "KAYSVILLE, UTAH" AND "PETERSON, UTAH"
BOTH DATED 1998

GSH

LOWE COMPANIES
JOB NO. 1696-001-14



FIGURE 2
SITE PLAN

REFERENCE:
ADAPTED FROM DRAWING ENTITLED SKETCH PLAN
BY REEVE & ASSOCIATES, INC., DATED APRIL 24, 2014

APPROXIMATE SCALE IN FEET
100 0 100 200

GSH

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-1

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-001-14

PROJECT: Proposed Raymond Property Residential Development | DATE STARTED: 8/19/14 | DATE FINISHED: 8/19/14

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: JCB 214S - Backhoe | HAMMER: --- | WEIGHT: --- | DROP: ---

GROUNDWATER DEPTH: Not Encountered (8/19/14) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	SM	SILTY FINE SAND with occasional fine gravel; major roots (topsoil) to 3"; trace organics; brown								moist medium dense
		grades with some fine and coarse gravel; occasional cobbles; trace organics; brown	5							slightly moist
		grades silty fine and coarse gravel with fine to coarse sand; occasional cobbles; brown								
		grades with numerous cobbles and occasional boulders	10							
	GP	FINE AND COARSE GRAVEL with fine to coarse sand; brown			1		4			slightly moist dense
		End of Exploration at 13.0' No groundwater encountered at time of excavation	15							
			20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3A

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-2

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-001-14

PROJECT: Proposed Raymond Property Residential Development | DATE STARTED: 8/19/14 | DATE FINISHED: 8/19/14

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: JCB 214S - Backhoe | HAMMER: --- | WEIGHT: --- | DROP: ---

GROUNDWATER DEPTH: Not Encountered (8/19/14) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	SP	FINE SAND with occasional fine gravel; major roots (topsoil) to 4"; brown								slightly moist medium dense
		grades with some fine and coarse gravel; brown			2		6			
		occasional cobbles	5							
		numerous cobbles and occasional boulders	10							dense
		grades fine and coarse gravel with fine to coarse sand; trace to some silt; occasional cobbles								
		End of Exploration at 14.0' No groundwater encountered at time of excavation	15							
			20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3B

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-3

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-001-14

PROJECT: Proposed Raymond Property Residential Development | DATE STARTED: 8/19/14 | DATE FINISHED: 8/19/14

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: JCB 214S - Backhoe | HAMMER: --- | WEIGHT: --- | DROP: ---

GROUNDWATER DEPTH: Not Encountered (8/19/14) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	SP/ SM	SILTY FINE TO COARSE SAND with fine and coarse gravel; major roots (topsoil) to 4"; brown								slightly moist loose
					2		10			
			5							medium dense
			10		2		13			moist
			15							
		End of Exploration at 16.0' No groundwater encountered at time of excavation	20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3C

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-4

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-001-14

PROJECT: Proposed Raymond Property Residential Development | DATE STARTED: 8/19/14 | DATE FINISHED: 8/19/14

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: JCB 214S - Backhoe | HAMMER: --- | WEIGHT: --- | DROP: ---

GROUNDWATER DEPTH: Not Encountered (8/19/14) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	SM	SILTY FINE TO COARSE SAND with occasional fine gravel; major roots (topsoil) to 3"; brown								slightly moist loose
					4	104				medium dense
			5							
		grades fine to coarse sand with occasional fine gravel; brown			4		13			moist
			10							
		End of Exploration at 15.0' No groundwater encountered at time of excavation	15							
			20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3D

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-5

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-001-14

PROJECT: Proposed Raymond Property Residential Development | DATE STARTED: 8/19/14 | DATE FINISHED: 8/19/14

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: JCB 214S - Backhoe | HAMMER: --- | WEIGHT: --- | DROP: ---

GROUNDWATER DEPTH: Not Encountered (8/19/14) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	SM	SILTY FINE TO COARSE SAND with some fine and coarse gravel; major roots (topsoil) to 5"; brown								slightly moist medium dense
		grades with occasional to numerous cobbles 4" to 12" in diameter; occasional boulders up to 2.5' in diameter	5							
	GM	SILTY FINE AND COARSE GRAVEL with fine to coarse sand; brown								slightly moist medium dense
	ML	FINE SANDY SILT light brown	10		6		55			moist medium dense
	SM	SILTY FINE SAND light brown								moist loose
			15		10		33			medium dense
		End of Exploration at 16.0' No groundwater encountered at time of excavation Installed 1.25" diameter slotted PVC pipe to 16.0'	20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3E

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-6

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-001-14

PROJECT: Proposed Raymond Property Residential Development | DATE STARTED: 8/19/14 | DATE FINISHED: 8/19/14

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: JCB 214S - Backhoe | HAMMER: --- | WEIGHT: --- | DROP: ---

GROUNDWATER DEPTH: Not Encountered (8/19/14) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	SM/ GM	SILTY FINE TO COARSE SAND with fine and coarse gravel; major roots (topsoil) to 4"; brown								slightly moist medium dense
			5							moist loose
		grades some silt	10		5		13			loose to medium dense
	SP/ SM	FINE TO COARSE SAND with occasional fine and coarse gravel; brown			3		7			moist medium dense
		End of Exploration at 15.0' No groundwater encountered at time of excavation	15							
			20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3F

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-7

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-001-14

PROJECT: Proposed Raymond Property Residential Development | DATE STARTED: 8/19/14 | DATE FINISHED: 8/19/14

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: JCB 214S - Backhoe | HAMMER: --- | WEIGHT: --- | DROP: ---

GROUNDWATER DEPTH: Not Encountered (8/19/14) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	SM	SILTY FINE TO COARSE SAND with some fine and coarse gravel; major roots (topsoil) to 5"; trace organics; brown								slightly moist medium dense
		occasional cobbles and boulders	5							
					2		16			moist loose
			10							
										medium dense
		End of Exploration at 15.0' No groundwater encountered at time of excavation	15							
			20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3G

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-8

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-001-14

PROJECT: Proposed Raymond Property Residential Development | DATE STARTED: 8/20/14 | DATE FINISHED: 8/20/14

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: JCB 214S - Backhoe | HAMMER: --- | WEIGHT: --- | DROP: ---

GROUNDWATER DEPTH: Not Encountered (8/20/14) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	SM/ GM	SILTY FINE TO MEDIUM SAND with some fine and coarse gravel; major roots (topsoil) to 2"+; trace organics; light brown to brown								slightly moist medium dense
		grades with occasional cobbles			2		15			
			5		6	88				loose
					3		31			loose to medium dense
			10							
	SM	SILTY FINE SAND light brown								moist loose
			15							
		End of Exploration at 16.0' No groundwater encountered at time of excavation								
			20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3H

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-9

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-001-14

PROJECT: Proposed Raymond Property Residential Development | DATE STARTED: 8/20/14 | DATE FINISHED: 8/20/14

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: JCB 214S - Backhoe | HAMMER: --- | WEIGHT: --- | DROP: ---

GROUNDWATER DEPTH: Not Encountered (8/20/14) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	SM/ GM	SILTY FINE SAND with occasional fine and coarse gravel; major roots (topsoil) to 2"; trace organics; brown								slightly moist medium dense
		occasional boulders up to 2' in diameter	5							
		grades with occasional cobbles; some fine and coarse gravel								
	ML	FINE SANDY SILT light brown	10							slightly moist medium dense
		grades with occasional layers of fine sandy silt up to 1/4" thick; high oxidation; light brown								
					18		69			moist
		End of Exploration at 15.5' No groundwater encountered at time of excavation Installed 1.25" diameter slotted PVC pipe to 15.5'	15							
			20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3I

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-10

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-001-14

PROJECT: Proposed Raymond Property Residential Development | DATE STARTED: 8/20/14 | DATE FINISHED: 8/20/14

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: JCB 214S - Backhoe | HAMMER: --- | WEIGHT: --- | DROP: ---

GROUNDWATER DEPTH: Not Encountered (8/20/14) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	SP/ SM	FINE TO COARSE SAND with fine and coarse gravel; major roots (topsoil) to 3"; rootholes; trace organics; brown								slightly moist medium dense
			5							
		grades with some silt and occasional cobbles	10		3		7			moist
			15							
		End of Exploration at 16.0' No groundwater encountered at time of excavation	20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3J

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-11

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-001-14

PROJECT: Proposed Raymond Property Residential Development | DATE STARTED: 8/20/14 | DATE FINISHED: 8/20/14

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: JCB 214S - Backhoe | HAMMER: --- | WEIGHT: --- | DROP: ---

GROUNDWATER DEPTH: Not Encountered (8/20/14) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	SM	SILTY FINE TO COARSE SAND with fine and coarse gravel; major roots (topsoil) to 3"; brown								slightly moist medium dense
					2		14			
		occasional cobbles and boulders	5							
			10							
		grades with some silt			3		11			moist
			15							
		End of Exploration at 16.0' No groundwater encountered at time of excavation	20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3K

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-12

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-001-14

PROJECT: Proposed Raymond Property Residential Development | DATE STARTED: 8/20/14 | DATE FINISHED: 8/20/14

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: JCB 214S - Backhoe | HAMMER: --- | WEIGHT: --- | DROP: ---

GROUNDWATER DEPTH: Not Encountered (8/20/14) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	SP/ SM	SILTY FINE AND COARSE GRAVEL with fine to coarse sand; major roots (topsoil) to 4"; trace organics; brown								moist medium dense
					1		9			
		occasional cobbles	5							
			10							
			15							
		End of Exploration at 16.0' No groundwater encountered at time of excavation	20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3L

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-13

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-001-14

PROJECT: Proposed Raymond Property Residential Development | DATE STARTED: 8/20/14 | DATE FINISHED: 8/20/14

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: JCB 214S - Backhoe | HAMMER: --- | WEIGHT: --- | DROP: ---

GROUNDWATER DEPTH: Not Encountered (8/20/14) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	GM/SM	SILTY FINE AND COARSE GRAVEL with fine to coarse sand; major roots (topsoil) to 3"; trace organics; brown								slightly moist medium dense
					2		27			
			5							
		grades with occasional cobbles								moist
			10							
		grades fine and coarse gravel with fine to coarse sand; some silt; brown								
			15							
		End of Exploration at 16.0' No groundwater encountered at time of excavation Installed 1.25" diameter slotted PVC pipe to 15.5'								
			20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3M

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-14

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-001-14

PROJECT: Proposed Raymond Property Residential Development | DATE STARTED: 8/20/14 | DATE FINISHED: 8/20/14

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: JCB 214S - Backhoe | HAMMER: --- | WEIGHT: --- | DROP: ---

GROUNDWATER DEPTH: Not Encountered (8/20/14) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	GM	SILTY FINE AND COARSE GRAVEL with some fine to coarse sand; major roots (topsoil) to 3"; trace organics; brown								slightly moist medium dense
	SP	FINE TO COARSE SAND with some fine gravel; trace to some silt	5		1		6			slightly moist medium dense to loose
	SP	FINE SAND with some silt; occasional layers of fine sandy silt up to 1/2" thick; light brown	10							moist loose
		End of Exploration at 15.0' No groundwater encountered at time of excavation	15							
			20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3N

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-15

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-001-14

PROJECT: Proposed Raymond Property Residential Development | DATE STARTED: 8/20/14 | DATE FINISHED: 8/20/14

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: JCB 214S - Backhoe | HAMMER: --- | WEIGHT: --- | DROP: ---

GROUNDWATER DEPTH: Not Encountered (8/20/14) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	SM	SILTY FINE SAND occasional fine and coarse gravel; major roots (topsoil) to 3"; dark brown								moist loose
		grades brown	5							
			10		6		31			
			15							
		End of Exploration at 16.0' No groundwater encountered at time of excavation Installed 1.25" diameter slotted PVC pipe to 15.0'	20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3O

CLIENT: Lowe Companies
PROJECT: Proposed Raymond Property Residential Development
PROJECT NUMBER: 1696-001-14

KEY TO TEST PIT LOG

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
①	②	③	④	⑤	⑥	⑦	⑧	⑨	⑩	⑪

COLUMN DESCRIPTIONS

① **Water Level:** Depth to measured groundwater table. See symbol below.

② **USCS:** (Unified Soil Classification System) Description of soils encountered; typical symbols are explained below.

③ **Description:** Description of material encountered; may include color, moisture, grain size, density/consistency,

④ **Depth (ft.):** Depth in feet below the ground surface.

⑤ **Sample Symbol:** Type of soil sample collected at depth interval shown; sampler symbols are explained below.

⑥ **Moisture (%):** Water content of soil sample measured in laboratory; expressed as percentage of dryweight of

⑦ **Dry Density (pcf):** The density of a soil measured in laboratory; expressed in pounds per cubic foot.

⑧ **% Passing 200:** Fines content of soils sample passing a No. 200 sieve; expressed as a percentage.

⑨ **Liquid Limit (%):** Water content at which a soil changes from plastic to liquid behavior.

⑩ **Plasticity Index (%):** Range of water content at which a soil exhibits plastic properties.

⑪ **Remarks:** Comments and observations regarding drilling or sampling made by driller or field personnel. May include other field and laboratory test results using the following abbreviations:

CEMENTATION:	MODIFIERS:	MOISTURE CONTENT (FIELD TEST):
Weakly: Crumbles or breaks with handling or slight finger pressure.	**Trace** <5%	**Dry:** Absence of moisture, dusty, dry to the touch.
Moderately: Crumbles or breaks with considerable finger pressure.	**Some** 5-12%	**Moist:** Damp but no visible water.
Strongly: Will not crumble or break with finger pressure.	**With** > 12%	**Saturated:** Visible water, usually soil below water table.

Descriptions and stratum lines are interpretive; field descriptions may have been modified to reflect lab test results. Descriptions on the logs apply only at the specific boring locations and at the time the borings were advanced; they are not warranted to be representative of subsurface conditions at other locations or times.

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

MAJOR DIVISIONS			USCS SYMBOLS	TYPICAL DESCRIPTIONS
COARSE-GRAINED SOILS More than 50% of material is larger than No. 200 sieve size.	**GRAVELS** More than 50% of coarse fraction retained on No. 4 sieve.	**CLEAN GRAVELS** (little or no fines)	GW	Well-Graded Gravels, Gravel-Sand Mixtures, Little or No Fines
			GP	Poorly-Graded Gravels, Gravel-Sand Mixtures, Little or No Fines
		GRAVELS WITH FINES (appreciable amount of fines)	GM	Silty Gravels, Gravel-Sand-Silt Mixtures
			GC	Clayey Gravels, Gravel-Sand-Clay Mixtures
	SANDS More than 50% of coarse fraction passing through No. 4 sieve.	**CLEAN SANDS** (little or no fines)	SW	Well-Graded Sands, Gravelly Sands, Little or No Fines
			SP	Poorly-Graded Sands, Gravelly Sands, Little or No Fines
		SANDS WITH FINES (appreciable amount of fines)	SM	Silty Sands, Sand-Silt Mixtures
			SC	Clayey Sands, Sand-Clay Mixtures
FINE-GRAINED SOILS More than 50% of material is smaller than No. 200 sieve size.	**SILTS AND CLAYS** Liquid Limit less than 50%		ML	Inorganic Silts and Very Fine Sands, Rock Flour, Silty or Clayey Fine Sands or Clayey Silts with Slight Plasticity
			CL	Inorganic Clays of Low to Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean Clays
			OL	Organic Silts and Organic Silty Clays o f Low Plasticity
	SILTS AND CLAYS Liquid Limit greater than 50%		MH	Inorganic Silts, Micacious or Diatomacious Fine Sand or Silty Soils
			CH	Inorganic Clays of High Plasticity, Fat Clays
			OH	Organic Silts and Organic Clays of Medium to High Plasticity
HIGHLY ORGANIC SOILS			PT	Peat, Humus, Swamp Soils with High Organic Contents

Note: Dual Symbols are used to indicate borderline soil classifications.

STRATIFICATION:

DESCRIPTION	THICKNESS
Seam	up to 1/8"
Layer	1/8" to 12"
Occasional: One or less per 6" of thickness	
Numerous; More than one per 6" of thickness	

TYPICAL SAMPLER GRAPHIC SYMBOLS

- Bulk/Bag Sample
- Standard Penetration Split Spoon Sampler
- Rock Core
- No Recovery
- 3.25" OD, 2.42" ID D&M Sampler
- 3.0" OD, 2.42" ID D&M Sampler
- California Sampler
- Thin Wall

WATER SYMBOL

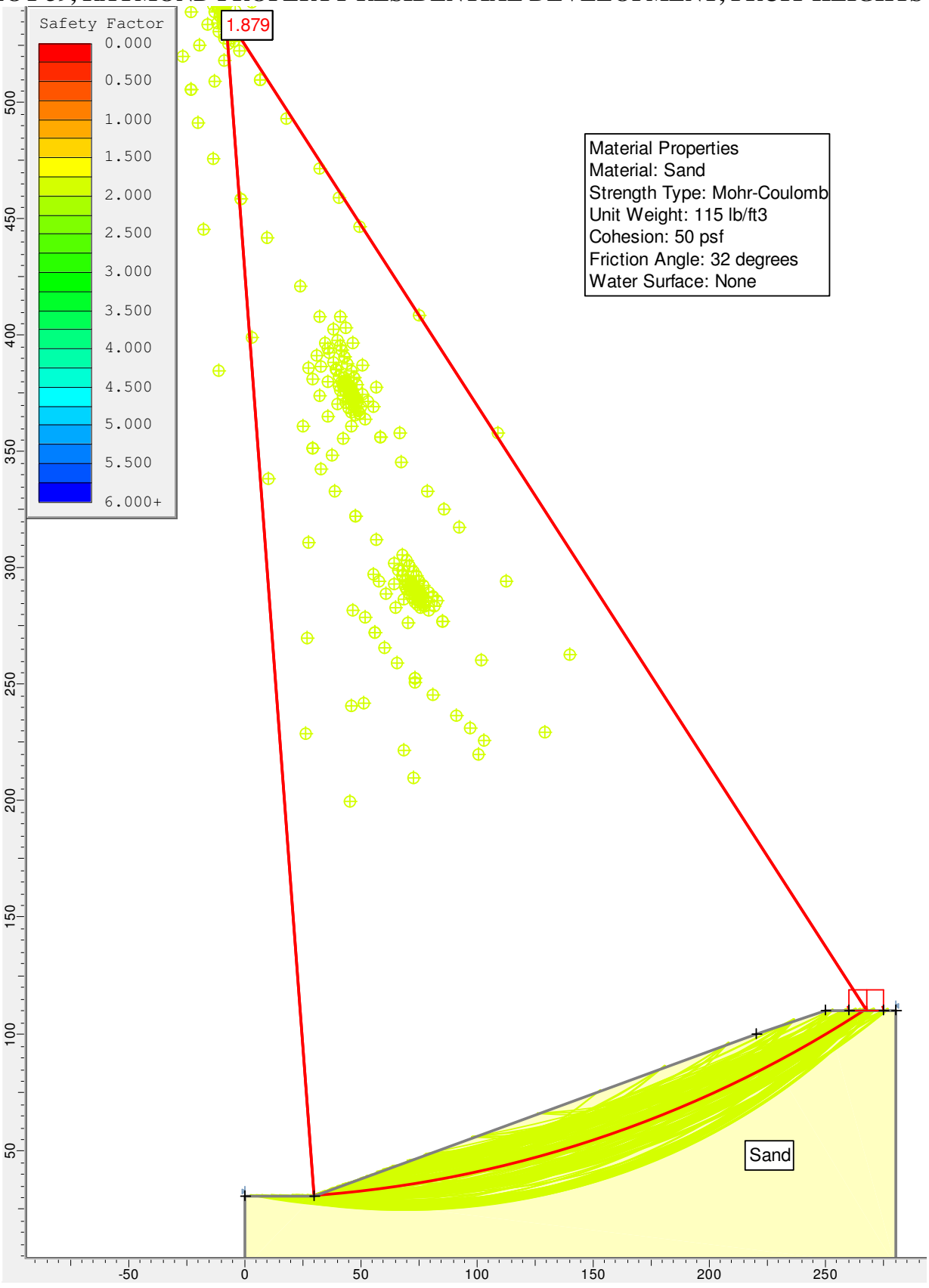
- Water Level

FIGURE 4

GSH

STABILITY RESULTS

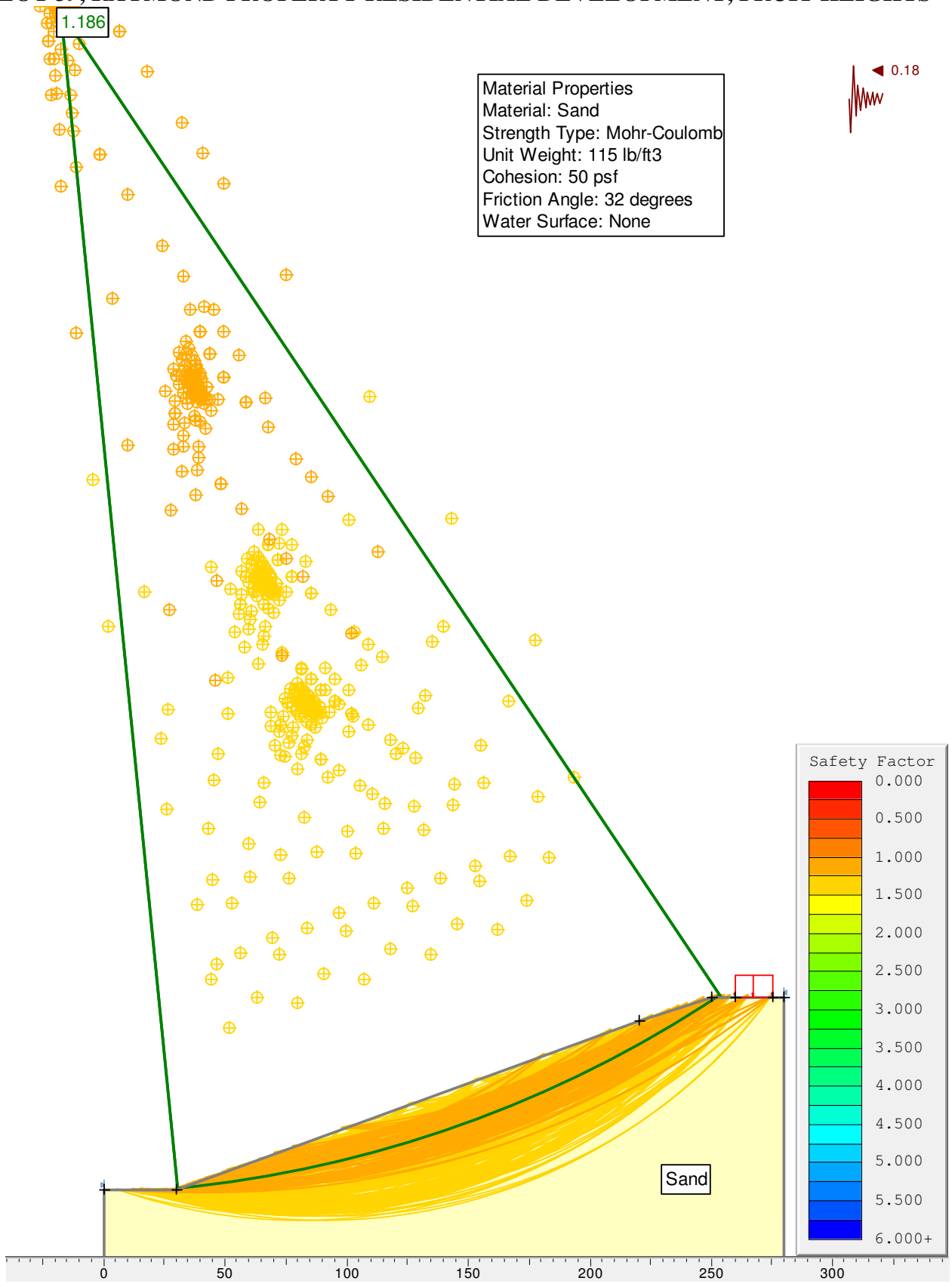
LOT 39, RAYMOND PROPERTY RESIDENTIAL DEVELOPMENT, FRUIT HEIGHTS



PROJECT NO.: 1696-001-14	GSH	**FIGURE NO.:** 5

STABILITY RESULTS

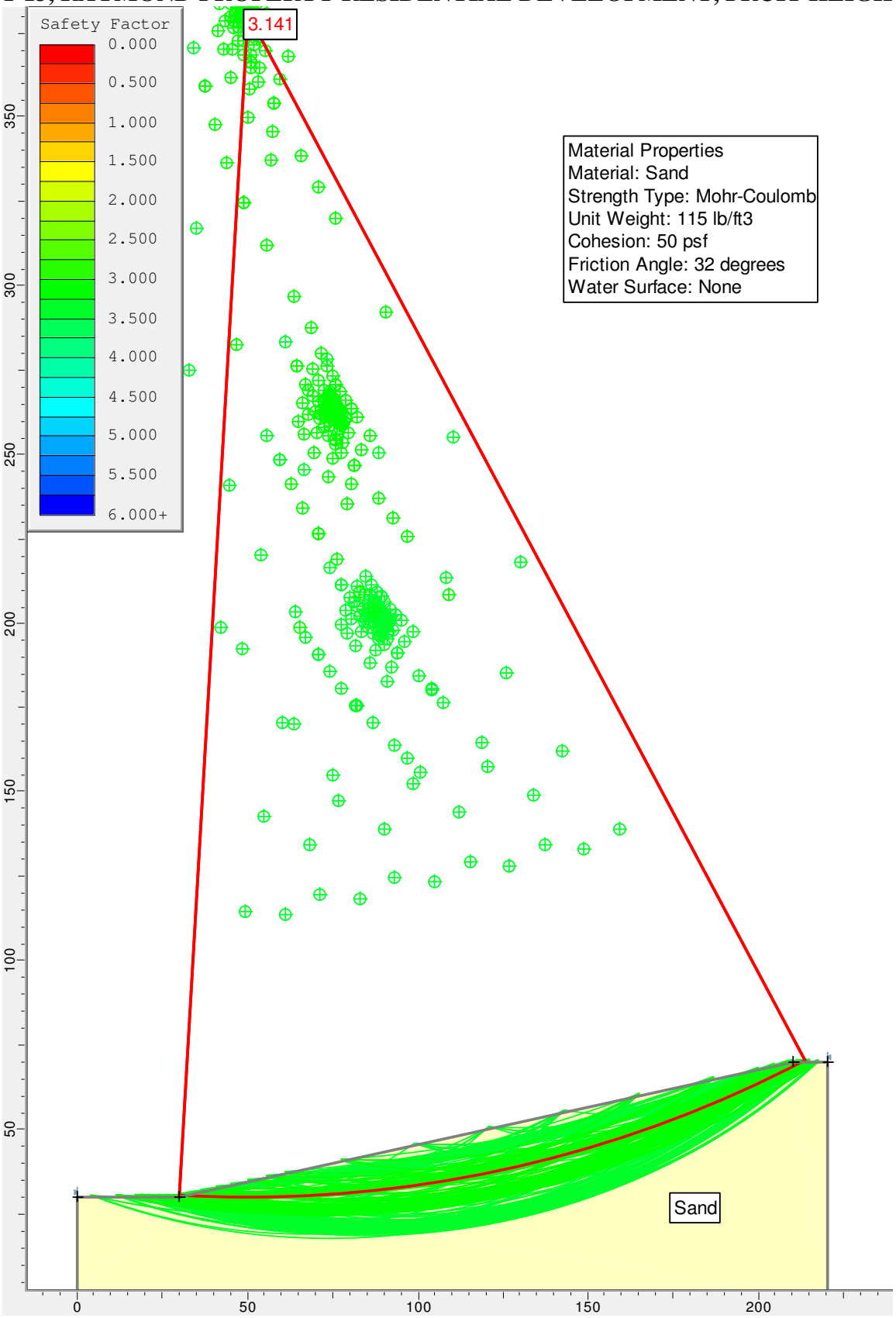
LOT 39, RAYMOND PROPERTY RESIDENTIAL DEVELOPMENT, FRUIT HEIGHTS



PROJECT NO.: 1696-001-14	GSH	**FIGURE NO.:** 6

STABILITY RESULTS

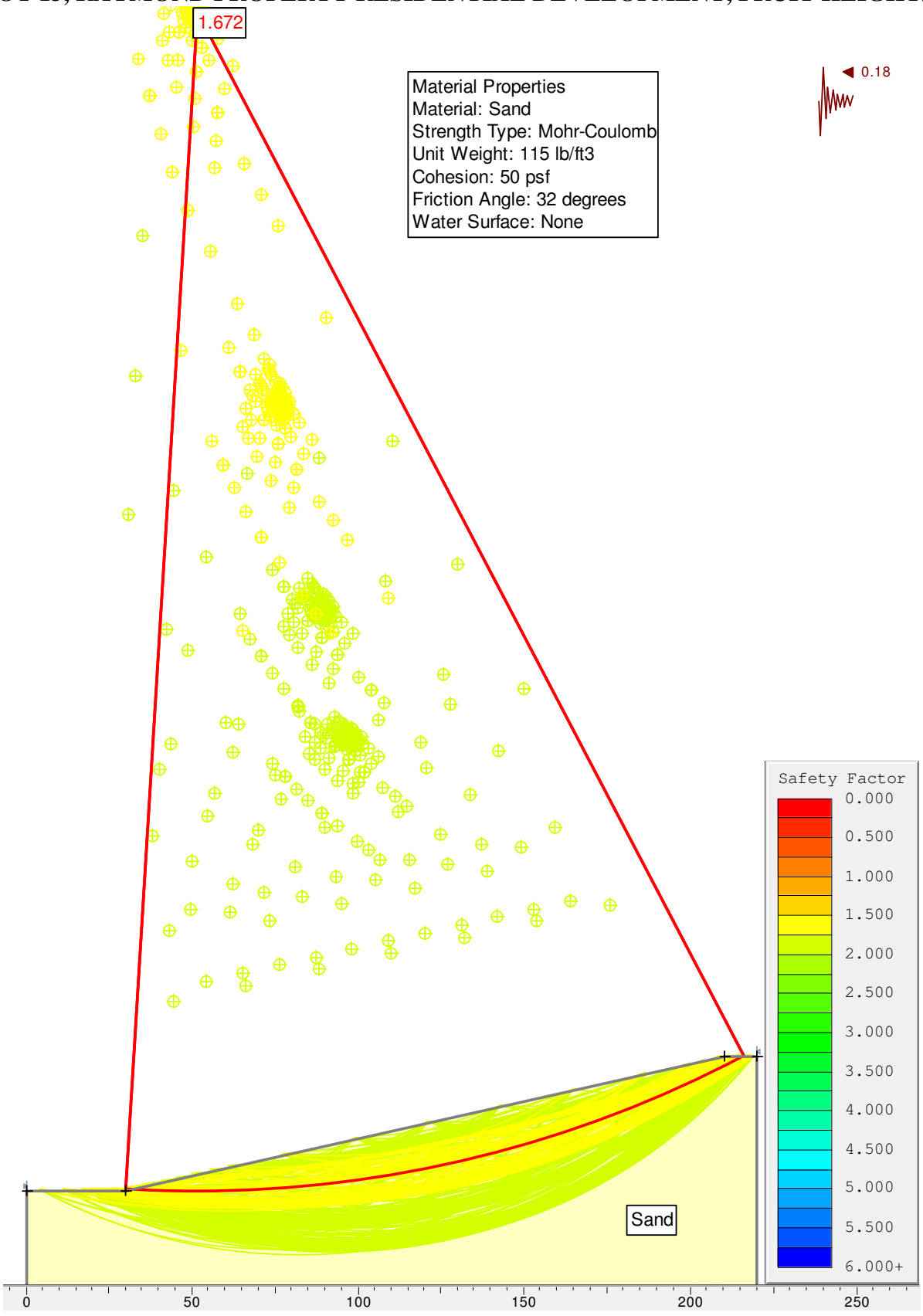
LOT 13, RAYMOND PROPERTY RESIDENTIAL DEVELOPMENT, FRUIT HEIGHTS



PROJECT NO.: 1696-001-14	GSH	FIGURE NO.: 7

STABILITY RESULTS

LOT 13, RAYMOND PROPERTY RESIDENTIAL DEVELOPMENT, FRUIT HEIGHTS



PROJECT NO.: 1696-001-14	GSH	**FIGURE NO.:** 8

June 30, 2015
Job No. 1696-02N-15

Mr. Matt Lowe
Lowe Companies
6028 South Ridge Drive
South Ogden, Utah 84403

Mr. Lowe:

Re: Work Plan
Supplemental Geotechnical Study
Proposed Raymond Property Residential Development
East of the Intersection of Mountain Road and Green Road
Fruit Heights, Utah

As requested by Mr. Matt Lowe of Low Companies, GSH Geotechnical, Inc. (GSH) intends to provide a supplemental geotechnical study for the proposed Raymond Property Residential Development in Fruit Heights, Utah. A geotechnical study[1] was previously prepared for the site by GSH, dated September 24, 2014.

Based on initial review comments from Geostrata, we understand that additional information related to slope stability and geologic hazards at the site are now required. Further subsurface investigation and geotechnical analyses related to slope stability at the site will be provided by GSH and the associated investigative approach is discussed below. Geologic hazards for the property will be evaluated by Western Geologic and the associated investigative approach is discussed in the attached Work Plan Geologic Hazards Evaluation by Western Geologic, dated June 24, 2015.

Scope of Work

In order to provide additional information for the subject property, an additional subsurface investigation is proposed for the site. The field investigation for the referenced geotechnical study for the subject property utilized relatively shallow test pits excavated across the site to depths of about 13.0 to 16.0 feet below existing grades. Supplemental information related to the subsurface soils will be gathered through geotechnical soil borings at 3 locations across the site (see Figure 1, attached). The depth of soil borings are proposed to extend to about 40 to 60 feet (or auger refusal) at boring B-1, about 40 to 75 feet (or auger refusal) at boring B-2, and about 30 to 50 feet (or auger refusal) at boring B-3. The soil borings will be drilled with a truck-mounted

[1] "Report Geotechnical Study, Proposed Raymond Property Residential Development, East of the Intersection of Mountain Road and Green Road, Fruit Heights, Utah," GSH Geotechnical, Inc., GSH Job No. 1696-001-15, September 24, 2014.

geotechnical drill rig equipped with hollow stem augers. A 3.25-inch outside diameter, 2.42-inch inside diameter drive sampler (Dames & Moore), a 3.0-inch outside diameter, 2.42-inch inside diameter drive sampler (Dames & Moore), and a 2.0-inch outside diameter, 1.38-inch inside diameter drive sampler (SPT) will be utilized for subsurface sampling at select locations. Following completion of drilling operations, one and one-quarter-inch diameter slotted PVC pipe will be installed in borings in order to provide a means of monitoring potential groundwater fluctuations. The borings will be backfilled with auger cuttings.

Following the field investigation, a laboratory testing program will be performed to determine parameters necessary for our engineering analyses. Laboratory tests may include, but are not limited to, moisture content, density, partial and full gradation, direct shear, residual direct shear, ring shear, and vane shear tests.

The information gathered from the field and laboratory testing will be utilized for our engineering analyses of the slope stability. The global stability of the slope will be evaluated using the computer program *SLIDE*. The cross-sections analyzed will be based on topographic survey data and proposed grading plans provided by the project civil engineer (Reeve and Associates) in conjunction with cross-section information obtained during the geologic hazards evaluation by Western Geologic.

A supplemental geotechnical study letter report will be issued at the completion of the project. The letter report will contain a description of the field investigation, subsurface conditions encountered, laboratory testing, engineering analyses, geoseismic setting and recommendations related to slope stability at the site. Attachments to this letter report will include a site plan showing the location of the soil borings completed at the site, a site plan with topographic information depicting the location of the slope cross-sections analyzed, slope stability analysis results, and boring logs.

Closure

If you have any questions or would like to discuss these items further, please feel free to contact us at (801) 393-2012.

Respectfully submitted,

GSH Geotechnical, Inc.

Andrew M. Harris, P.E.
State of Utah No. 7420456
Senior Geotechnical Engineer

Reviewed by:

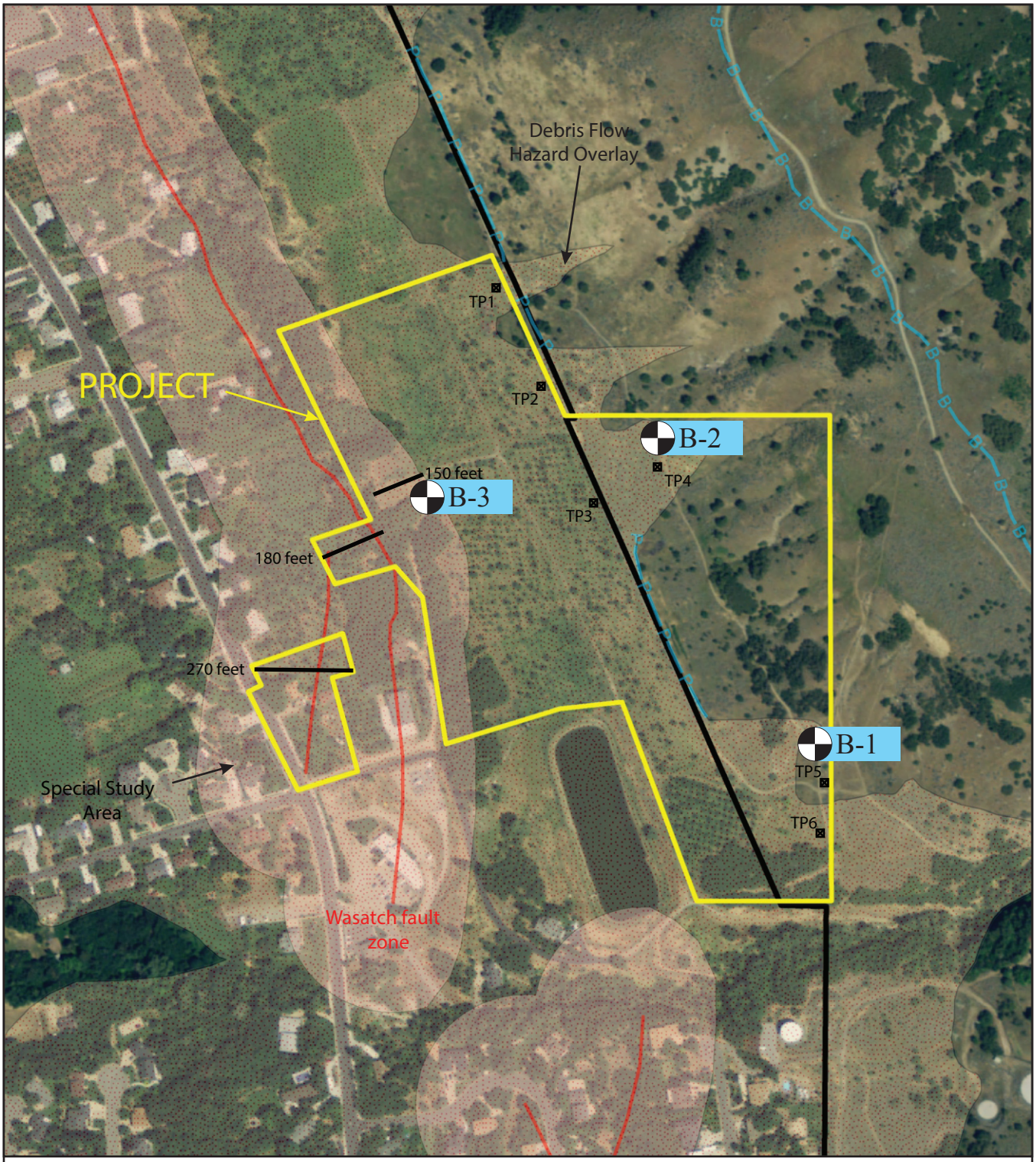
Bryan N. Roberts, P.E.
State of Utah No. 276476
Senior Geotechnical Engineer

AMH/BNR:mmh

Enclosures: Exploration Plan

Addressee (email)

CC: Tim Thompson (timt@geostrata-llc.com)



B-1 **PROPOSED BORING LOCATIONS**

ADAPTED FROM WORK PLAN BY WESTERN GEOLOGIC

N
W E
S

0 200 400 feet

Scale 1:4,800
(1 inch = 400 feet)

WESTERN GEOLOGIC

EXPLORATION PLAN

WORK PLAN
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 3

REPORT
SUPPLEMENTAL GEOTECHNICAL STUDY
PROPOSED RAYMOND PROPERTY
RESIDENTIAL DEVELOPMENT
EAST OF THE INTERSECTION OF
MOUNTAIN ROAD AND GREEN ROAD
FRUIT HEIGHTS, UTAH

Submitted To:

Lowe Companies
Attention: Mr. Matt Lowe
6028 South Ridge Drive
South Ogden, Utah

Submitted By:

GSH Geotechnical, Inc.
1596 West 2650 South, Suite 107
Ogden, Utah

August 27, 2015

Job No. 1696-02N-14

August 27, 2015
Job No. 1696-02N-15

Mr. Matt Lowe
Lowe Companies
6028 South Ridge Drive
South Ogden, Utah 84403

Mr. Lowe:

Re: Report
Supplemental Geotechnical Study
Proposed Raymond Property Residential Development
East of the Intersection of Mountain Road and Green Road
Fruit Heights, Utah
(41.0333 N, 111.8995 W)

1. INTRODUCTION

1.1 GENERAL

This report presents the results of our geotechnical study performed at the site of the proposed Raymond Property Residential Development, to be located east of the intersection of Mountain Road and Green Road in Fruit Heights, Utah. The general location of the site with respect to surrounding roadways, as of 2014, is presented on Figure 1, Vicinity Map. A more detailed layout of the site showing existing and proposed facilities and roadways is presented on Figure 2, Site Plan. The locations of the borings drilled in conjunction with this study are also presented on Figure 2. A geotechnical study[1] was previously completed for the site property by GSH, dated September 24, 2014. The information contained in the referenced report was utilized in the preparation of this study. The locations of the test pits excavated in conjunction with the referenced study are also presented on Figure 2 and the test pit logs associated with the referenced study are attached.

[1] "Report, Geotechnical Study, Proposed Park Raymond Property Residential Development, East of the Intersection of Mountain Road and Green Road, Fruit Heights, Utah," GSH Geotechnical, Inc, Job No. 1696-001-14, September 24, 2014.

1.2 OBJECTIVES AND SCOPE

The objectives and scope of our study were planned in discussions between Mr. Matt Lowe of the Lowe Companies, Mr. Alan Spilker of GSH Geotechnical, Inc. (GSH), and Mr. Andrew Harris of GSH Geotechnical, Inc. (GSH).

In general, the objectives of this study were to:

1. Define and evaluate the subsurface soil and groundwater conditions at the site.

2. Provide appropriate foundation, earthwork, slope stability, and pavement recommendations and geoseismic information to be utilized in the design and construction of the proposed development.

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

1. A field program consisting of the excavating, logging, and sampling of 15 exploration test pits during our initial study for the site and drilling, logging, and sampling 3 exploration borings as part of the current study.

2. A laboratory testing program.

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

1.3 AUTHORIZATION

Authorization was provided by returning a signed copy of our Professional Services Agreement No. 15-0643N dated June 23, 2015.

1.4 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 test pits and borings, 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, GSH 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

The proposed project consists of subdividing and constructing a 50 lot residential subdivision on the parcels located east of the intersection of Mountain Road and Green Road in Fruit Heights, Utah. The development will include single-family residences, installation of utilities to service the proposed residences, and associated roadways and pavements.

Construction will likely consist of reinforced concrete footings and basement foundation walls supporting 2 to 3 wood-framed levels above grade with some stone, brick, or stucco veneer. Projected maximum column and wall loads are on the order of 10 to 20 kips and 1 to 3 kips per lineal foot, respectively.

New residential roadways will be part of the development. It is anticipated that the residential streets will be constructed of asphalt pavement with relatively light projected traffic that includes primarily passenger vehicles, daily delivery trucks, daily buses, and an occasional semi-tractor/trailer combination.

Site development will require a moderate amount of earthwork in the form of site grading. We estimate in general that maximum cuts and fills to achieve design grades will be on the order of 2 to 5 feet. Larger fills and cuts may be required at isolated areas and should be engineered accordingly to maintain stability of the slopes at the site.

3. SITE INVESTIGATIONS

3.1 FIELD PROGRAM

In order to define and evaluate the subsurface soil and groundwater conditions at the site, 3 borings were drilled to depths of about 33.0 to 41.5 feet below existing grade within the proposed development. The borings were drilled using a truck-mounted drill rig equipped with hollow-stem augers. Locations of the borings are presented on Figure 2. Additionally, 15 test pits were excavated to depths of about 13.0 to 16.0 feet below existing grade to define and evaluate the subsurface soil and groundwater conditions at the site. The test pits were excavated using a rubber tire-mounted backhoe. Test pit locations are also presented on Figure 2.

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 excavating and drilling 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 3A through 3C, Boring Log, and on Figures 4A through 4O, Test Pit Log. Soils were classified in accordance with the nomenclature described on Figure 5, Key to Boring Log (USCS) and on Figure 6, Key to Test Pit Log (USCS).

A 3.0-inch outside diameter, 2.42-inch inside diameter drive sampler (Dames & Moore) was utilized in the subsurface soil sampling. Additionally, a 2.0-inch outside diameter, 1.38-inch inside diameter drive sampler (SPT) was utilized at select locations. 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.

A 2.42-inch inside diameter thin-wall drive sampler was utilized in the subsurface sampling of the test pits at the site.

Following completion of drilling operations, 1.25-inch diameter slotted PVC pipe was installed in Boring B-1, B-2, and B-3 in order to provide a means of monitoring potential groundwater fluctuations. The borings were backfilled with auger cuttings.

Following completion of excavating and logging, each test pit 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 LABORATORY TESTING

3.2.1 General

In order to provide data necessary for our engineering analyses, a laboratory testing program was performed. The program included performing moisture, density, partial gradation, direct shear, vane shear, and chemical tests on representative subsurface soil samples. The following paragraphs describe the tests and summarize the test data.

3.2.2 Moisture and Density

To provide index parameters and to correlate other test data, moisture and density tests were performed on selected samples. The results of these tests are presented on the boring logs, Figures 3A through 3C, and on the test pit logs, Figures 4A through 4O.

3.2.3 Partial Gradation Test

To aid in classifying the granular soils, partial gradation tests were performed. Results of the tests are tabulated on the following page.

Boring/Test Pit No.	Depth (feet)	Percent Passing No. 200 Sieve	Soil Classification
TP-1	12.0	4	GP
TP-2	4.0	6	SP
TP-3	2.0	10	SP-SM
TP-3	10.5	13	SM
TP-4	8.0	13	SM
TP-5	7.5	55	ML
TP-5	15.0	33	SM
TP-6	9.0	13	SM
TP-6	14.0	7	SP-SM
TP-7	9.0	16	SM
TP-8	4.0	15	SM
TP-8	8.0	31	SM
TP-9	14.0	69	ML
TP-10	10.5	7	SP-SM
TP-11	2.0	14	SM
TP-11	12.0	11	SM
TP-12	3.0	9	SP-SM
TP-12	15.5	6	SP
TP-13	3.0	27	GM
TP-14	8.0	6	SP
TP-15	10.0	31	SM
B-1	20.0	17	SM
B-1	30.0	12	GP/Bedrock
B-3	15.0	69	SM/ML

3.2.4 Laboratory Direct Shear Tests

To determine the shear strength of the soils encountered at the site, laboratory direct shear tests were performed. The results of the tests are tabulated below:

Boring/Test Pit No.	Depth (feet)	Soil Type	In-Situ Moisture Content (percent)	Dry Density (pcf)	Internal Friction Angle (degrees)	Apparent Cohesion (psf)
TP-4	3.5	SM	4	104	32	325
TP-8	5.0	SM	6	88	36	20
B-2	10.5	SM	6	123	36	20
B-3	35.0	CL*	20	96	38	0

*Interbedded sand seams within sample

3.2.5 Laboratory Vane Shear Tests

To determine the undrained shear strength of the soils encountered at the site, laboratory vane shear tests were performed. The results of the tests are tabulated below:

Boring No.	Depth (feet)	Soil Type	In-Situ Moisture Content (percent)	Dry Density (pcf)	Ultimate Shear Strength (psf)
B-3	20.0	CL	19	103	4180
B-3	30.0	CL	28	95	1300
B-3	40.0	CL	19	96	1800

3.2.6 Chemical Tests

In order to determine if the site soils will react detrimentally with concrete, chemical tests were performed on representative samples of the onsite soils. The results of the chemical tests are tabulated below:

Test Pit No.	Depth (feet)	Soil Classification	pH	Total Water Soluble Sulfate SO_4 (mg/kg-dry)
TP-3	5.0	SP-SM	6.9	None Detected
TP-12	3.0	SP-SM	7.0	None Detected

4. SITE CONDITIONS

4.1 GEOLOGIC SETTING

A geologic hazards evaluation study[2] dated August 15, 2015 was prepared for the subject property by Western Geologic, LLC, and a copy of that report is included in the attached Appendix.

4.2 SURFACE

The subject property consists of two parcels east of the intersection of Green Road and Mountain Road in Fruit Heights, Utah. The first parcel is a small rectangular parcel at the northeast corner of the intersection of Mountain Road and Green Road. It is anticipated that one lot will be developed on this parcel. The parcel contains an existing home and outbuilding along the north edge of the property. Vegetation at the site consists primarily of native weeds, grasses, and a number of mature trees. The subject property is relatively flat and generally slopes to the west-southwest. The parcel is bordered by Mountain Road to the west, Green Road to the south, and residential development to the north and east.

The second parcel is a large, irregularly shaped parcel east of the end of Green Road. It is anticipated that 49 lots will be developed within this parcel. The subject property is currently utilized as an orchard and contains a number of small, unpaved roads as well as an extensive irrigation system. Vegetation at the site consists primarily of native weeds, grasses, and many mature trees. The subject property generally slopes to the west-southwest at grades of approximately 10% to 50% with a change in elevation of about 230 feet across the subject property. The subject property is bordered by a residential development to the northwest, by an irrigation reservoir to the southwest, and by undeveloped property to the north, south, and east.

4.3 SUBSURFACE SOIL

The soil conditions encountered vary slightly across the site. At the majority of the boring and test pit locations, topsoil and disturbed soils were encountered at the surface of the site to about 2 to 10 inches below existing grades. Boring B-1 encountered fill material consisting of silty fine to coarse sand from the surface to about 4.0 feet below existing grades. The fill material encountered in boring B-1 was likely the result of grading fill for the adjacent trail. Natural soils were encountered beneath the fill material, topsoil, and disturbed soils within the borings and test pits to the full depth penetrated, about 13.0 to 41.4feet, and consisted of fine to coarse sand with varying amounts of silt and fine and coarse gravel, fine and coarse gravel with fine to coarse sand and varying amounts of silt, fine sandy silt, silty clay with varying fine sand content, and occasional mixtures of these soils. Occasional cobbles and boulders were encountered within the sands and gravels at the site. Major soil layering encountered ranged from a thickness of roughly 1.0 feet up to about 16.0 feet.

[2] "Report, Geologic Hazards Evaluation, Raymond Property, About 175 South Mountain Road, Fruit Heights, Utah," Western Geologic, LLC, August 15, 2015.

Similar subsurface soil conditions were encountered in the test trenches and test pits excavated as part of the referenced geologic study (see Appendix). Please refer to the referenced geologic study for a detailed description of the conditions observed.

The natural fine-grained silt and clay soils encountered were medium stiff to stiff, moist, light brown to brown in color, and will exhibit moderate strength and moderate compressibility characteristics under the anticipated loading range.

The natural sand and gravel soils encountered were loose to very dense, slightly moist to moist, dark brown to light brown in color, and will exhibit high strength and low compressibility characteristics under the anticipated loading range.

For a more detailed description of the subsurface soil conditions please refer to figures 3A through 3C, Boring Log, and figures 4A through 4O, Test Pit Log. The lines designating the interface between soil types on the boring logs and test pit logs generally represent approximate boundaries. In-situ, the transition between soil types may be gradual.

4.4 GROUNDWATER

During drilling and excavating, groundwater was not encountered to depths of 41.5 feet. Groundwater is anticipated to be at significant depths in the area. Seasonal and longer-term groundwater fluctuations on the order of 1.0 to 2.0 feet should be anticipated with the highest levels occurring during the late spring and summer months. Landscape irrigation on this and surrounding areas may also create additional seasonal groundwater fluctuations. The limitations of landscape irrigation at the site are discussed further in Section 5.9, Site Irrigation, and measures to reduce infiltration of surface water at the site are discussed further in Section 5.8, Subdrains.

5. DISCUSSIONS AND RECOMMENDATIONS

5.1 SUMMARY OF FINDINGS

The results of our analyses indicate that the proposed structures may be supported upon conventional spread and/or continuous wall foundations established upon suitable natural soils or granular structural fill extending to suitable natural soils.

The most significant geotechnical aspects of the site are the moderate strength characteristics of the near surface sandy silt soils encountered at some areas of the site and the stability of the slopes at the property.

To reduce the impact of grading at the site and maximize the distance between slopes and structures, reduction of the front yard setback to 20 feet is recommended.

The on-site soils may be re-utilized as structural site grading fill if they meet the requirements for such, as stated herein. It should be noted that from a handling and compaction standpoint, soils

containing high amounts of fines (silts and clays) are very sensitive to changes in moisture content and will require very close moisture control during placement and compaction. This will be very difficult, if not impossible, during wet and cold periods of the year.

A qualified geotechnical engineer from GSH will need to verify that all non-engineered fills, topsoil, deleterious material and/or disturbed soils have been completely removed prior to the placement of structural site grading fills, floor slabs, footings, foundations, or rigid pavements. In the following sections, detailed discussions pertaining to earthwork, foundations, at-grade concrete slabs, slope stability, and the geoseismic setting of the site are provided.

5.2 EARTHWORK

5.2.1 Site Preparation

Additional site preparation will consist of the removal of existing non-engineered fills from an area extending out at least 3 feet from the perimeter of residential structures and 1 foot beyond rigid pavements. Fill material was encountered within boring B-1 up to about 4.0 feet below existing site grade. Variation in the depth and lateral extent of non-engineered fill materials is likely. Unless testing data is available indicating that existing fills have been compacted to the requirements for structural fill, as outlined in this report, they must be considered non-engineered and removed.

Non-engineered fills may remain in asphalt pavement and sidewalk areas as long as they are properly prepared. Below rigid pavements non-engineered fills must be removed. Additionally, the surface of any existing engineered fills must be prepared prior to placing additional site grading fills.

Proper preparation shall consist of scarifying, moisture conditioning, and re-compacting the upper 12 inches to the requirements for structural fill. Fine-grained soils will require that very close moisture control be maintained for recompacting, which will be very difficult, if not impossible, to recompact during wet and cold periods of the year. As an option to proper preparation and recompaction, the upper 12 inches of non-engineered fill (where encountered) may be removed and replaced with granular subbase. Even with proper preparation, pavements established overlying non-engineered fills may encounter some long-term movements unless the non-engineered fills are completely removed.

Subsequent to stripping and prior to the placement of structural site grading fill, pavements, driveway and garage slabs on grade, the prepared subgrade must be proofrolled by passing moderate-weight rubber tire-mounted construction equipment over the surface at least twice. If excessively soft or loose soils are encountered, they must be removed to a maximum depth of 2 feet and replaced with structural fill. Beneath footings, all soft, loose, and disturbed soils must be totally removed. Non-engineered shall be handled as described above.

Surface vegetation and other deleterious materials should generally be removed from the site. Topsoil, although unsuitable for utilization as structural fill, may be stockpiled for subsequent landscaping purposes.

5.2.2 Excavations

Temporary excavations up to 8 feet deep in fine-grained cohesive soils, above or below the water table, may be constructed with sideslopes no steeper than one-half horizontal to one vertical (0.5H:1.0V). Excavations deeper than 8 feet are not anticipated at the site.

For granular (cohesionless) soils, construction excavations above the water table, not exceeding 4 feet, should be no steeper than one-half horizontal to one vertical (0.5H:1.0V). For excavations up to 8 feet, in granular soils and above the water table, the slopes should be no steeper than one horizontal to one vertical (1.0H:1.0V). Excavations encountering saturated cohesionless soils will be very difficult and will require very flat sideslopes and/or shoring, bracing and dewatering.

To reduce disturbance of the natural soils during excavation, it is recommended that smooth edge buckets/blades be utilized.

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.

5.2.3 Structural Fill

Structural fill will be required as site grading fill, as backfill over foundations and utilities, and possibly as replacement fill beneath some footings. All structural fill must be free of sod, rubbish, construction debris, 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. The maximum particle size within structural site grading fill should generally not exceed 4 inches; although, occasional particles up to 6 to 8 inches may be incorporated provided that they do not result in “honeycombing” or preclude the obtainment of the desired degree of compaction. In confined areas, the maximum particle size should generally be restricted to 2.5 inches.

On-site soils may be re-utilized as structural site grading fill if they do not contain deleterious material and meet the requirements of structural fill. However, fine-grained soils will require very close moisture control and may be very difficult, if not impossible, to properly place and compact during wet and cold periods of the year. Where the in-situ soils are likey above optimum moisture content for compacting, these soils will require some period of drying in order to properly recompact.

Only granular soils are recommended in confined areas such as utility trenches, below footings, etc. Generally, we recommend that all imported granular structural fill consist of a well-graded

mixture of sands and gravels with no more than 20 percent fines (material passing the No. 200 sieve) and no more than 30 percent retained on the three-quarter-inch sieve.

To stabilize soft subgrade conditions (if encountered) or where structural fill is required to be placed closer than 1.0 foot above the water table at the time of construction, a mixture of coarse gravels and cobbles and/or 1.5- to 2.0-inch gravel (stabilizing fill) should be utilized. It may also help to utilize a stabilization fabric, such as Mirafi 600X or equivalent, placed on the native ground if 1.5- to 2.0-inch gravel is used as stabilizing fill.

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.

5.2.4 Fill Placement and Compaction

All structural fill shall be placed in lifts not exceeding 8 inches in loose thickness. Structural fills shall be compacted in accordance with the percent of the maximum dry density as determined by the AASHTO[3] T-180 (ASTM[4] D-1557) compaction criteria in accordance with the following table:

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
Site grading fill outside the area defined above	0 to 5	90
Site grading fill outside the area defined above	5 to 8	95
Utility trench backfill below structural areas	--	96
Aggregate Base/Subbase	---	96

Structural fills greater than 8 feet thick are not anticipated at the site.

Subsequent to stripping and prior to the placement of structural site grading fill, the subgrade shall be prepared as discussed in Section 5.2.1, Site Preparation, of this report. In confined areas, subgrade preparation shall consist of the removal of all loose or disturbed soils.

[3] American Association of State Highway and Transportation Officials
[4] American Society for Testing and Materials

Coarse gravel and cobble mixtures (stabilizing fill) if utilized, should 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 stabilizing fill may be compacted by passing moderately heavy construction equipment or large self-propelled compaction equipment at least twice. Subsequent fill material placed over the coarse gravels and cobbles should be adequately compacted so that the "fines" are "worked into" the voids in the underlying coarser gravels and cobbles.

Utilization of a filter fabric, such as Mirafi 600X or equivalent, over soft subgrade may also be advantageous.

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.

5.2.5 Utility Trenches

All utility trench backfill material below structurally loaded facilities (flatwork, floor slabs, paved areas, etc.) shall be placed to the same material and density requirements established for structural fill. If the surface of the trench backfill at design subgrade becomes disturbed during the course of construction, the subgrade should be proofrolled and/or properly compacted prior to construction of any overlying exterior flatwork, floor slabs, or pavements.

Most utility companies and City-County governments are now requiring that Type A-1a or A-1b (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.

Natural or imported fine-grained cohesive soils are not recommended for use as trench backfill in structurally loaded areas.

5.3 SLOPE STABILTY

5.3.1 Input Parameters

The properties of the natural sand/gravel soils and silt/clay soils observed at the test pit and boring locations were estimated using laboratory testing, published correlations, and our experience with similar soils. Based on tests performed by the Bureau of Reclamation[5], silty sands have an internal friction angle ranging from 30 to 36 degrees and a cohesion varying from

[5] U.S. Bureau of Reclamation, 1987, "Design Standards No. 13, Embankment Dams," Denver, Colorado.

180 to 420 pounds per square foot, inorganic clays of low to medium plasticity have an internal friction angle ranging from 26 to 30 degrees and a cohesion varying from 240 to 320 pounds per square foot, and silts have an internal friction angle ranging from 30 to 34 degrees and a cohesion of about 180 pounds per square foot. Accordingly, we estimated the following parameters for use in the stability analyses:

Material	Internal Friction Angle (degrees)	Apparent Cohesion (psf)	Saturated Unit Weight (pcf)
Sands/Gravels	36	50	115
Silts/Clays	28	150	120
Bedrock	35	500	140

Guidelines set forth by Fruit Heights City for the evaluation of slope stability under seismic (pseudostatic) conditions indicate that peak horizontal accelerations should be based on a 2% probability of exceedance in a 50 year return period. This value will be used in place of the 2/3 peak ground acceleration value (adjusted for site class) allowed by IBC 2012. For the seismic (pseudostatic) analysis, a peak horizontal ground acceleration of 0.5714*g* after adjusting for Site Class D was obtained for site (grid) locations of 41.0333 degrees latitude (north) and 111.8995 degrees longitude (west). To model sustained accelerations at the site, one-half of this value is typically employed. Accordingly, a value of 0.285g was used as the pseudostatic coefficient for the stability analysis.

5.3.2 Stability Analyses

We evaluated the global stability of the existing slope using the computer program *SLIDE*. This program uses a limit equilibrium (Simplified Bishop) method for calculating factors of safety against sliding on an assumed failure surface and evaluates numerous potential failure surfaces, with the most critical failure surface identified as the one yielding the lowest factor of safety of those evaluated. We analyzed the following configurations based on cross-sections provided in the referenced geologic study (see appendix for cross-section information and locations):

- A-A' consisted of an approximately 20-foot high slope inclined at approximately 1V:9H (Vertical:Horizontal) grading to an approximately 15-foot high slope inclined at approximately 1V:3.3H (Vertical:Horizontal) grading to an approximately 45-foot high slope inclined at about 1V:4.5H (Vertical:Horizontal) grading to an approximately 40-foot high slope inclined at about 1V:1.9H (Vertical:Horizontal) grading to an approximately 75-foot high slope inclined at about 1V:6H (Vertical:Horizontal) grading to an approximately 40-foot high slope inclined at about 1V:5H (Vertical:Horizontal). To simulate the load imposed by the proposed homes, a 1,500 psf load was modeled about 30 feet from the crest of the 1V:1.9H (Vertical:Horizontal) slope.

- B-B' consisted of an approximately 25-foot high slope inclined at approximately 1V:7.5H (Vertical:Horizontal) grading to an approximately 20-foot high slope inclined at approximately 1V:3.25H (Vertical:Horizontal) grading to an approximately 15-foot high slope inclined at about 1V:5H (Vertical:Horizontal).

Typically, the required minimum factors of safety are 1.5 for static conditions and 1.0 for seismic (pseudostatic) conditions. The results of our analyses indicate that the existing slope configuration B-B' will meet both these requirements provided our recommendations are followed (see Figures 10 and 11). The results of our analyses indicate that the proposed slope configuration A-A' will meet the static stability requirements provided the proposed homes are set back at least 30 feet from the top of the steepest portion of the lower slope, but the seismic stability requirement will not be met (see Figures 7 and 8, attached). Therefore, we performed a deformation analysis using the Newmark sliding block method, using a yield acceleration (where the factor of safety is 1.0) of 0.21g (see Figure 9). The results of this analysis indicate that approximately 3.25 inches of deformation could occur due to seismic shaking at the site (see Figure 12). This value is typically considered tolerable, and no further mitigation is needed.

Slope movements or even failure can occur if the slope soils are undermined or become saturated. Groundwater was not encountered during the course of our field investigation; however saturation of the slope soils can adversely affect the stability of the slope. To model the potential effect of saturation of the site soils, the slope cross-sections described above were analyzed with an artificial water surface surface. The water surface was placed 2 feet above the clay layer in section A-A' and 2 feet below the ground surface for section B-B'. The results of this modeling indicate that the slopes for section A-A' will likely fail if the clay layers are allowed to saturate however, the slope section B-B' maintains the required factor of safety for static conditions (see Figures 13 and 14). Measures must be implemented to reduce the potential for saturation of the soils at the site. Surface drainage at the bottom and top of the slope should be directed to prevent ponding at the toe or crest of the slope, and a cut-off drain at the crest of the lower slope is recommended to reduce the potential for infiltration of surface water at the site, as discussed further in Section 5.8, Subdrains. Landscape irrigation on this and surrounding areas may also create additional seasonal groundwater fluctuations. The limitations of landscape irrigation at the site are discussed further in Section 5.9, Site Irrigation. The property owner and the owner's representatives should be made aware of the risks should these or other conditions occur that could saturate or erode/undermine the slope soils.

Changes to the grading at the site and any retaining walls must be properly engineered to maintain stability of the slopes. <u>GSH must review the final grading plans for the project prior to initiation of any construction</u>.

5.4 SPREAD AND CONTINUOUS WALL FOUNDATIONS

5.4.1 Design Data

The proposed structure may be supported upon conventional spread and continuous wall foundations established upon suitable natural soils/bedrock and/or structural fill extending to suitable natural soils/bedrock. For design, the following parameters are provided:

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	- 16 inches
Minimum Recommended Width for Isolated Spread Footings	- 24 inches
Recommended Net Bearing Pressure for Real Load Conditions	- 1,500 pounds per square foot
Bearing Pressure Increase for Seismic Loading	- 50 percent

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.

5.4.2 Installation

Under no circumstances shall the footings be established upon non-engineered fills, loose or disturbed soils, topsoil, sod, 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. For instance if the footing width is 2 feet and the structural fill depth beneath the footing is 2 feet, the fill replacement width should be 4 feet, centered beneath the footing.

5.4.3 Settlements

Maximum settlements of foundations designed and installed in accordance with recommendations presented herein and supporting maximum anticipated loads as discussed in Section 2, Proposed Construction, are anticipated to be 1 inch or less.

Approximately 60 percent of the quoted settlement should occur during construction.

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. For estimated frictional resistance, a coefficient of friction of 0.35 should be utilized for foundations established on native soils. 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, are for backfills which will consist of drained granular soil placed and compacted in accordance with the recommendations presented herein. 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 walls (moderately yielding), generally not exceeding 8 feet in height, granular backfill may be considered equivalent to a fluid with a density of 45 pounds per cubic foot. The above values assume that the surface of the soils slope behind the wall is no steeper than 4 horizontal to 1 vertical and that the granular fill within 3 feet of the wall will be compacted with hand-operated compacting equipment.

For seismic loading, a uniform pressure should be added. The uniform pressures based on different wall heights are provided in the table on the following page.

Wall Height (feet)	Seismic Loading Active Case (psf)	Seismic Loading Moderately Yielding (psf)
4	25	55
6	40	85
8	55	115

5.7 FLOOR SLABS

Lightly loaded at-grade floor slabs may be established upon suitable natural soils and/or upon structural fill extending to suitable natural soils. Under no circumstance should floor slabs be established upon loose or disturbed soils, sod, non-engineered fill, deleterious materials, expansive soils, frozen soils, or within ponded water. To facilitate construction and to provide a capillary moisture break, we recommend that at-grade slabs be immediately underlain by a minimum of 4 inches of "free-draining" granular material, such as "pea" gravel or three-quarter- to one-inch minus clean gap-graded gravel.

Settlement of lightly loaded floor slabs (average uniform pressure of 200 pounds per square foot or less) is anticipated to be less than 1/2 inch.

5.8 SUBDRAINS

5.8.1 General

Groundwater was not encountered at the site, however we recommend that the perimeter foundation subdrains and cutoff drain at the crest of the lower slope be installed as indicated below.

5.8.2 Foundation Subdrains

Foundation subdrains should consist of a 4-inch diameter perforated or slotted plastic or PVC pipe enclosed in clean gravel. The invert of a subdrain should be at least 2 feet below the top of the lowest adjacent floor slab. The gravel portion of the drain should extend 2 inches laterally and below the perforated pipe and at least 1 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 4-inch-wide zone of "free-draining" sand/gravel should be placed adjacent to the foundation walls and extend to within 2 feet of final grade. The upper 2 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/gravel, 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 drain pipe should be clean 0.75-inch to 1.0-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.

5.8.3 Cutoff Drain

To reduce potential infiltration of surface water and groundwater into the subsurface soils at the site, a cutoff drain should be installed near the crest of the lower slope. The drain should consist of a perforated 4-inch minimum diameter pipe wrapped in fabric and placed near the bottom of a minimum 24 inch wide trench excavated to a depth of at least 15 feet below existing grade and lined in filter fabric. The pipe should daylight at one or both ends of the drain and discharge to an appropriate drainage device or area. Clean gravel up to 2 inches in maximum size, with less than 10 percent passing the No. 4 sieve and less than 5 percent passing the No. 200 sieve, should be placed around the drain pipe. A fabric, such as Mirafi 140N or equivalent, should be placed between the clean gravel and the adjacent soils. A zone of clean gravel and fabric at least 24 inches wide should also extend above the drain, to within 2 feet of the ground surface, with fabric placed over the gravel. The upper 2 feet of soils should consist of a compacted clayey cap to reduce surface water infiltration into the drain.

5.9 SITE IRRIGATION

Proper site drainage is important to maintaining slope stability at the site. Saturation of soils at the site may result in slope movement or failure. Therefore, we recommend that sprinklers and sprinkler/irrigation lines not be placed within 30 feet from the crest of the slope and no irrigation lines should be placed on the slope. Landscaping at the site should be planned to utilize drought resistant plants that require minimal watering. Plants or lawn may be placed up to the crest of the slope, with plants watered using direct drip systems targeted only for each plant, and any lawn areas watered using sprinklers placed a minimum of 30 feet from the crest of the slope. Overwatering should be strictly avoided. The surface of the site should be graded to prevent the accumulation or ponding of surface water at the site. The property owner and the owner's representatives should be made aware of the risks should these or other conditions occur that could saturate or erode/undermine the slope soils.

To reduce the potential for saturation of the site soils, overwatering at the site should be strictly avoided. Watering at the site should be limited to a maximum equivalent rainfall of 0.5 inches per week. Irrigation at the site should be strictly avoided during periods of natural precipitation.

5.10 PAVEMENTS

5.10.1 Design Criteria

It is projected that proposed roadways will consist of primarily asphalt concrete. The existing natural sand soils encountered at the site will exhibit moderate pavement support characteristics when saturated or near saturated.

All pavement areas must be prepared as previously discussed (see Section 5.2.1, Site Preparation). With the subgrade soils and the projected traffic as discussed in Section 2, Proposed Construction, the pavement sections below are recommended.

Minor Streets/Cul-de-Sac Traffic
(Light to Moderate Volume of Automobiles and Light Trucks,
Light Volume of Medium-Weight Trucks,
and occasional Heavy-Weight Trucks)
[4 equivalent 18-kip axle loads per day]

Flexible Pavement:

3.0 inches	Asphalt concrete
8.0 inches	Aggregate base
Over	Suitable natural soils, properly prepared soils, and/or structural site grading fill extending to properly prepared/suitable natural soils.

Or

3.0 inches	Asphalt concrete
5.0 inches	Aggregate base
5.0 inches	Sub base
Over	Suitable natural soils, properly prepared soils, and/or structural site grading fill extending to properly prepared/suitable natural soils.

5.11 CEMENT TYPES

The laboratory tests indicate that the natural soils tested contain a negligible amount of water soluble sulfates. Based on our test results, concrete in contact with the on-site soil will have a low potential for sulfate reaction (ACI 318, Table 4.3.1). Therefore, all concrete which will be in contact with the site soils may be prepared using Type I or IA cement.

5.12 GEOSEISMIC SETTING

5.12.1 General

Utah municipalities adopted the International Building Code (IBC) 2012 and International Residential Code (IRC) for One- to Two-Family Dwellings 2012 on July 1, 2013. The IBC and IRC 2012 codes determine 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).

5.12.2 Site Class

For dynamic structural analysis, the Site Class D – Stiff Soil Profile as defined in Chapter 20 of ASCE 7 (per Section 1613.3.2, Site Class Definitions, of IBC 2012) can be utilized.

5.12.3 Faulting

Based upon our review of available literature, the Wasatch Fault passes along the eastern edge of the proposed development. A site specific discussion of faulting at the site is contained within the referenced geologic study, contained within the appendix of this report.

5.12.4 Ground Motions

The IBC 2012 code is based on 2008 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 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 D soil profile in the second column. Based on the site latitude and longitude (41.0333 degrees north and -111.8995 degrees west, respectively), the values for this site are tabulated below.

Spectral Acceleration Value, T	Site Class B Boundary [mapped values] (% g)	Site Coefficient	Site Class D [adjusted for site class effects] (% g)	Design Values (% g)
Peak Ground Acceleration	51.0	F_a = 1.000	51.0	34.0
0.2 Seconds (Short Period Acceleration)	S_S = 127.4	F_a = 1.000	S_{MS} = 127.4	S_{DS} = 84.9
1.0 Second (Long Period Acceleration)	S_1 = 49.5	F_v = 1.505	S_{M1} = 74.5	S_{D1} = 49.7

5.12.5 Liquefaction

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.

Liquefaction of near-surface soils at the site is not anticipated during the design seismic event due to the unsaturated nature of the site soils. However the liquefaction susceptibility of the underlying soils (deeper than our explorations) is unknown and would require further investigation to quantify.

5.13 SITE VISITS

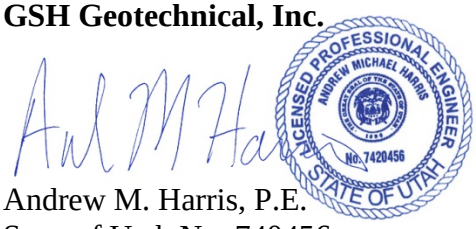
As stated previously, prior to placement of foundations, floor slabs, pavements, and site grading fills, a geotechnical engineer from GSH must verify that all non-engineered fills, topsoil, and disturbed/loose soils have been removed and suitable subgrade conditions encountered. Additionally, GSH must observe fill placement and verify in-place moisture content and density of fill materials placed at the site.

5.14 CLOSURE

If you have any questions or would like to discuss these items further, please feel free to contact us at (801) 393-2012.

Respectfully submitted,

GSH Geotechnical, Inc.



Andrew M. Harris, P.E.
State of Utah No. 740456
Senior Geotechnical Engineer

Reviewed by:



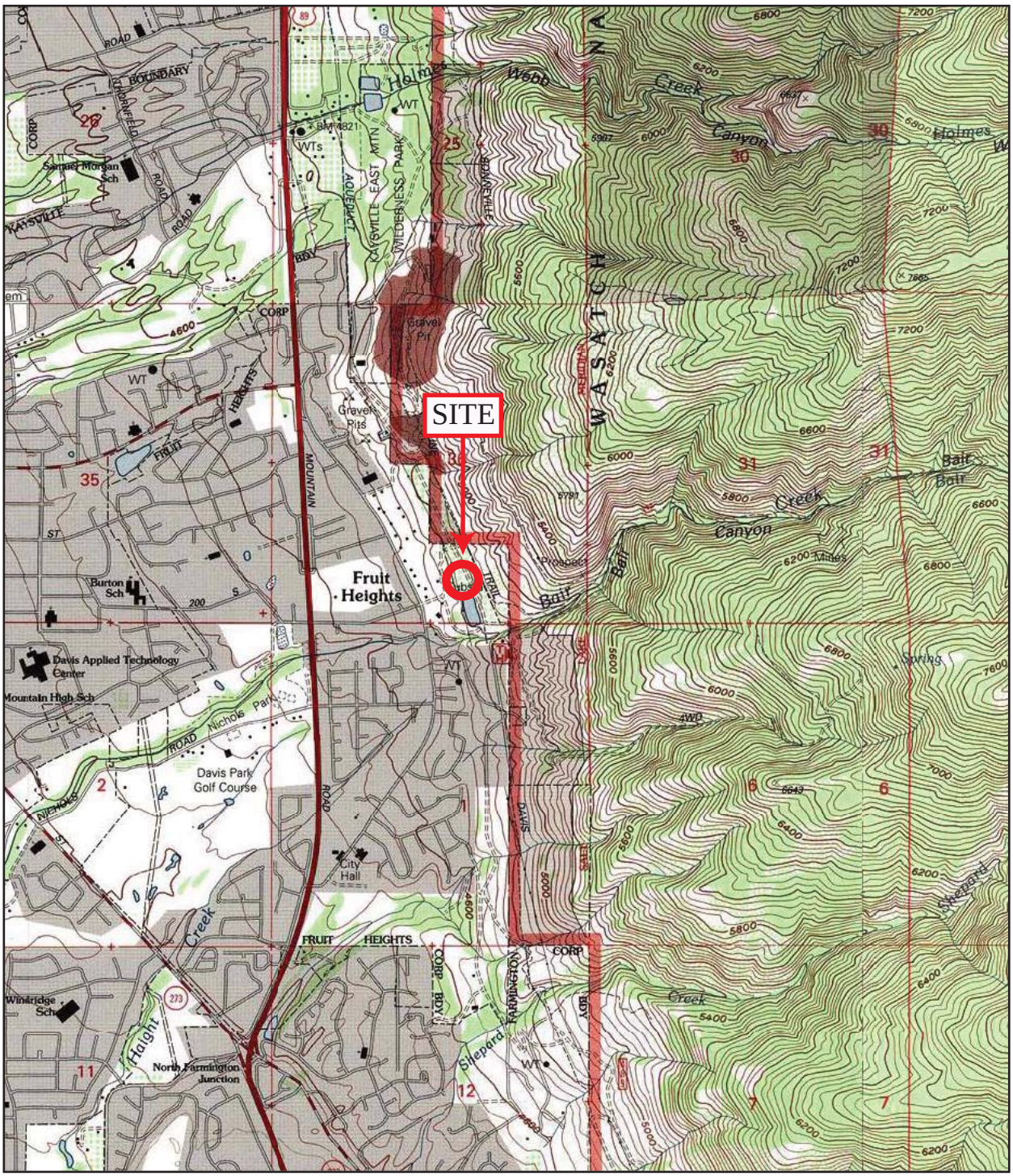
Michael S. Huber, P.E.
State of Utah No. 343650
Vice President/Senior Geotechnical Engineer

AMH/MSH:mmh

Encl. Figure 1, Vicinity Map
Figure 2, Site Plan
Figures 3A through 3C, Boring Log
Figures 4A through 4O, Test Pit Log
Figure 5, Key to Boring Log (USCS)
Figure 6, Key to Test Pit Log (USCS)
Figures 7 through 11, Stability Results
Figure 12, Newmark Displacement Method – Analysis Results
Figures 13 and 14, Stability Results – Saturated Conditions
Appendix

Addressee (email)

LOWE COMPANIES
JOB NO. 1696-001-14



SCALE IN FEET
1000 0 1000 2000

FIGURE 1
VICINITY MAP

REFERENCE:
USGS 7.5 MINUTE TOPOGRAPHIC QUADRANGLE MAP(S)
ENTITLED "KAYSVILLE, UTAH" AND "PETERSON, UTAH"
BOTH DATED 1998

GSH

LOWE COMPANIES
JOB NO. 1696-02N-15

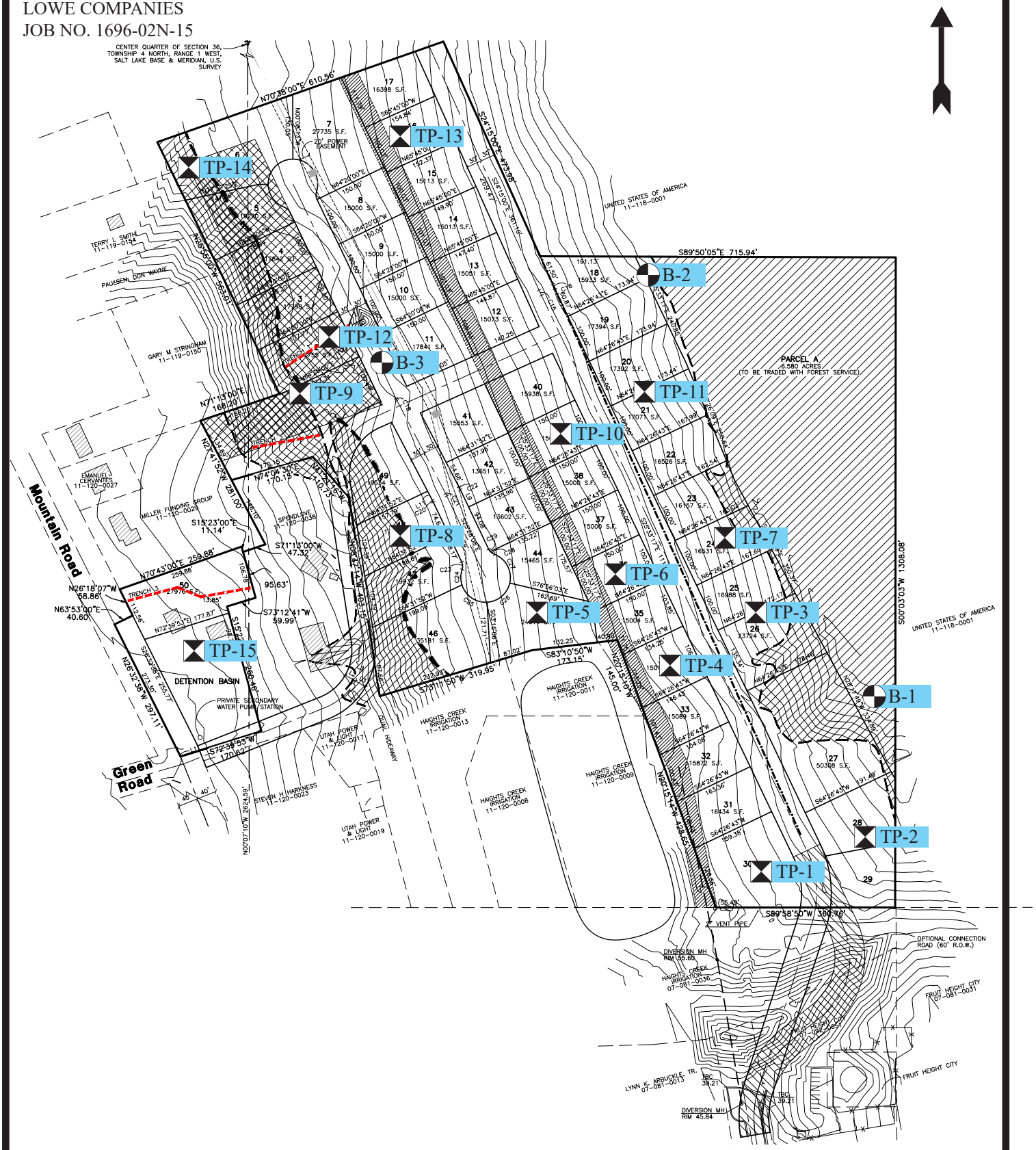


FIGURE 2
SITE PLAN

REFERENCE:
ADAPTED FROM DRAWING ENTITLED
RAYMOND PROPERTY PRELIMINARY PLAN
BY REEVE & ASSOCIATES, INC., DATE 12 MAY 2015

GSH

BORING LOG

Page: 1 of 2

BORING: B-1

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-02N-15

PROJECT: Raymond Property | DATE STARTED: 7/10/15 | DATE FINISHED: 7/10/15

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: 3-3/4" ID Hollow-Stem Auger | HAMMER: Automatic | WEIGHT: 140 lbs | DROP: 30"

GROUNDWATER DEPTH: Not Encountered (7/10/15), Not Encountered (7/30/15) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	BLOW COUNT	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0								
	SM FILL	SILTY FINE TO COARSE SAND, FILL with some fine gravel; clay; major roots (topsoil) to 10"; brown									slightly moist loose
				19							
	SM	SILTY FINE TO COARSE SAND with some fine gravel	5								
		grades with some coarse gravel		17		7	105				
	SM	FINE TO COARSE SAND with silt; occasional layers of fine sandy fine gravel up to 1/2" thick; brown	10	26		4	109				slightly moist medium dense
	GM	FINE TO COARSE SANDY FINE AND COARSE GRAVEL with silt; brown	15	41							slightly moist medium dense
	SM	FINE TO COARSE SAND with silt; brown									slightly moist medium dense to very dense
			20	50/5"		8		17			
	GP	FINE AND COARSE GRAVEL with some fine to coarse sand; trace silt; brownish-gray									slightly moist very dense
			25								

See Subsurface Conditions section in the report for additional information.

FIGURE 3A

BORING LOG

Page: 2 of 2

BORING: B-1

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-02N-15

PROJECT: Raymond Property | DATE STARTED: 7/10/15 | DATE FINISHED: 7/10/15

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	BLOW COUNT	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
			25								
				50/4"							
		grades weathered bedrock; gray	30	50/3"		1		12			dry
				50/3"							
		End of Exploration at 33.5' No groundwater encountered at time of drilling Installed 1.25" diameter slotted PVC pipe to 33.0'	35								
			40								
			45								
			50								

See Subsurface Conditions section in the report for additional information.

FIGURE 3A
(continued)

BORING LOG

Page: 1 of 2

BORING: B-2

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-02N-15

PROJECT: Raymond Property | DATE STARTED: 7/12/15 | DATE FINISHED: 7/12/15

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RAG

DRILLING METHOD/EQUIPMENT: 3-3/4" ID Hollow-Stem Auger | HAMMER: Automatic | WEIGHT: 140 lbs | DROP: 30"

GROUNDWATER DEPTH: Not Encountered (7/12/15), Not Encountered (7/30/15) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	BLOW COUNT	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0								
	SM	SILTY FINE TO COARSE SAND with some fine gravel; dark brown									moist medium dense
				12							
		grades brown	5								
				26		6	123				
		grades with some fine and coarse gravel; brown	10	23		6	123				
	SP	FINE TO COARSE SAND with some fine gravel; some silt; brown	15	19							moist medium dense
			20	23		4	99				
	SP	FINE TO COARSE SAND	25								

See Subsurface Conditions section in the report for additional information.

FIGURE 3B

GSH

BORING LOG

Page: 2 of 2

BORING: B-2

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-02N-15

PROJECT: Raymond Property | DATE STARTED: 7/12/15 | DATE FINISHED: 7/12/15

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	BLOW COUNT	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
	SP	FINE TO COARSE SAND with some fine and coarse gravel; some silt; brown	25	37							moist medium dense
		grades fine sand with occasional to some fine and coarse gravel; some silt; brown	30	13		7	98				loose
	GP	COARSE GRAVEL with fine to coarse sand; trace silt; brown	35	100+							moist very dense
				50/1"							
		End of Exploration at 38.0' due to auger refusal No groundwater encountered at time of drilling Installed 1.25" diameter slotted PVC pipe to 38.0'	40								
			45								
			50								

See Subsurface Conditions section in the report for additional information.

FIGURE 3B
(continued)

BORING LOG

Page: 1 of 2

BORING: B-3

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-02N-15

PROJECT: Raymond Property | DATE STARTED: 7/9/15 | DATE FINISHED: 7/9/15

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: 3-3/4" ID Hollow-Stem Auger | HAMMER: Automatic | WEIGHT: 140 lbs | DROP: 30"

GROUNDWATER DEPTH: Not Encountered (7/9/15), Not Encountered (7/30/15) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	BLOW COUNT	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0								
	GP GM	FINE TO COARSE SANDY FINE AND COARSE GRAVEL with some silt; major roots (topsoil) to 8"; brownish-gray		84							slightly moist dense
	SP SM	FINE TO COARSE SAND with some fine gravel; some silt; brown	5	39		3	118				slightly moist medium dense
		grades with occasional layers of silty fine to medium sand up to 4" thick									
	SP SM	FINE SAND with some silt; occasional layers of clayey silt with some fine sand up to 2" thick; brown	10	72							slightly moist dense
	SM/ ML	SILTY FINE SAND/FINE SANDY SILT with some clay; occasional layers of fine sandy clay with some silt up to 1" thick; brown	15	39		15	101	69			slightly moist medium dense/ very stiff
	CL	FINE SANDY CLAY with some silt; occasional layers of silty fine sand up to 1/4" thick; brown	20	44		19	103				slightly moist very stiff
			25								

See Subsurface Conditions section in the report for additional information.

FIGURE 3C

GSH

BORING LOG

BORING: B-3

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-02N-15

PROJECT: Raymond Property | DATE STARTED: 7/9/15 | DATE FINISHED: 7/9/15

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	BLOW COUNT	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
			25	35							moist
	CL	SILTY CLAY with some fine sand; occasional layers of fine sandy clay up to 1/8" thick; brown	30	25		28	95				very moist very stiff
		grades with occasional layers of sandy clay up to 2" thick	35	42		20	96				
			40	35		19	96				
		End of Exploration at 41.5' No groundwater encountered at time of drilling Installed 1.25" diameter slotted PVC pipe to 41.5'	45								
			50								

See Subsurface Conditions section in the report for additional information.

FIGURE 3C
(continued)

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-1

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-001-14

PROJECT: Proposed Raymond Property Residential Development | DATE STARTED: 8/19/14 | DATE FINISHED: 8/19/14

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: JCB 214S - Backhoe | HAMMER: --- | WEIGHT: --- | DROP: ---

GROUNDWATER DEPTH: Not Encountered (8/19/14) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	SM	SILTY FINE SAND with occasional fine gravel; major roots (topsoil) to 3"; trace organics; brown								moist medium dense
		grades with some fine and coarse gravel; occasional cobbles; trace organics; brown	5							slightly moist
		grades silty fine and coarse gravel with fine to coarse sand; occasional cobbles; brown								
		grades with numerous cobbles and occasional boulders	10							
		grades fine and coarse gravel with fine to coarse sand; some silt; brown								dense
		End of Exploration at 13.0' No groundwater encountered at time of excavation	15							
			20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 4A

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-2

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-001-14

PROJECT: Proposed Raymond Property Residential Development | DATE STARTED: 8/19/14 | DATE FINISHED: 8/19/14

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: JCB 214S - Backhoe | HAMMER: --- | WEIGHT: --- | DROP: ---

GROUNDWATER DEPTH: Not Encountered (8/19/14) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	SM	SILTY FINE SAND occasional fine gravel; major roots (topsoil) to 4"; brown								slightly moist medium dense
		grades with some fine and coarse gravel; brown	5							
		occasional cobbles								
		numerous cobbles and occasional boulders	10							dense
		grades fine and coarse gravel with fine to coarse sand; trace to some silt; occasional cobbles								
		End of Exploration at 14.0' No groundwater encountered at time of excavation	15							
			20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 4B

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-3

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-001-14

PROJECT: Proposed Raymond Property Residential Development | DATE STARTED: 8/19/14 | DATE FINISHED: 8/19/14

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: JCB 214S - Backhoe | HAMMER: --- | WEIGHT: --- | DROP: ---

GROUNDWATER DEPTH: Not Encountered (8/19/14) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	SM/ GM	SILTY FINE TO COARSE SAND with fine and coarse gravel; major roots (topsoil) to 4"; brown								slightly moist loose
			5							medium dense
			10							moist
		grades fine to medium sand; occasional fine gravel; trace silt; light brown	15							
		End of Exploration at 16.0' No groundwater encountered at time of excavation	20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 4C

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-4

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-001-14

PROJECT: Proposed Raymond Property Residential Development | DATE STARTED: 8/19/14 | DATE FINISHED: 8/19/14

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: JCB 214S - Backhoe | HAMMER: --- | WEIGHT: --- | DROP: ---

GROUNDWATER DEPTH: Not Encountered (8/19/14) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	SM	SILTY FINE TO COARSE SAND with occasional fine gravel; major roots (topsoil) to 3"; brown								slightly moist loose
										medium dense
		grades fine to coarse sand with occasional fine gravel; brown	5							moist
			10							
		End of Exploration at 15.0' No groundwater encountered at time of excavation	15							
			20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 4D

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-5

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-001-14

PROJECT: Proposed Raymond Property Residential Development | DATE STARTED: 8/19/14 | DATE FINISHED: 8/19/14

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: JCB 214S - Backhoe | HAMMER: --- | WEIGHT: --- | DROP: ---

GROUNDWATER DEPTH: Not Encountered (8/19/14) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	SM	SILTY FINE TO COARSE SAND with some fine and coarse gravel; major roots (topsoil) to 5"; brown								slightly moist medium dense
		grades with occasional to numerous cobbles 4" to 12" in diameter; occasional boulders up to 2.5' in diameter	5							
	GM	SILTY FINE AND COARSE GRAVEL with fine to coarse sand; brown								slightly moist medium dense
	ML/SM	FINE SANDY SILT/SILTY FINE SAND light brown	10							moist loose
		grades silty fine sand; light brown								
			15							medium dense
		End of Exploration at 16.0' No groundwater encountered at time of excavation Installed 1.25" diameter slotted PVC pipe to 16.0'	20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 4E

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-6

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-001-14

PROJECT: Proposed Raymond Property Residential Development | DATE STARTED: 8/19/14 | DATE FINISHED: 8/19/14

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: JCB 214S - Backhoe | HAMMER: --- | WEIGHT: --- | DROP: ---

GROUNDWATER DEPTH: Not Encountered (8/19/14) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	SM/ GM	SILTY FINE TO COARSE SAND with fine and coarse gravel; major roots (topsoil) to 4"; brown								slightly moist medium dense
			5							moist loose
		grades fine to coarse sand with some fine gravel; some silt; brown	10							loose to medium dense
		occasional fine and coarse gravel; brown	15							medium dense
		End of Exploration at 15.0' No groundwater encountered at time of excavation	20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 4F

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-7

CLIENT: Lowe Companies | PROJECT NUMBER: 1696-001-14

PROJECT: Proposed Raymond Property Residential Development | DATE STARTED: 8/19/14 | DATE FINISHED: 8/19/14

LOCATION: East of the intersection of Mountain Road and Green Road, Fruit Heights, Utah | GSH FIELD REP.: RG

DRILLING METHOD/EQUIPMENT: JCB 214S - Backhoe | HAMMER: --- | WEIGHT: --- | DROP: ---

GROUNDWATER DEPTH: Not Encountered (8/19/14) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	SM/ GM	SILTY FINE TO COARSE SAND with some fine and coarse gravel; major roots (topsoil) to 5"; trace organics; brown								slightly moist medium dense
		occasional cobbles and boulders	5							
		grades silty fine sand with occasional to some fine and coarse gravel; brown	10							slightly moist loose
										medium dense
		End of Exploration at 15.0' No groundwater encountered at time of excavation	15							
			20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 4G

CLIENT: Lowe Companies
PROJECT: Raymond Property
PROJECT NUMBER: 1696-02N-15

KEY TO BORING LOG

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	BLOW COUNT	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
①	②	③	④	⑤	⑥	⑦	⑧	⑨	⑩	⑪	⑫

COLUMN DESCRIPTIONS

① **Water Level:** Depth to measured groundwater table. See symbol below.

② **USCS:** (Unified Soil Classification System) Description of soils encountered; typical symbols are explained below.

③ **Description:** Description of material encountered; may include color, moisture, grain size, density/consistency,

④ **Depth (ft.):** Depth in feet below the ground surface.

⑤ **Blow Count:** Number of blows to advance sampler 12" beyond first 6", using a 140-lb hammer with 30" drop.

⑥ **Sample Symbol:** Type of soil sample collected at depth interval shown; sampler symbols are explained below.

⑦ **Moisture (%):** Water content of soil sample measured in laboratory; expressed as percentage of dryweight of

⑧ **Dry Density (pcf):** The density of a soil measured in laboratory; expressed in pounds per cubic foot.

⑨ **% Passing 200:** Fines content of soils sample passing a No. 200 sieve; expressed as a percentage.

⑩ **Liquid Limit (%):** Water content at which a soil changes from plastic to liquid behavior.

⑪ **Plasticity Index (%):** Range of water content at which a soil exhibits plastic properties.

⑫ **Remarks:** Comments and observations regarding drilling or sampling made by driller or field personnel. May include other field and laboratory test results using the following abbreviations:

CEMENTATION:

Weakly: Crumbles or breaks with handling or slight finger pressure.

Moderately: Crumbles or breaks with considerable finger pressure.

Strongly: Will not crumble or break with finger pressure.

MODIFIERS:

Trace <5%

Some 5-12%

With > 12%

MOISTURE CONTENT (FIELD TEST):

Dry: Absence of moisture, dusty, dry to the touch.

Moist: Damp but no visible water.

Saturated: Visible water, usually soil below water table.

Descriptions and stratum lines are interpretive; field descriptions may have been modified to reflect lab test results. Descriptions on the logs apply only at the specific boring locations and at the time the borings were advanced; they are not warranted to be representative of subsurface conditions at other locations or times.

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

MAJOR DIVISIONS			USCS SYMBOLS	TYPICAL DESCRIPTIONS
COARSE-GRAINED SOILS More than 50% of material is larger than No. 200 sieve size.	GRAVELS More than 50% of coarse fraction retained on No. 4 sieve.	CLEAN GRAVELS (little or no fines)	GW	Well-Graded Gravels, Gravel-Sand Mixtures, Little or No Fines
			GP	Poorly-Graded Gravels, Gravel-Sand Mixtures, Little or No Fines
		GRAVELS WITH FINES (appreciable amount of fines)	GM	Silty Gravels, Gravel-Sand-Silt Mixtures
			GC	Clayey Gravels, Gravel-Sand-Clay Mixtures
	SANDS More than 50% of coarse fraction passing through No. 4 sieve.	CLEAN SANDS (little or no fines)	SW	Well-Graded Sands, Gravelly Sands, Little or No Fines
			SP	Poorly-Graded Sands, Gravelly Sands, Little or No Fines
		SANDS WITH FINES (appreciable amount of fines)	SM	Silty Sands, Sand-Silt Mixtures
			SC	Clayey Sands, Sand-Clay Mixtures
FINE-GRAINED SOILS More than 50% of material is smaller than No. 200 sieve size.	SILTS AND CLAYS Liquid Limit less than 50%		ML	Inorganic Silts and Very Fine Sands, Rock Flour, Silty or Clayey Fine Sands or Clayey Silts with Slight Plasticity
			CL	Inorganic Clays of Low to Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean Clays
			OL	Organic Silts and Organic Silty Clays o f Low Plasticity
	SILTS AND CLAYS Liquid Limit greater than 50%		MH	Inorganic Silts, Micacious or Diatomacious Fine Sand or Silty Soils
			CH	Inorganic Clays of High Plasticity, Fat Clays
			OH	Organic Silts and Organic Clays of Medium to High Plasticity
HIGHLY ORGANIC SOILS			PT	Peat, Humus, Swamp Soils with High Organic Contents

Note: Dual Symbols are used to indicate borderline soil classifications.

STRATIFICATION:

DESCRIPTION	THICKNESS
Seam	up to 1/8"
Layer	1/8" to 12"
Occasional:	One or less per 6" of thickness
Numerous;	More than one per 6" of thickness

TYPICAL SAMPLER GRAPHIC SYMBOLS

- Bulk/Bag Sample
- Standard Penetration Split Spoon Sampler
- Rock Core
- No Recovery
- 3.25" OD, 2.42" ID D&M Sampler
- 3.0" OD, 2.42" ID D&M Sampler
- California Sampler
- Thin Wall

WATER SYMBOL

Water Level

FIGURE 5

GSH

CLIENT: Lowe Companies
PROJECT: Proposed Raymond Property Residential Development
PROJECT NUMBER: 1696-001-14

KEY TO TEST PIT LOG

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
①	②	③	④	⑤	⑥	⑦	⑧	⑨	⑩	⑪

COLUMN DESCRIPTIONS

① **Water Level:** Depth to measured groundwater table. See symbol below.

② **USCS:** (Unified Soil Classification System) Description of soils encountered; typical symbols are explained below.

③ **Description:** Description of material encountered; may include color, moisture, grain size, density/consistency,

④ **Depth (ft.):** Depth in feet below the ground surface.

⑤ **Sample Symbol:** Type of soil sample collected at depth interval shown; sampler symbols are explained below.

⑥ **Moisture (%):** Water content of soil sample measured in laboratory; expressed as percentage of dryweight of

⑦ **Dry Density (pcf):** The density of a soil measured in laboratory; expressed in pounds per cubic foot.

⑧ **% Passing 200:** Fines content of soils sample passing a No. 200 sieve; expressed as a percentage.

⑨ **Liquid Limit (%):** Water content at which a soil changes from plastic to liquid behavior.

⑩ **Plasticity Index (%):** Range of water content at which a soil exhibits plastic properties.

⑪ **Remarks:** Comments and observations regarding drilling or sampling made by driller or field personnel. May include other field and laboratory test results using the following abbreviations:

CEMENTATION:	MODIFIERS:	MOISTURE CONTENT (FIELD TEST):
Weakly: Crumbles or breaks with handling or slight finger pressure.	**Trace** <5%	**Dry:** Absence of moisture, dusty, dry to the touch.
Moderately: Crumbles or breaks with considerable finger pressure.	**Some** 5-12%	**Moist:** Damp but no visible water.
Strongly: Will not crumble or break with finger pressure.	**With** > 12%	**Saturated:** Visible water, usually soil below water table.

Descriptions and stratum lines are interpretive; field descriptions may have been modified to reflect lab test results. Descriptions on the logs apply only at the specific boring locations and at the time the borings were advanced; they are not warranted to be representative of subsurface conditions at other locations or times.

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			GP	Poorly-Graded Gravels, Gravel-Sand Mixtures, Little or No Fines
		GRAVELS WITH FINES (appreciable amount of fines)	GM	Silty Gravels, Gravel-Sand-Silt Mixtures
			GC	Clayey Gravels, Gravel-Sand-Clay Mixtures
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			SP	Poorly-Graded Sands, Gravelly Sands, Little or No Fines
		SANDS WITH FINES (appreciable amount of fines)	SM	Silty Sands, Sand-Silt Mixtures
			SC	Clayey Sands, Sand-Clay Mixtures
FINE-GRAINED SOILS More than 50% of material is smaller than No. 200 sieve size.	**SILTS AND CLAYS** Liquid Limit less than 50%		ML	Inorganic Silts and Very Fine Sands, Rock Flour, Silty or Clayey Fine Sands or Clayey Silts with Slight Plasticity
			CL	Inorganic Clays of Low to Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean Clays
			OL	Organic Silts and Organic Silty Clays o f Low Plasticity
	SILTS AND CLAYS Liquid Limit greater than 50%		MH	Inorganic Silts, Micacious or Diatomacious Fine Sand or Silty Soils
			CH	Inorganic Clays of High Plasticity, Fat Clays
			OH	Organic Silts and Organic Clays of Medium to High Plasticity
HIGHLY ORGANIC SOILS			PT	Peat, Humus, Swamp Soils with High Organic Contents

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- No Recovery
- 3.25" OD, 2.42" ID D&M Sampler
- 3.0" OD, 2.42" ID D&M Sampler
- California Sampler
- Thin Wall

WATER SYMBOL

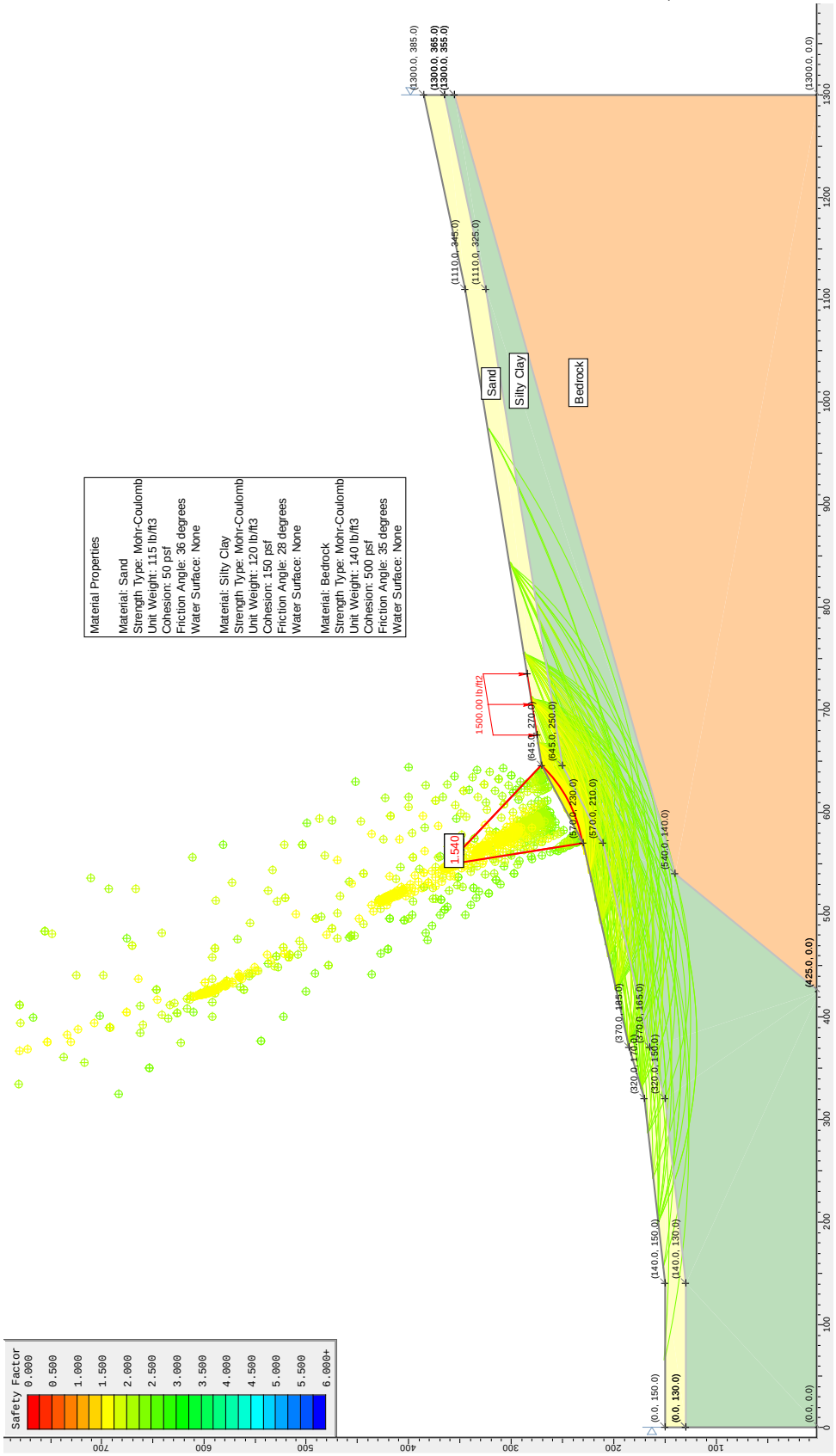
- Water Level

FIGURE 6

GSH

STABILITY RESULTS

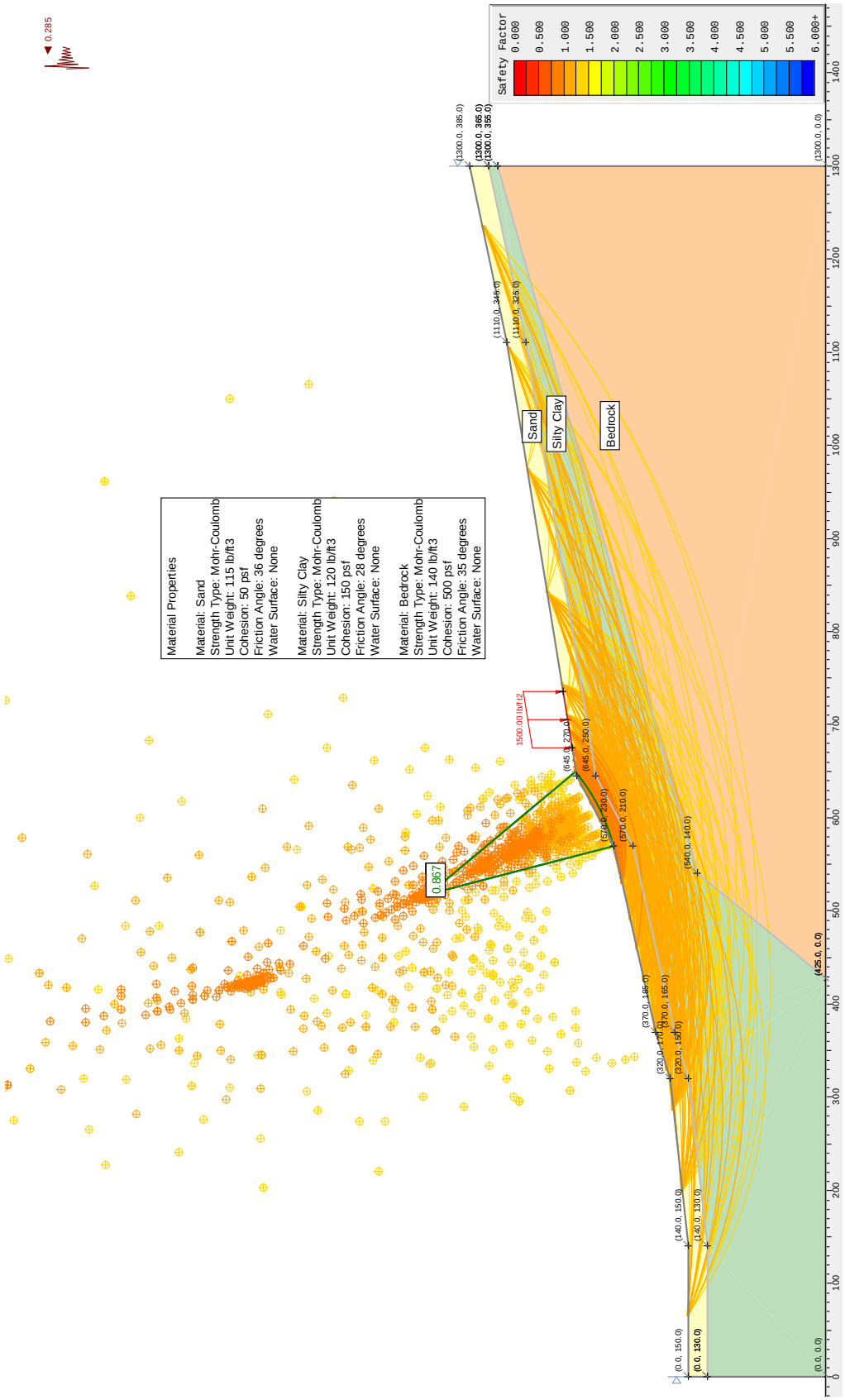
RAYMOND PROPERTY DEVELOPMENT, FRUIT HEIGHTS (SECTION A-A')



PROJECT NO.: 1696-02N-15	GSH	**FIGURE NO.:** 7

STABILITY RESULTS

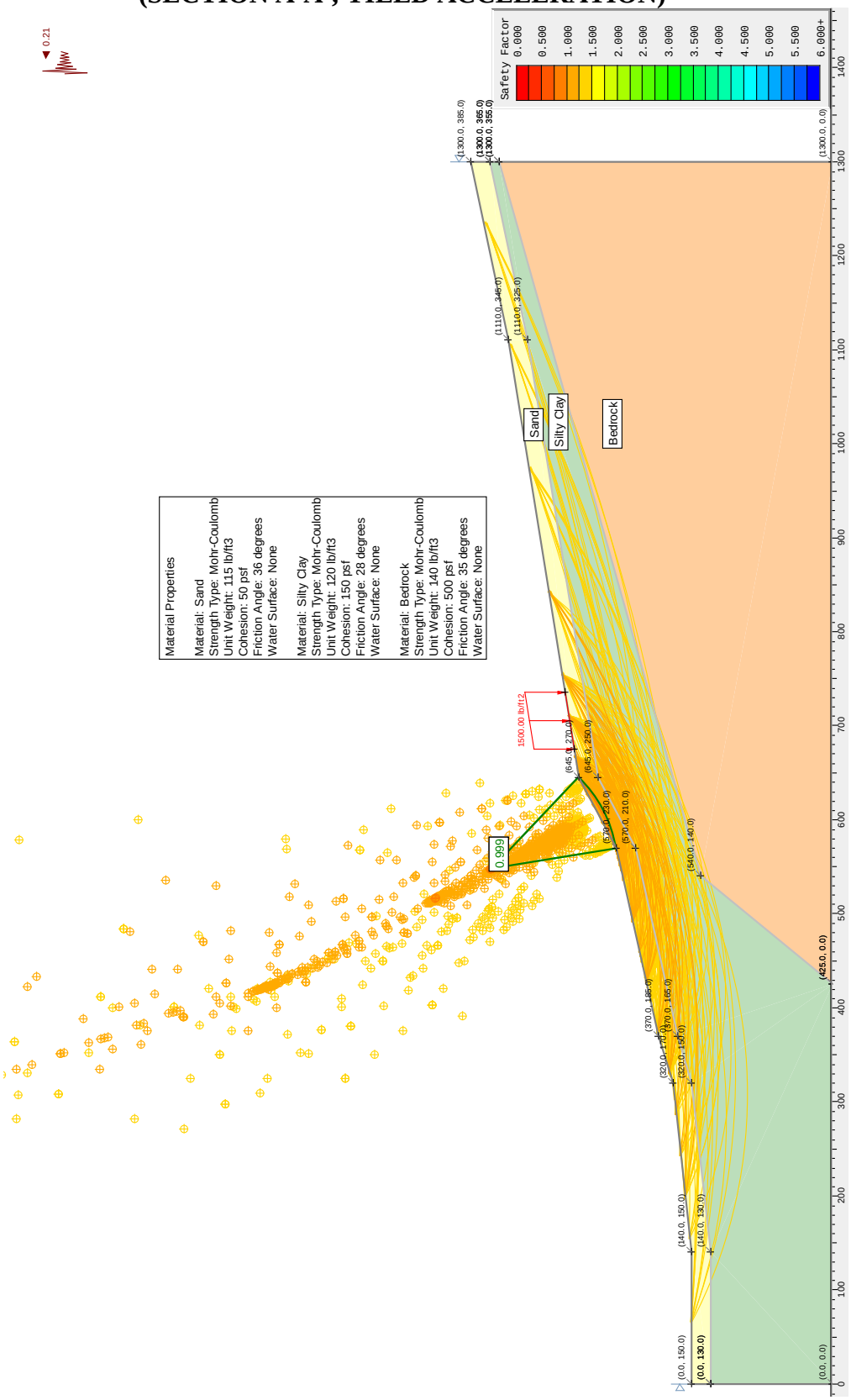
RAYMOND PROPERTY DEVELOPMENT, FRUIT HEIGHTS (SECTION A-A')



PROJECT NO.: 1696-02N-15	GSH	**FIGURE NO.:** 8

STABILITY RESULTS

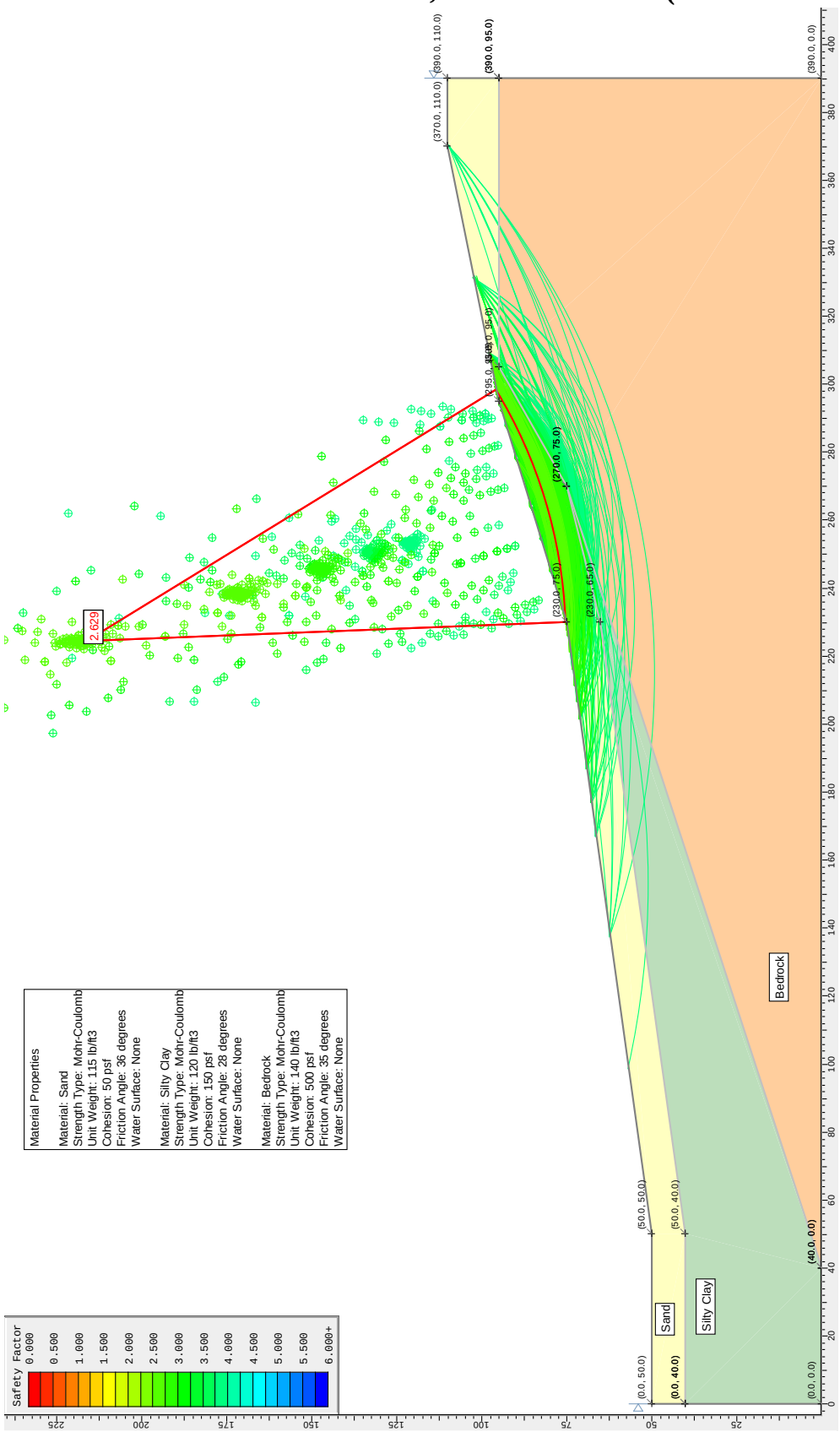
RAYMOND PROPERTY DEVELOPMENT, FRUIT HEIGHTS
(SECTION A-A', YIELD ACCELERATION)



PROJECT NO.: 1696-02N-15	GSH	**FIGURE NO.:** 9

STABILITY RESULTS

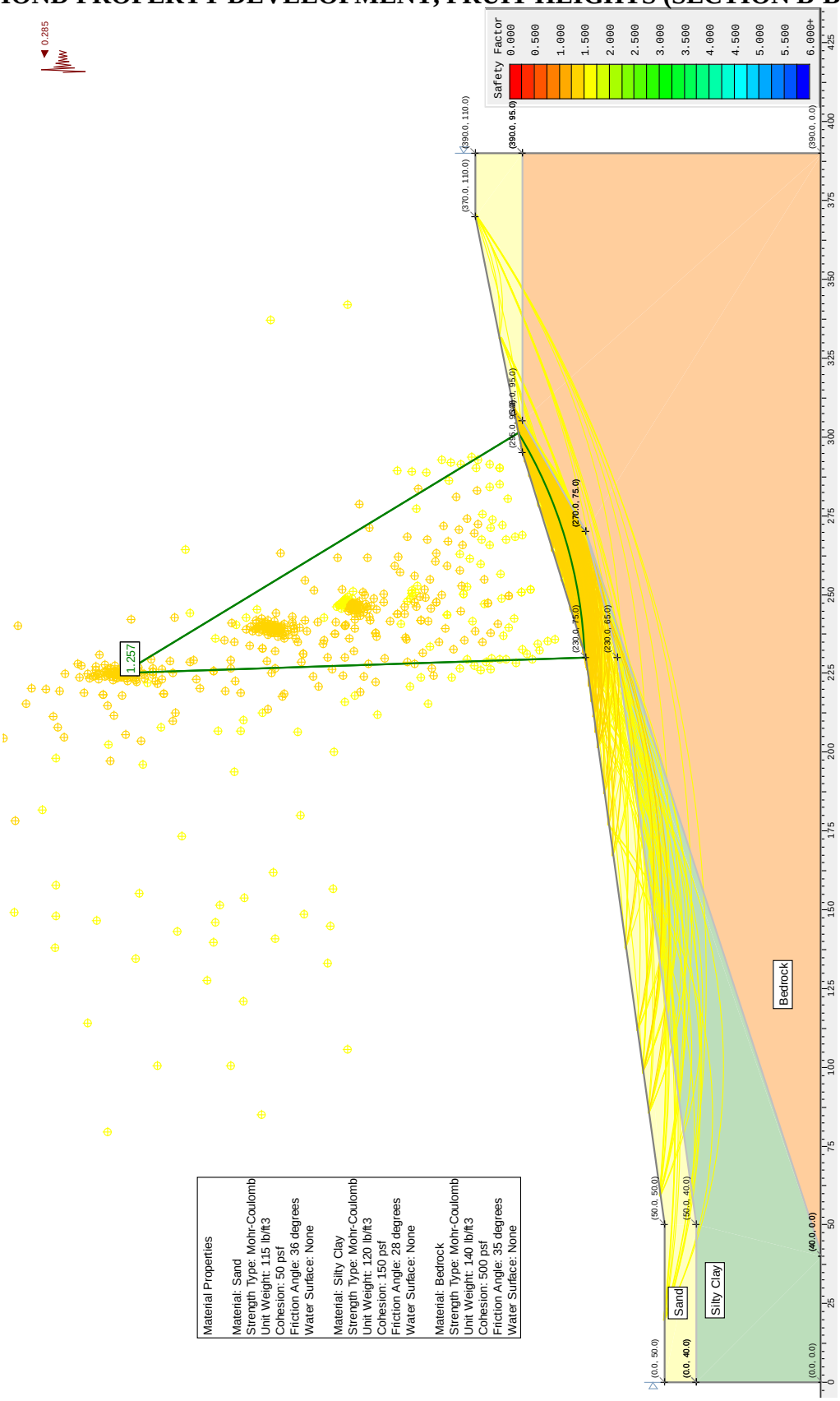
RAYMOND PROPERTY DEVELOPMENT, FRUIT HEIGHTS (SECTION B-B')



PROJECT NO.: 1696-02N-15	GSH	**FIGURE NO.:** 10

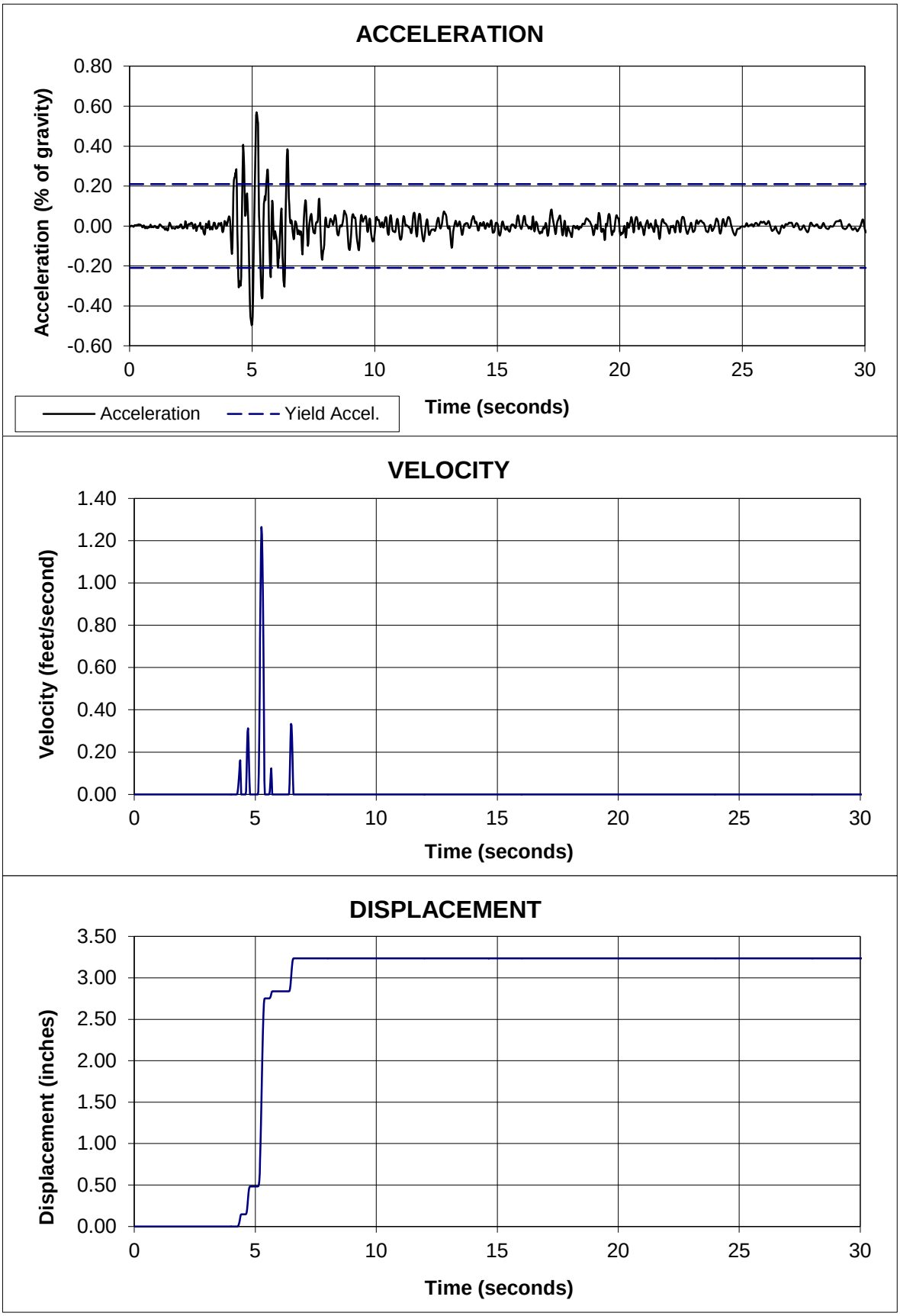
STABILITY RESULTS

RAYMOND PROPERTY DEVELOPMENT, FRUIT HEIGHTS (SECTION B-B')



PROJECT NO.: 1696-02N-15	GSH	**FIGURE NO.:** 11

NEWMARK DISPLACEMENT METHOD - ANALYSIS RESULTS

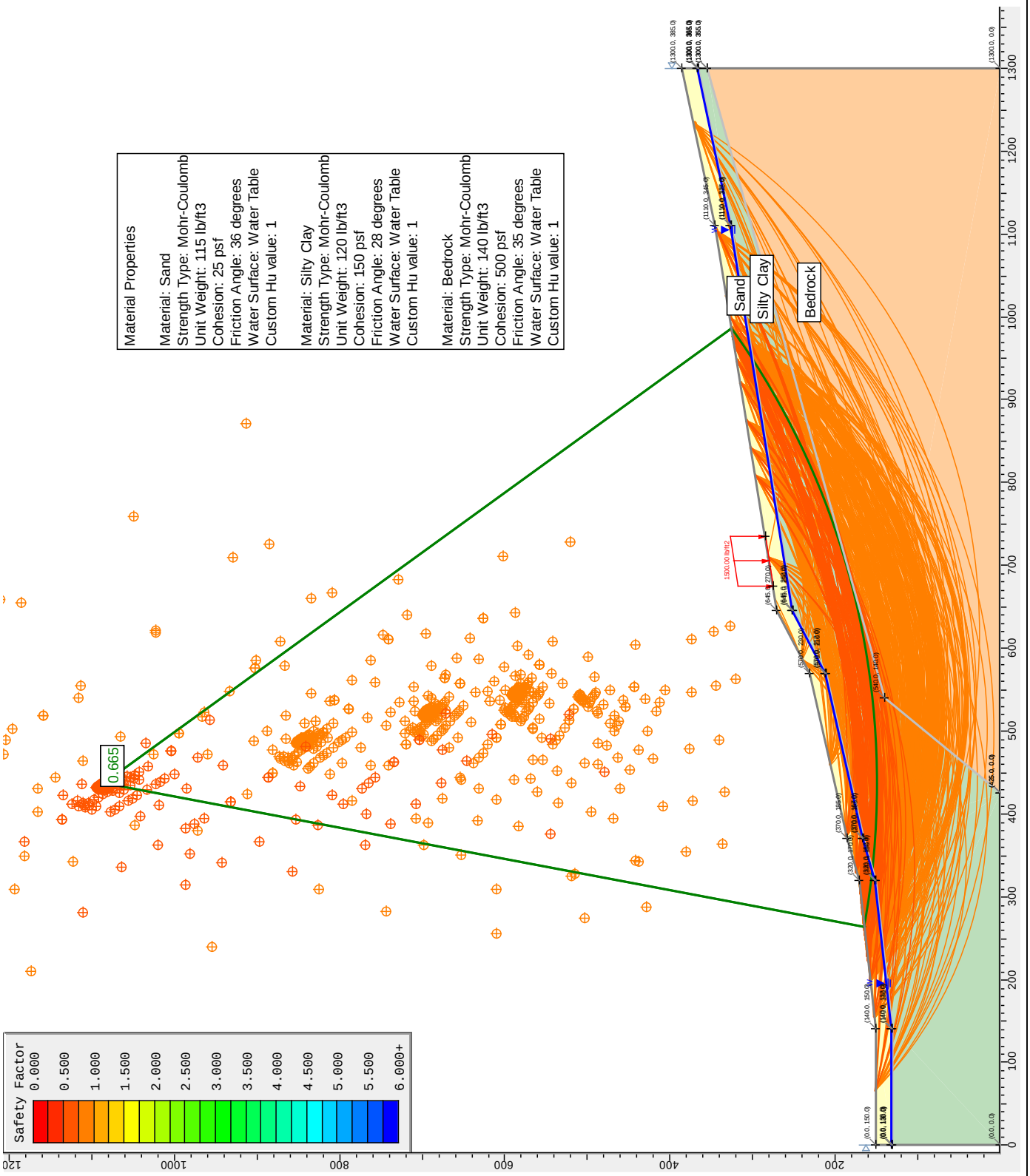


Reference Earthquake: Cape Mendocino 04/25/92 1806, Rio Dell Overpass FF, 360

PROJECT NO.: 1696-02N-15	GSH	FIGURE NO.: 12

STABILITY RESULTS

RAYMOND PROPERTY DEVELOPMENT, FRUIT HEIGHTS
(SECTION A-A' SATURATED)

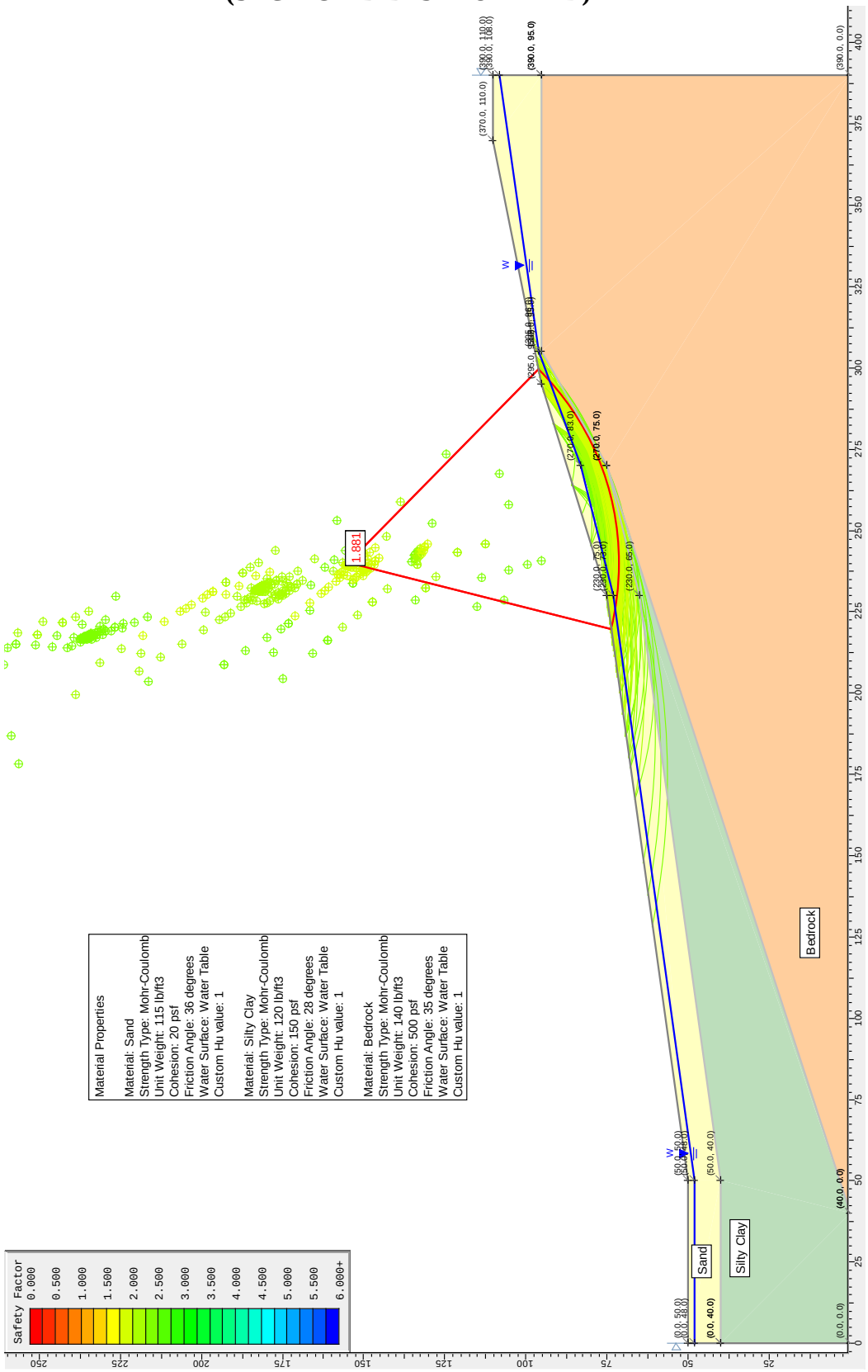


PROJECT NO.: 1696-02N-15	GSH	**FIGURE NO.:** 13

STABILITY RESULTS

RAYMOND PROPERTY DEVELOPMENT, FRUIT HEIGHTS

(SECTION B-B' SATURATED)



PROJECT NO.: 1696-02N-15	GSH	**FIGURE NO.:** 14

R E P O R T

GEOLOGIC HAZARDS EVALUATION

RAYMOND PROPERTY

ABOUT 175 SOUTH MOUNTAIN ROAD

FRUIT HEIGHTS, UTAH



Prepared for

GSH Geotechnical
1596 West 2650 South, Suite 107
Ogden, Utah 84401

August 15, 2015

Prepared by

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August 15, 2015

Andrew M. Harris, PE
Senior Geotechnical Engineer
GSH Geotechnical, Inc.
1596 West 2650 South, Suite 107
Ogden, Utah 84401

SUBJECT: Geologic Hazards Evaluation
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

Dear Mr. Harris:

This report presents results of a reconnaissance-level engineering geology and geologic hazards review and evaluation conducted by Western GeoLogic, LLC (Western GeoLogic) for the Raymond Property in Fruit Heights, Davis County, Utah (Figure 1 – Project Location). The site is northwest of the mouth of Bair Canyon in foothills at the western base of the Wasatch Range in Section 36, Township 4 North, Range 1 West (Salt Lake Base Line and Meridian). Elevation of the site ranges from 4,690 feet to about 5,100 feet above sea level. It is our understanding that the intended site use is for development of a 50-lot residential development.

PURPOSE AND SCOPE

The purpose and scope of this investigation is to identify and interpret evident surficial geologic conditions at the site and identify potential risk from geologic hazards to the Project. This investigation is intended to: (1) provide 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 to provide.

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;

- Excavation and logging of six test pits and three trenches at the property between July 13-17, 2015 to evaluate subsurface conditions and potential risk from surface faulting;

- Review of readily-available geologic maps, reports, and air photos; 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 following generally accepted professional engineering geologic principles and practice in Utah; the Guidelines for Preparing Engineering Geologic reports in Utah (Utah Section of the Association of Engineering Geologists, 1986); and in accordance with Fruit Heights City Municipal Code Title 10, Chapter 10 Sensitive Lands Evaluation & Development Standards.

HYDROLOGY

The U.S. Geological Survey (USGS) topographic map of the Kaysville Quadrangle shows no natural springs in the Project vicinity, and no active streams are mapped crossing the site. Bair Creek flows west from the Wasatch Range about 125 feet south of the southeast site corner (Figure 1) and was active at the time of our field program. The creek appears downcut about 30 feet in the area. Several inactive drainages enter the eastern boundary of the property from higher slopes, some of which continue across the upper part of the site to the southwest, although none of these drainages are mapped on Figure 1.

The subsurface hydrology in the area is dominated by the East Shore aquifer system. This aquifer system is comprised of a shallow, unconfined water table zone, and the deeper, often confined, Sunset and Delta aquifers (Feth and others, 1966). The depth to the shallow unconfined aquifer varies somewhat depending on topography and climatic and seasonal fluctuations. It is influenced by seepage from irrigation systems, and infiltration from precipitation and urban runoff. The Sunset aquifer (typical depth 250-400 feet) and Delta aquifer (typical depth 500-700 feet) provide water that generally meets the standards for public drinking water supply. Based on topography, the local groundwater flow is expected to be to the west-southwest toward Farmington Bay.

Elevation of the shallow aquifer varies somewhat based on seasonal and climatic fluctuations. Perched and confined groundwater conditions are found at the site above and below (respectively) less permeable, fine-grained lacustrine layers commonly found in the area. No groundwater was encountered in any of the borings conducted by GSH at the site to depths of from 33 to 50 feet. No groundwater was also encountered in any of the test pits or trenches conducted for this report. We anticipate the depth to groundwater to be greater than 50 feet in the area in the upper part of the site, and between 30 and 50 feet in the lower part. However, it is possible that small seasonal seeps may be found in steep slope areas that are from perched conditions.

GEOLOGY

Seismotectonic Setting

The property is located along the western base of the Wasatch Range, a major north-south trending mountain range marking the eastern boundary 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 (WFZ) at the base of the Wasatch Range. Late Cenozoic normal faulting, a characteristic of the Basin and Range, began between about 17 and 10 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 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 segments, each of which rupture independently and are capable of generating large-magnitude surface-faulting earthquakes (Machette and others, 1992). The central five segments 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 Project straddles the main trace of the Weber segment, which divides the upper and lower parts of the site.

The Weber segment of the WFZ extends for about 35 miles from the southern edge of the Plain View salient near North Ogden to the northern edge of the Salt Lake salient near North Salt Lake (Machette and others, 1992). Previous paleoseismic studies indicate four large-magnitude surface-faulting earthquakes have occurred on the Weber segment since mid-Holocene time. Nelson and others (2006) report finding evidence for four events at the Garner Canyon and East Ogden sites, including what they infer was a partial segment rupture (with 1.6 feet of displacement) around 500 years ago. This partial segment rupture was not evident at the Kaysville site of McCalpin and others (1994), although chronologic intervals for the remaining three earthquakes were similar. DuRoss and others (2009) report paleoseismic data from the 2007 Rice Creek site support a preferred scenario of six surface-faulting earthquakes in Holocene time, with four events since about 5,400 years ago, and confirm Nelson and others' (2006) partial segment rupture timing.

Lund (2005) indicates preferred earthquake timing for the last four surface-faulting earthquakes on the Weber segment is: (1) an event Z between 200 to 800 years ago (partial segment rupture) and/or between 500 and 1,400 years ago (complete segment

rupture), (2) an event Y between 2,300 and 3,700 years ago, (3) an event X between 3,800 and 5,200 years ago, and (4) an event W between 5,400 and 6,800 years ago. The consensus preferred recurrence interval for the Weber segment, as determined by the Utah Quaternary Fault Working Group, is 1,400 years for the past four surface-faulting earthquakes (Lund, 2005).

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_S 7.5 event in 1959 near Hebgen Lake, Montana. However, 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 of these events was the 1934 Hansel Valley (M_S 6.6) event north of the Great Salt Lake.

Surficial Geology

The site is located within the Wasatch Front Valley System, a deep sediment-filled, structural basin flanked by the Wasatch Range to the east and the Lakeside Mountains to the west. The Project is below the Bonneville shoreline of Lake Bonneville, but straddles the Provo shoreline. Surficial geology of the site is mapped by Solomon (2007) as mainly lacustrine sand, silt, and gravel from Pleistocene Lake Bonneville (units Qlsp and Qlgb) and post-lake coalesced alluvial-fan deposits (units Qaf1, Qafy, and Qafp; Figure 2). Small areas of Holocene to upper Pleistocene alluvium and colluvium (unit Qac) and landslide deposits (units Qms1 and Qmsy) are also mapped along the main trace of the Weber segment and extending into the southeast boundary of the site (Figure 2). Solomon (2007) maps the main, west-dipping trace, and an east-dipping antithetic trace of the Weber segment dividing the proposed development (Figure 2). Most of the development is on the upthrown side of the main fault trace; one lot and a proposed detention basin are on the downthrown fault side.

Solomon (2007) describes surficial units in the site vicinity, from youngest to oldest in age, as follows:

> ***Qms1 -- Landslide deposits, unit 1 (Historical to Middle Holocene)***. Very poorly sorted clay, silt, sand, and minor gravel; grain size and texture varies with the nature of the deposits in the source area; unit 1 landslides occur in bluffs bordering the Weber River flood plain and north of Kaysville along steep bluffs incised by drainages following regression of Lake Bonneville; characterized by moderately fresh scarps and hummocky topography, with freshest scarps in areas of historical movement. Most unit 1 landslides along the Weber River originate in sandy Lake Bonneville deltaic deposits, form gently sloping alluvial-fan shaped deposits overlying unit 2 landslide deposits on river terraces that fringe the bluffs, and apparently result from flow failure, perhaps liquefaction induced, of the bluff face; one landslide along the Weber River, the Cedar Bench landslide, is an earth slide

resulting from fill failure on the slope bordering an irrigation pond (Solomon, 1999). Landslides along other drainages are found at and slightly below the gradational contact between overlying Lake Bonneville sand and silt and underlying silt and clay and are the result of a combination of rotational and translational earth slides; parts of several of these landslides have been historically reactivated, resulting in damage to roads and houses. Thickness of the deposits is highly variable.

Qmsy – Younger landslide deposits, undivided (Historical to Upper Pleistocene). Very poorly sorted, clay to boulder-size gravel in a matrix of silt, sand, clay, and pebbles; grain size and texture varies with the nature of the deposits in the source area. Younger landslide deposits are found near Adams, Hobbs, and Holmes Reservoirs and along the Wasatch Range front; deposits along the steep bluffs near the reservoirs are derived from Lake Bonneville deposits incised by drainages following regression of Lake Bonneville and resemble unit 1 landslide deposits but lack relatively fresh scarps; younger landslide deposits along the range front are derived from the Farmington Canyon Complex (Xfc) and either result from incision of drainages after Lake Bonneville regression, obscure the Bonneville shoreline, or overlie Lake Bonneville deposits; younger landslide deposits may include unit 1 and unit 2 landslide deposits that are undifferentiated because units complexly overlap; thickness of the deposits is highly variable.

Qaf1 – Modern (level 1) alluvial-fan deposits (upper Holocene). Poorly to moderately sorted, weakly to non-stratified, silt- to boulder-size sediment deposited principally by debris flows and debris floods at the mouths of mountain-front canyons; upper parts characterized by abundant boulders and debris-flow levees that radiate away from the fan apex; small, finer grained alluvial-fan deposits are found on gentle slopes in the northern part of the quadrangle, deposited by shallow, dispersive flows; equivalent to the younger part of Qafy but differentiated where deposits can be mapped separately; commonly overlies lacustrine and older alluvial-fan deposits; debris-flow and debris-flood hazards are highest on modem alluvial fans, their undivided equivalents within younger alluvial fans (Qafy), and debris-flow deposits (Qmf); less than 30 feet (9 m) thick.

Qac – Alluvial and colluvial deposits, undifferentiated (Holocene to upper Pleistocene). Poorly to moderately sorted, generally poorly stratified, clay- to boulder-size, locally derived sediment deposited in drainages along the Wasatch Range front by fluvial, rockfall, slopewash, and creep processes; generally less than 30 feet (9 m) thick.

Qc – Colluvial deposits (Holocene to upper Pleistocene). Poorly to moderately sorted, angular, clay- to boulder-size, locally derived sediment deposited by rock fall, slopewash, and soil creep on moderate to steep mountain-front spurs above the Bonneville shoreline; most bedrock is covered by at least a thin veneer of colluvium, and only the larger, thicker deposits are mapped; maximum thickness about 20 feet (6 m).

Qafy – Younger alluvial-fan deposits, undivided (Holocene to upper Pleistocene). Poorly to moderately sorted, weakly to non-stratified, silt- to boulder-size sediment deposited principally by debris flows, debris floods, and streams; equivalent to modern and level 2 alluvial-fan deposits (Qaf1, and Qaf2.) that are undifferentiated because units are complexly overlapping or too small to show separately; also mapped where the age of Holocene alluvial-fan deposits is uncertain; upper parts of fans are locally deeply incised; occurs near canyon mouths along the mountain front; probably less than 40 feet (12 m) thick.

Qafp – Alluvial-fan deposits related to the Provo (regressive) phase of the Bonneville lake cycle (upper Pleistocene). Poorly to moderately sorted, silt- to cobble-size sediment, with local boulders, deposited principally by debris flows whose surfaces are graded to the Provo shoreline at the mouth of Bair Canyon; incised by active streams; probably less than about 40 feet (12 m) thick.

Qlsp - Lacustrine sand and silt related to the Provo (regressive) phase of the Bonneville lake cycle (upper Pleistocene). Fine- to coarse-grained lacustrine sand and silt with minor gravel deposited at and below the Provo shoreline; locally overlain by a thin veneer of eolian sand; grades downslope into fine-grained Lake Bonneville deposits and laterally northward to sandy deltaic deposits (Qldp); typically thick bedded and well sorted; gastropods locally common; deposited in relatively shallow water in beaches and, at the mouths of larger canyons, in deltas that no longer retain distinctive morphology; as much as 50 feet (15 m) thick.

Qlgb – Lacustrine gravel and sand related to the Bonneville (transgressive) phase of the Bonneville Lake cycle (upper Pleistocene). Moderately to well-sorted, moderately to well-rounded, clast-supported, pebble to cobble gravel interbedded with lesser amounts of pebbly sand and clean sand, deposited between the Bonneville and Provo shorelines; thin to thick bedded; gastropods locally common in sandy lenses; locally partly cemented with calcium carbonate; typically forms a bench along the Bonneville shoreline near the base of the Wasatch Range; locally as much as 200 feet (60 m) thick.

Xfc – Farmington Canyon Complex (Early Proterozoic). Metamorphic and igneous rocks, not mapped in detail for this study; Bryant (1984) concluded that these rocks in the Kaysville quadrangle record a Late Archean metamorphic event, but Barnett and others (1993) and Nelson and others (2002) found evidence that metamorphism was Early Proterozoic, affecting rocks either initially deposited as Archean sediment or Early Proterozoic sediment incorporating material eroded from nearby Archean crust; Yonkee and Lowe (2004) supported this interpretation and provided further interpretations based on detailed mapping of the Farmington Canyon Complex in the adjacent Ogden 7.5-minute quadrangle.

References included in the above unit descriptions are not provided in this report, but are provided in Solomon (2007).

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 underlie the site and site vicinity.

Timing of events related to the transgression and regression of Lake Bonneville is indicated by calendar age estimates of significant radiocarbon dates in the Bonneville Basin (Donald Currey, University of Utah; written communication to the Utah Geological Survey, 1996; and verbal communication to the Utah Quaternary Fault Parameters Working Group, 2004). Approximately 32,500 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, after about 18,000 years ago. During the transgression and highstand, major drainages that emanate from within the Wasatch Range (such as the Weber River) formed large deltaic complexes in the lake at their canyon mouths. The lake remained at its highest level until 16,500 years ago, when headward erosion of the Snake River-Bonneville basin drainage divide caused a catastrophic incision of the threshold and the lake level lowered by roughly 360 feet in fewer than two months (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. 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. Drainages that fed Lake Bonneville began downcutting through stranded deltaic complexes and near-shore deposits as the lake receded from the Provo shoreline, including the alluvial fan at the mouth of Bair Canyon. Great Salt Lake 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).

SITE CHARACTERIZATION

Air Photo Observations

A 1937 aerial photograph available from the U.S. Geological Survey (frame 10-AAK3-41, Figure 3) was reviewed to obtain information about the geomorphology of the site and surrounding area. Figure 4 also shows a site plan at a scale of 1 inch equals 100 feet on a 2012 Utah AGRC high-resolution digital orthophoto. The main trace of the Weber

segment of the WFZ is evident on Figure 3 crossing the western part of the property, corresponding to the steep escarpment dividing the upper and lower portions of the site. Trenching evidence (discussed below) indicates that this escarpment includes several west-dipping faults, which appear to converge northward and southward (Figure 3). A weak lineament is also evident on Figure 3 that corresponds to the antithetic fault observed in trench 3. We note that Solomon (2007; Figure 2) maps the main fault further west in the area south of trench 2, and the antithetic fault is mapped further east than found in trench 3. Solomon's (2007) fault locations do not appear to correspond to any topographic features. We also note that the slope failure east of the site (discussed above and mapped on Figure 2) is not evident on the air photo. This suggests that the failure is historical (post-1937).

Several small inactive drainage swales are also evident on Figure 3 entering the eastern part of the property. The two northernmost drainages head slightly above the Bonneville shoreline and have small associated alluvial fans at their mouths (labeled 1 and 2, Figure 3). A third drainage swale is to the south that heads below the shoreline and also has a small alluvial fan (labeled 3, Figure 3). The fan size and morphology for all three drainages decreases southward. Further southward, the remaining drainages entering the site appear to be small erosional gullies that head below the Bonneville shoreline (Figure 3). These drainages have no associated alluvial fans, and many continue to the southwest across the site. All the ephemeral drainages entering the site are likely only active following cloudburst storms and spring snowmelt, and may not be consistently active.

Empirical Observations

On July 14, 2015, Mr. Bill D. Black of Western GeoLogic conducted a reconnaissance of the property and immediate vicinity. Weather at the time of the site reconnaissance was clear with temperatures in the 90's (°F). The site is on west-facing slopes at the base of the Wasatch Range northwest of the mouth of Bair Canyon. Vegetation at the site consists of fruit orchards and native trees, scrub oak, sage brush, and grasses

The property is generally on two gently sloping surfaces separated by a steep west-dipping escarpment formed by displacement along the Weber segment of the WFZ; Figure 4 shows most of the proposed development is on the upper surface east of the escarpment. Slope gradients vary across the site and exceed 30 percent in some areas, including along the fault escarpment and slopes bordering the Project to the east. The escarpment north of trench 1 showed evidence for rotational slumping in interbedded lacustrine sand and silt deposits, and Solomon (2007) maps a small landslide further north in the escarpment on Figure 2 in the northwest corner of the property. We consider the entire escarpment to be an active mass-wasting zone mitigated only by heavy tree cover. Solomon (2007) also maps a landslide northeast of the southeast site corner and a low arcuate scarp from this failure was evident at the surface. However, deposits from the failure do not extend onto the site (although they are mapped as such on Figure 2). The landslide appeared to be a broad, shallow rotational failure sourced in lacustrine deposits overlying weathered schist.

No active streams cross the site, but two drainage swales were observed east of the site that appeared capable of generating small debris flows. Such flows would likely initiate at or above the Bonneville shoreline and entrain lacustrine deposits below the shoreline. Some large boulders up to about three feet across were evident embedded in the surficial sediment veneer, but no evidence of debris flow levees or recent flows was observed at the drainage mouths. Scattered weathered outcrops of schist of the Precambrian Farmington Canyon Complex were also observed in slopes east of the property and in one area in the southeast part of the site near test pit 5. No boulders from rock falls were observed at the property, and upslope bedrock outcrops appeared to have low relief and likely incapable of generating large rock fall boulders due to their composition. No evidence of other geologic hazards was observed.

Prior Evaluations

Great Basin Earth Science (GBES) conducted a previous geologic hazards evaluation for the proposed Cherry Ridge development in 2008. The present site constitutes part of the former Cherry Ridge development. The GBES (2008) study included excavation and logging of twelve test pits and twelve trenches at the site to evaluate subsurface conditions and risk from surface faulting, as well as surficial observations and calculations of potential debris flow sediment volumes.

With regard to surface faulting, GBES (2008) found no evidence for active faults in the trenches excavated at the site. GBES (2008) trench locations are shown on Figure 4. GBES (2008) inferred the main trace of the Weber segment was located in the lower half of the west-facing escarpment, although no trenching was conducted to confirm its location. GBES (2008) instead recommended a broad setback area extending east and west from the escarpment toe and crest. Six trenches (A through F, Figure 4) were excavated to expose the antithetic fault to the west, but found no evidence for the fault. However, we note that: (1) the antithetic fault was found in our trench 3 further west than mapped, and (2) GBES' (2008) trench E encountered fill from an onsite septic system in this area that would have complicated recognition of the antithetic fault. The antithetic fault was also not recognized in GBES' (2008) trench D, but may correspond to the abrupt down-to-the-east inflection in the unit II upper contact between stations 80 and 85 feet.

With regard to landslide hazards, GBES (2008) identified three areas of landsliding in the fault escarpment. Two of these areas correspond to our observations north of trench 1, whereas the third was queried and located in the area of our trench 2. This latter failure does not exist given the lack of evidence for landsliding in trench 2. GBES (2008) did not believe that the slope failure east of the site mapped by Solomon (2007) existed.

With regard to debris flows, GBES (2008) concluded a debris flow hazard existed on their mapped alluvial fans A, B, and E. They indicate the remaining fans showed no evidence for Holocene activity, based on surface observations and test pit data. Fans A and B are north of the Project and do not impact this site, whereas fan E corresponds to our alluvial fan 2 on Figure 4. GBES (2008) estimated a design magnitude debris volume of 1,400 cubic yards for the fan E [2] drainage using empirical bulking.

With regard to rock fall hazards, GBES (2008) did not identify any potential hazard areas within the Project boundaries. Two areas were identified north of the Project that do not impact the site.

Subsurface Investigation

On July 13, 2015 six test pits were excavated at the site with a large trackhoe to evaluate subsurface conditions. Test pit locations are shown on Figure 4. Logs of the test pits at a scale of 1 inch equals 5 feet are shown on Figures 5A through 5C. Test pit 1 was excavated within alluvial fan 1 (Figure 4) and found evidence for a sequence of post-Lake Bonneville debris flow and flood deposits displaying two paleosol A horizons (units 2 through 4, and units 2A and 3A; Figure 5A). Unit 4 was overlain by fill from prior site disturbance. Discounting the surface fill, the alluvial units have thicknesses ranging between 0.4 to 2.3 feet, and an average thickness of about 1.6 feet. This evidence confirms a debris flow hazard exists on alluvial fan 1 on Figures 3 and 4. Test pits 2 through 4 exposed post-lake alluvium overlying Lake Bonneville sand and gravel deposits, but no evidence for paleosols or discrete debris flows (Figures 5A-B). Test pit 5 exposed about 3 feet of sandy slope colluvium overlying fractured schist bedrock (Figure 5C). Shallow bedrock in this test pit was anticipated given an outcrop is located nearby to the west. Test pit 6 exposed only lacustrine sand and gravel overlying interbedded sand and silt (Figure 5C).

To evaluate the hazard from surface faulting, three trenches were also excavated at the Project using a large trackhoe. Trench locations are shown on Figure 4. Trench 1 extended across lot 2 an overall S57°W for a distance of 149 feet, and generally covered the area between the crest of the steep escarpment to the east edge of the Surface Fault Rupture Special Study Area (SFRSSA). Trench 2 was located about 140 feet south of trench 1, and extended across the escarpment an overall S80°W for a distance of 140 feet. Trench 3 was located about 270 feet south of trench 2, and extended sinuously across lot 50 for a total distance of 255 feet. Trench 3 was excavated in this fashion to avoid following GBES'(2008) trench E and encountering fill from the onsite septic system exposed in that trench. The trenches were excavated to a depth sufficient to expose evidence for active faulting. Trench logging followed methodology in McAlpin (1996). Trench and active fault locations were surveyed by Reeves & Associates on July 17, 2015. The trenches were reviewed by Tim Thompson, P.G. of GeoStrata, Fruit Heights City contract geologic reviewer, also on July 17, 2015. No significant complications were encountered that impacted our findings.

Trench 1 exposed a coarsening-upward sequence of lacustrine silt, sand, and gravel (units 1a, 1b, and 1c), overlain by gravelly to sandy post-lake alluvium (unit 2) in which the modern A-horizon soil was forming (Figures 6A-B). Unit 1b was observed to contain a basal cobbly zone that coarsened westward. The stratigraphic sequence exposed in the trench appeared similar to that exposed in test pits 2 and 3 (Figures 5A-B), with the addition of unit 1a underlying the lacustrine sand and gravel (Figures 6A-B). No evidence for active faulting was exposed in the trench.

Trench 2 exposed lacustrine silt with thin fine sand interbeds (unit 1a) in the east trench end overlain by a thin veneer of alluvium (unit 2), both of which are displaced down-to-the-west across a series of three faults (F1, F2a, and F2b; Figure 7A). Fault F1 displays 3.0 feet of offset in correlative bedding, fault F2a displays 0.5 feet of offset, and fault F2b displays more than 25 feet of offset. No correlative stratigraphy was observed across fault F2b, which we infer is the main trace of the Weber segment of the WFZ. Unit 1a lies beneath the lacustrine sequence exposed in trench 1. However, the sequence exposed in trench 1 is eroded and not present in the footwall of the main fault. We infer units 1a-1c in trench 1 to correlate to units 1b and 1c in trench 2, and unit 2 is correlative between the trenches. Unit 1a in the footwall of trench 2 is at depth and no longer exposed in the hanging wall, but the overlying sequence is exposed (Figure 7A). Further west the lacustrine sequence is displaced down-to-the-west across three additional faults: (1) fault F3 with 3.5 feet of offset, (2) fault F4 with 2.6 feet of offset, and (3) fault F5 with 3.5 feet of offset (Figures 7B-C). Five colluvial wedges from fault scarp degradation are exposed on the downthrown sides of faults F1, F2a, F3, F4, and F5 (units 3 through 7; Figures 7A-C). The fault scarp colluvium is overlain by alluvium and colluvium in which the modern A-horizon soil is forming (unit 8 and 8A, Figures 7A-C).

Trench 3 exposed lacustrine sand with pea gravel (unit 1) resembling unit 1c in trench 1 west of station 0+82 feet (Figures 8B-8D). This unit is overlain by a sequence of three Holocene alluvial deposits (units 2 through 4, Figures 8A-8D). Backfill from GBES' (2008) trench E was encountered between stations 1+30 and 1+40 feet, and an area of fill with pipes was encountered at the location of the onsite septic system. However, west of this area the stratigraphic sequence is displaced 1.6 feet down to the east across fault AF1. Recognition of this fault would have been difficult in GBES' (2008) trench E due to the onsite septic system.

No other evidence for faulting was observed in the trenches excavated for this study, though the exposed sediments were of sufficient age to show evidence for active faulting.

Cross Sections
Figure 9 shows two cross sections across the slopes in the central and south parts of the site. Cross section A-A' is at a scale of 1 inch equals 100 feet, whereas B-B' is at a scale of 1 inch equals 50 feet. The cross sections have no vertical exaggeration. Cross section locations are shown on Figure 4. Unit contacts and dips are inferred the test pit and trenching data, GSH boring logs, and site observations.

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 to the proposed development. A "moderate" hazard rating (M) indicates a hazard that poses an equivocal risk or only impacts a small portion of the development. 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. High and moderate-risk hazards may require further studies or mitigation, whereas 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 for the Raymond Property.*

Hazard	H	M	L	…Hazard Rating
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	
Radon	X			
Problem Soil			X	

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. Mapped active faults within this distance include: the East and West Cache fault zones; the Brigham City, Weber, Salt Lake, and Provo segments of the Wasatch fault zone; the East Great Salt Lake fault zone; the Morgan fault; the West Valley fault zone; the Oquirrh fault zone; and the Bear River fault zone (Black and others, 2003).

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 (Costa and Baker, 1981). Peak ground, 0.2 second spectral, and 1.0 second spectral accelerations (percent of gravity, %g) at the site with 10% and 2% probabilities of exceedance in 50 years are estimated in Frankel and others (2002) as follows:

41.033483° N, -111.899899° W	**10% PE in 50yr**	**2% PE in 50yr**
PGA	20.32	61.71
0.2 sec SA	49.42	142.85
1.0 sec SA	17.50	59.27

Given the above information, earthquake ground shaking is a high risk to the site. The hazard from earthquake ground shaking can be adequately mitigated by prudent design and construction.

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.

Trench 2 at the site exposed evidence for five west-dipping traces associated with faulting on the Weber segment of the Wasatch fault zone. Trench 3 also exposed one east-dipping antithetic fault trace. Fault locations are shown on Figure 4, based on trenching and air photo evidence. We infer total displacement across faults F1 through F5, based on relative elevations of the upper contact of the lacustrine sequence in trenches 1 and 2 (4,785 feet to 4,735 feet), to be a total of about 50 feet. Net displacement across the entire fault zone is between 45 and 50 feet (Figure 9, cross section A-A'). The fault displacement likely represents multiple post-Lake Bonneville events. DuRoss and others (2009) prefer a scenario of six Holocene events, which would give an average per-event displacement of about 8 feet.

Given the above, the risk from surface faulting is high at the site. Based on the results of this investigation and our current understanding that surface fault rupture and deformation tend to follow past patterns, we recommend a non-buildable (setback) zone around the fault traces as shown on Figure 4 (shaded in red). Table 2 shows relevant data and calculated setbacks for each fault. Setback distances are based on guidelines in Christenson and others (2003).

The fault setback for the downthrown block side of the fault is calculated using the following formula:

$$S = U (2D + F/\tan\Theta)$$

where:

S = Setback distance from active faults

U = Criticality factor (2.0 for IBC class R residential structures)

D = Fault displacement per event in feet (assumed to be 8 feet if not measurable)

F = Maximum depth of footing or subgrade portion of the building (assumed to be 10 feet)

Θ = Dip of the fault

The fault setback for the upthrown block side of the fault was calculated using:

$$S = U (2D)$$

Table 2. *Fault data and setback calculations.*

Trench	Fault	Station	Dip	Offset (feet)	Dip Direction	Calculated Downthrown Setback	Calculated Upthrown Setback	Downthrown Setback Distance	Upthrown Setback Distance
2	F1	0+7.2	52	3.0	West	20.7	9.0	20.7	15.0
2	F2a	0+15.8	55	0.5	West	12.0	1.5	15.0	15.0
2	F2b	0+17.7	61	8.0	West	32.3	24.0	32.3	24.0
2	F3	0+51.3	63	3.5	West	18.1	10.5	18.1	15.0
2	F4	0+61.6	58	2.6	West	17.2	7.8	17.2	15.0
2	F5	1+4.3	70	3.5	West	16.0	10.5	16.0	15.0
3	AF1	2+18.7	59	1.6	East	13.8	4.8	15.0	15.0

Minimum fault setback is 15 feet where calculated distances are less than 15 feet (Table 2). No structure intended for human occupancy should be located in the setback zones 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.

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 (earthquake ground shaking) accelerations, groundwater conditions, and presence of susceptible soils.

Sandy lacustrine deposits possibly susceptible to liquefaction are present in the upper 30 feet of the site subsurface, and the site is in an area of potentially strong shaking (as discussed in the Earthquake Ground Shaking Section above). However, groundwater at the site appears to be greater than at least 30 feet deep, and possibly more than 50 feet deep in the upper portion of the development. Based on the above, we rate the hazard from liquefaction as low, although the risk may vary locally depending on factors such as perched groundwater.

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 a normal fault. The main trace of the Weber segment of the WFZ divides the upper and lower parts of the site. Lots 1 through 49 in the upper part of the development are on the upthrown fault side and therefore have a low risk from tectonic deformation. However, lot 50 is adjacent to the downthrown fault side and could experience a few degrees of tilting in a large-magnitude surface-faulting earthquake on the Weber segment. Tectonic deformation is typically not a life safety issue, but could cause damage that requires expensive repairs such as tilted foundations and altered sewer and water-line gradients.

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, the risk to the subject property from seismic seiches is rated as low.

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 cross the site or were evident during our reconnaissance. However, several ephemeral drainages enter the site from higher slopes to the east, some of which continue across the property to the southwest. These drainages are likely only active following cloudburst storms and spring snowmelt and may not be consistently active, but could cause produce small flash floods. We therefore rate the hazard from stream flooding as moderate. Site hydrology and runoff should be therefore addressed in the civil engineering design and grading plan for the Project.

Shallow Groundwater
No springs are shown on the topographic map for the Kaysville quadrangle at the site and no springs were observed during our site reconnaissance. No groundwater was encountered in any of the borings conducted by GSH at the site to depths of from 33 to 50 feet. No groundwater was also encountered in any of the test pits or trenches conducted for this report. We anticipate the depth to groundwater to be greater than 50 feet in the area in the upper part of the site, and between 30 and 50 feet in the lower part. However, it is possible that small seasonal seeps may be found in steep slope areas that are from perched conditions. Given the above, we rate the risk from shallow groundwater as low, although the risk may vary locally depending on factors such as perched groundwater.

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.

The steep west-facing escarpment formed by faulting along the Weber segment of the WFZ, which divides the proposed development, is an active mass-wasting zone prone to landsliding. At least two landslide areas were observed or mapped north of trench 1, as shown on Figure 4. These slide areas are in portions of lots 2 through 6. Similar landslide-prone slopes also border lots 46 through 49. The risk from landsliding is high to these areas of the proposed development. Lots 18 through 27 also border steep slopes on the east that may be prone to instability given the landslide mapped and observed east of the site. This slide appears to be a historical failure. Based on surficial observations and subsurface data from test pit 5, the landslide does not appear to extend into the proposed development.

Given all of the above, we rate the risk from landsliding at the site as high. We recommend stability of the slopes be evaluated in a geotechnical engineering evaluation prior to building based on site specific data and subsurface information included in this report. Recommendations for reducing the risk from landsliding should be provided if factors of safety are determined to be unsuitable. Care should also be taken that site grading does not destabilize slopes in this area without prior geotechnical analysis and grading plans, that proper drainage is maintained, and no non-engineered cuts are made in slope toes.

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.

Several small inactive drainages enter the eastern part of the property. The two northernmost drainages head slightly above the Bonneville shoreline and have small associated alluvial fans at their mouths (labeled 1 and 2, Figures 3 and 4). A third drainage swale is to the south that heads below the shoreline and also has a small alluvial fan (labeled 3, Figures 3 and 4). The fan size and morphology for all three drainages decreases southward. Further southward, the remaining drainages entering the site appear to be small erosional gullies that head below the Bonneville shoreline (Figures 3 and 4). These drainages have no associated alluvial fans, and some continue to the southwest across the property. Based on surface observations and test pit data, GBES (2008) concluded a debris flow hazard existed at the current proposed development only on corresponding alluvial fan 2 on Figure 4. Our surface observations and test pit data also indicate a hazard also exists on alluvial fan 1. Alluvial fan 3 does not appear to have any evidence for Holocene activity, and is the smallest of the three with the weakest morphology.

Figure 1 shows the drainage basin areas and estimated drainages for alluvial fans 1 through 3, along with their estimated area and drainage lengths. Drainages 1 and 2 are assumed to head at the Bonneville shoreline for purposes of calculating the drainage length. Drainage basins 1 and 2 are 6.4 and 12.6 acres in size (Figure 1), which would be

below the statistical threshold for volume estimates using VanDine's (1996) drainage basin area correlation. However, Hungr and others (1984) provides a correlation to estimate design magnitude debris flow volumes based on drainage length (empirical bulking). Based on our observations, the drainages are in loose lacustrine sediments and soil over weathered schist bedrock, which would be a Hunger and others' (1984) Channel Type B drainage with a corresponding sediment bulking factors of 2-4 yd^3/ft. Giraud (2005) confirms bulking rates for intermittent and ephemeral streams are generally lower than rates for perennial streams. Similar ephemeral drainages have showed bulking rates of 1.5 to 5 yd^3/ft (Mulvey and Lowe, 1992; McDonald and Giraud, 2002). We believe an average bulking rate of 3 yd^3/ft is appropriate. Drainage lengths for basins 1 and 2 (Figure 1) are 596 feet and 669 feet, respectively. At a bulking rate of 3 yd^3/ft, the drainages would have design magnitude debris flow volumes of 1,788 and 2,007 cubic yards. GBES (2008) estimated a design magnitude debris flow volume for alluvial fan 2 of 1,400 cubic yards based on a drainage length of 700 feet and a bulking rate of 2 yd^3/ft.

Based on the above, we rate the hazard from debris flows as moderate. Given the small calculated volumes and assuming a design flow is further attenuated by deposition in other areas, it appears likely that only a small amount of debris and muddy water would reach lots 14 through 17. Lot 18 appears to have the highest risk since it occupies a substantial area of alluvial fan 2 and is closer to the fan mouth. Recommendations to reduce the hazard from debris flows and floods to specific lots at the site should be provided in the civil engineering design for the development, and may include raising building pads, eliminating upslope below-grade entryways, grading routing channels and berms to direct debris and water away from homes, or a combination of the above.

Rock Fall

Scattered weathered outcrops of schist of the Precambrian Farmington Canyon Complex were observed in slopes east of the property and in one area in the southeast part of the site near test pit 5. However, no boulders from rock falls were observed at the site and upslope outcrops appeared to have low relief and likely incapable of generating large rock fall boulders due to their composition. Based on the above, we rate the hazard from rock falls as low. GBES (2008) also did not identify any potential rock fall hazard areas within the Project boundaries.

Radon

Radon comes from the natural (radioactive) breakdown of uranium in soil, rock, and water and can seep into homes through cracks in floor slabs or other openings. The site is located in areas of "Moderate" and "High" radon-hazard potential (Black and Solomon, 1996). A moderate hazard potential indicates geologic factors are favorable for indoor radon concentrations between 2 and 4 picocuries per liter of air, whereas a high hazard potential indicates indoor-radon concentrations may exceed 4. Actual indoor radon levels can be affected by non-geologic factors such as building construction, maintenance, and weather. Long-term indoor testing following construction is the best method to characterize the radon hazard and determine if mitigation measures are required.

Swelling and Collapsible Soils

Surficial soils that contain certain clays can swell or collapse when wet. Given the subsurface soil conditions observed at the site, we do not anticipate that any soils

susceptible to swelling or collapse will be present. However, a geotechnical engineering evaluation should be performed to address soil conditions and provide specific recommendations for site grading, subgrade preparation, and footing and foundation design

CONCLUSIONS AND RECOMMENDATIONS

Geologic hazards posing a high relative risk to the site are earthquake ground shaking, surface fault rupture, landslides, and radon. Moderate-risk hazards include tectonic deformation, stream flooding, and debris flows. The following recommendations are provided to address these hazards:

- Proposed homes should be designed and constructed to current seismic standards to reduce the potential ground-shaking hazard.

- No structures intended for human occupancy should be located in the fault setback zones delineated 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.

- A design-level geotechnical engineering study should be conducted prior to construction to: (1) address soil conditions at the site for use in foundation design, site grading, and drainage; (2) provide recommendations regarding building design to reduce risk from seismic acceleration; and (3) evaluate stability of slopes at the site, including providing recommendations for reducing the risk of landsliding if the factors of safety are deemed unsuitable.

- Site grading and drainage should be addressed in the civil engineering design for the development, including providing recommendations to reduce risk from debris flows and floods to lots 14 through 18.

The site appears suitable for the proposed development given the scope of this report and findings herein, and assuming our recommendations provided above are followed.

Availability of Report

The report should be made available to architects, building contractors, and in the event of a future property sale, real estate agents and potential buyers. This 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.

LIMITATIONS

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.

It has been a pleasure working with you on this project. Should you have any questions, please call.

Sincerely,
Western GeoLogic, LLC



Bill. D. Black, P.G.
Senior Engineering Geologist

Reviewed by:



Craig V. Nelson, P.G.
Principal Engineering Geologist

ATTACHMENTS

Figure 1. Location Map (8.5”x11”)
Figure 2. Geologic Map (8.5”x11”)
Figure 3. 1937 Air Photo (11”x17”)
Figure 4. Site Plan (22”x25.5”)
Figures 5A-C. Test pit Logs (three 11”x17” sheets)
Figures 6A-B. Trench 1 Log (two 11”x17” sheets)
Figures 7A-C. Trench 2 Log (three 11”x17” sheets)
Figures 8A-D. Trench 3 Log (four 11”x17” sheets)
Figure 9. Cross Sections (one 11”x17” sheet)

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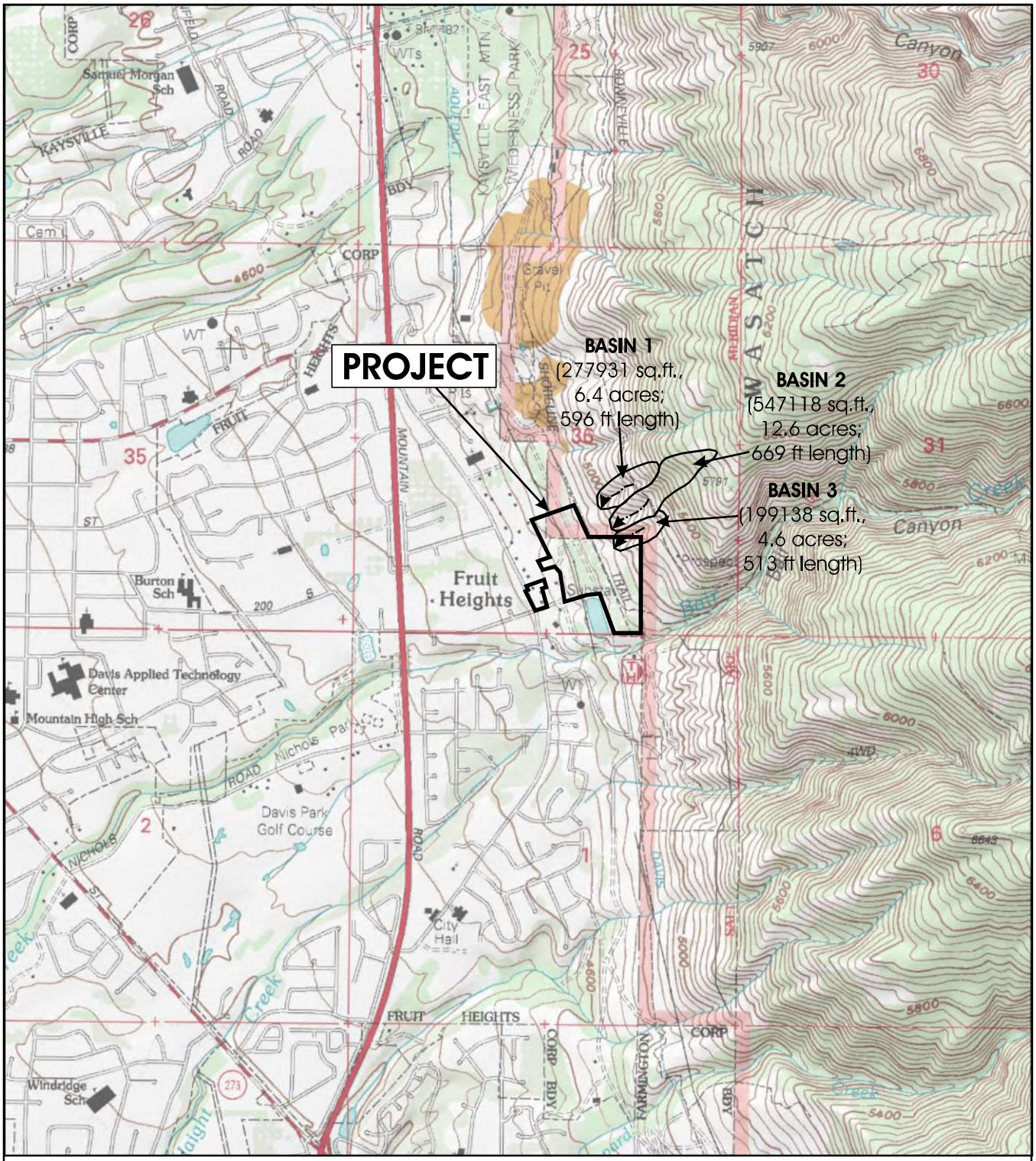
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Source: U.S. Geological Survey 7.5 Minute Series Topographic Maps, UT - Kaysville, 1998.

WESTERN GEOLOGIC

N
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S

0 1000 2000 feet

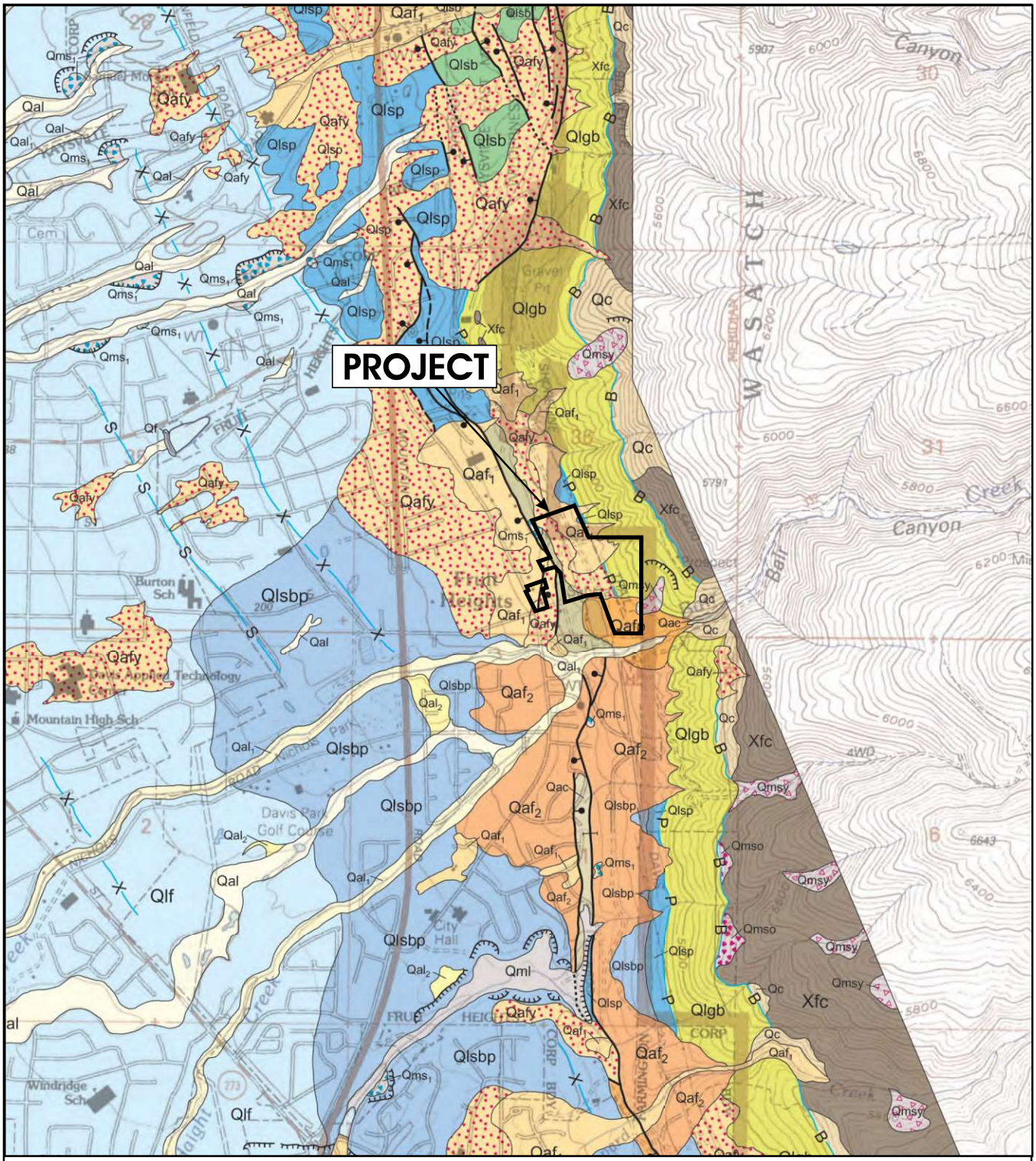
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(1 inch = 2000 feet)

LOCATION MAP

GEOLOGIC HAZARDS EVALUATION

Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 1



Source: Solomon, 2007.

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0 1000 2000 feet

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

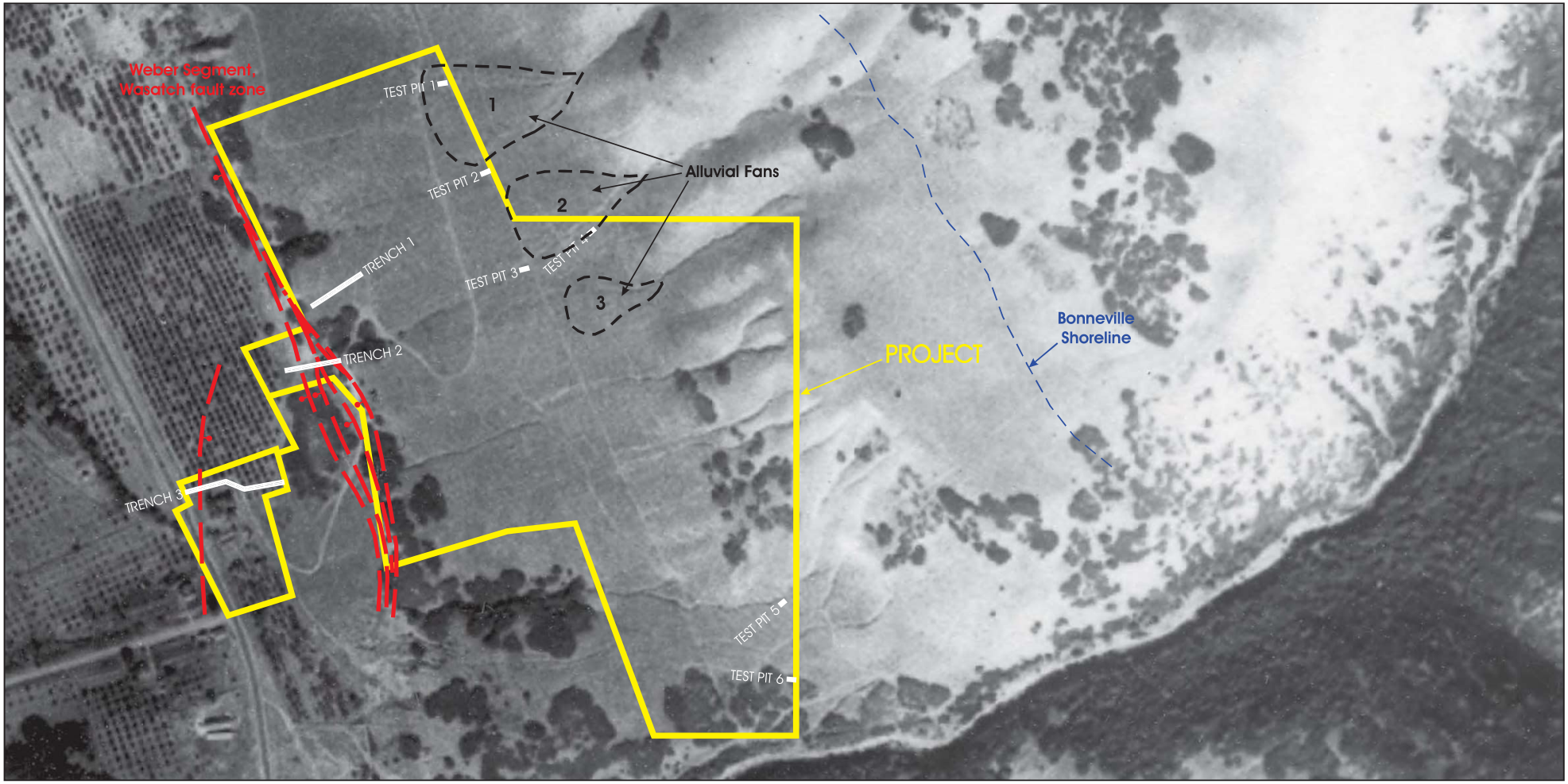
WESTERN GEOLOGIC

GEOLOGIC MAP

GEOLOGIC HAZARDS EVALUATION

Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 2



Air photo from U.S. Geological Survey, frame 10-AAK3-41, original scale 1:20,000.

N
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0 125 250 feet

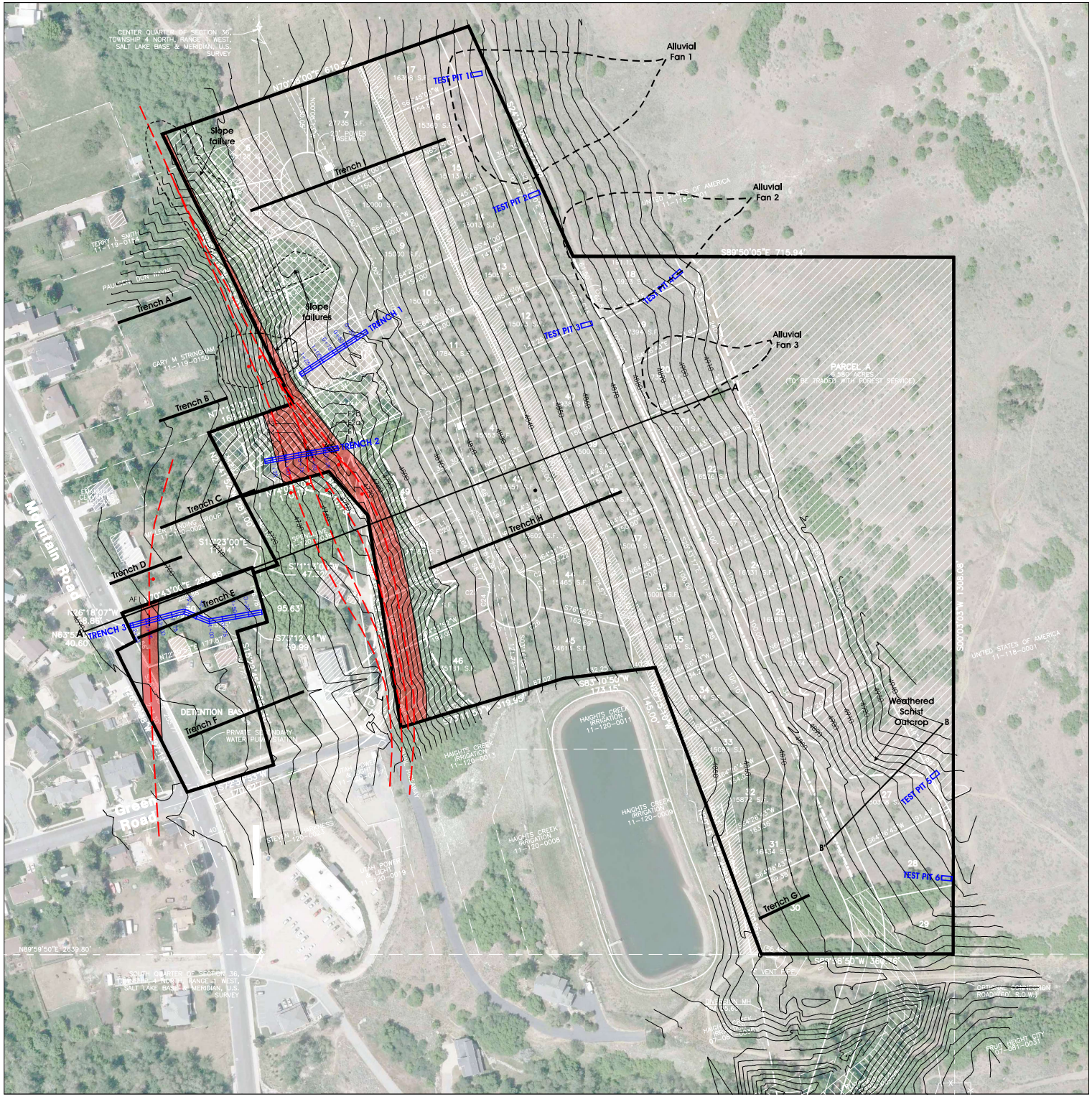
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(1 inch = 250 feet)

WESTERN GEOLOGIC

1937 AIR PHOTO

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 3



EXPLANATION

Fault, bar and ball on downthrown side, dashed where approximately located.

Setback zone. No structures designed for human occupancy.

Great Basin Earth Science (GBES, 2008) trench locations.

Air photo base, site plan, topography, and surveyed trench and fault locations from Reeve & Associates, Inc.

N

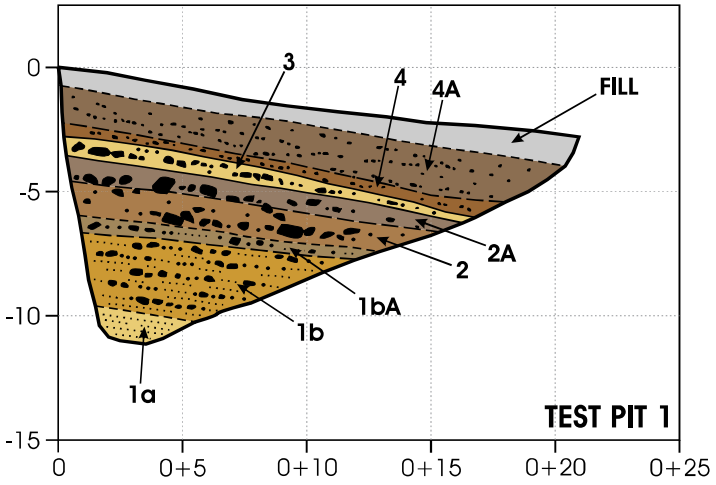
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(1 inch = 100 feet)

SITE PLAN

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 4



Scale in feet

UNIT DESCRIPTIONS

Unit 1. *Lacustrine sediments related to the transgressive stage of Lake Bonneville* - coarsening-upward sequence of sand and gravel.

1a. Pale brown, low to moderate density, poorly bedded, sand with gravel (SW) and trace cobbles.

1b. Brown, low to moderate density, poorly bedded, gravelly sand to sandy gravel (SW/GW) with trace rounded cobbles.

1bA. Paleosol A horizon formed in unit 1.

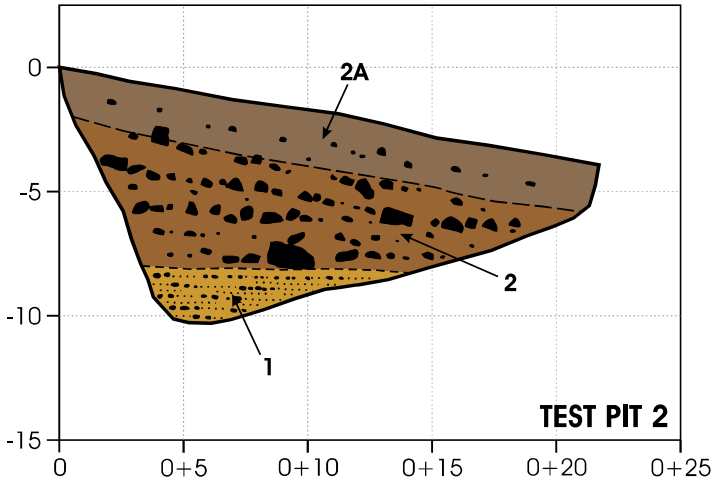
Unit 2. *Latest Pleistocene to Holocene alluvium* - Brown, moderate density, massive, sandy gravel (GW) with silt and cobbles; clasts subround to subangular with stage II carbonate.

2A. Paleosol A horizon formed in unit 2.

Unit 3. *Holocene debris flood deposit* - Pale brown, moderate density, sandy gravel (GW) with silt and cobbles.

Unit 4. *Holocene alluvium* - Brown to dark brown, low to moderate density, massive, root-penetrated, organic-enriched, gravelly sand (GW) with trace cobbles.

4A. Disturbed modern A-horizon soil, overlain by plowed zone.



Scale in feet

UNIT DESCRIPTIONS

Unit 1. *Lacustrine sand and gravel related to the transgressive stage of Lake Bonneville* - Brown, moderate density, poorly bedded, sandy gravel to gravelly sand (SW/GW) with trace rounded cobbles.

Unit 2. *Latest Pleistocene to Holocene alluvium* - Brown to dark brown, moderate density, massive, slightly root penetrated, slightly vesicular, sandy gravel to gravelly sand (GW/SW) with silt, cobbles, and boulders up to 3 feet across; clasts subangular to rounded with stage II carbonate.

2A. Modern A-horizon soil formed in unit 2.

WESTERN GEOLOGIC

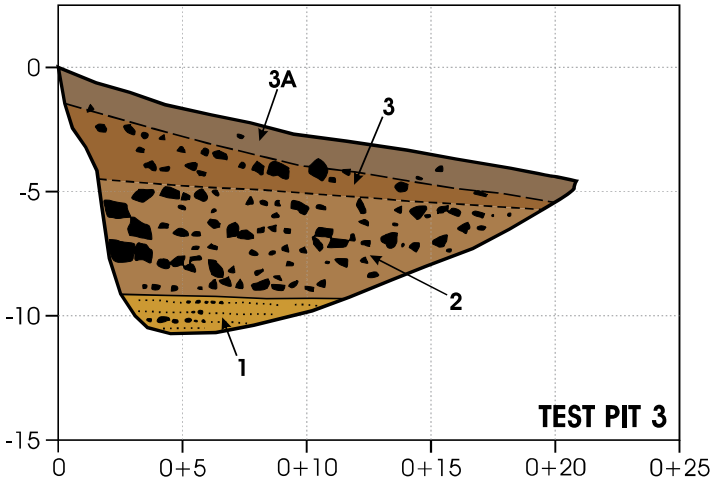
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Walls Logged, East to West

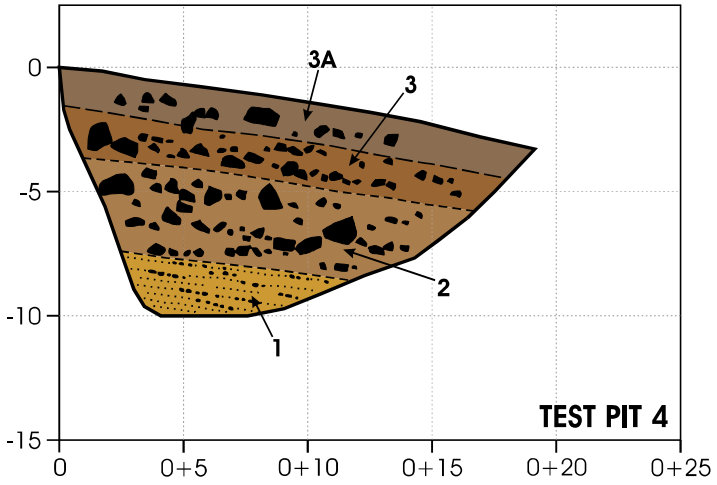
Test pits logged by Bill D Black, P.G. on
July 13, 2015
Reviewed by Craig V Nelson, P.G.

TEST PIT LOGS, SHEET 1

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 5A





UNIT DESCRIPTIONS

Unit 1. *Lacustrine sand and gravel related to the transgressive stage of Lake Bonneville* - Brown, low to moderate density, poorly bedded, gravelly sand (SW) with trace rounded cobbles.

Unit 2. *Latest Pleistocene to Holocene alluvium* - Brown, moderate density, massive, gravelly sand (SW) with cobbles and boulders; clasts subangular to subround with stage II carbonate.

Unit 3. *Holocene alluvium* - Brown to dark brown, moderate density, massive, sandy gravel (GW) with cobbles.

3A. Modern A-horizon soil formed in unit 3.

UNIT DESCRIPTIONS

Unit 1. *Lacustrine sand and gravel related to the transgressive stage of Lake Bonneville* - Brown, low to moderate density, poorly bedded, gravelly sand to sandy gravel (SW/GW).

Unit 2. *Latest Pleistocene to Holocene alluvium* - Brown, low to moderate density, massive, gravelly sand to sandy gravel (SW/GW) with silt, cobbles, and boulders up to 4 feet across; clasts subangular to subround with stage II carbonate.

Unit 3. *Holocene alluvium* - Brown to dark brown, low to moderate density, massive, slightly root penetrated, sandy gravel (GW) with cobbles.

3A. Modern A-horizon soil formed in unit 3.

WESTERN GEOLOGIC

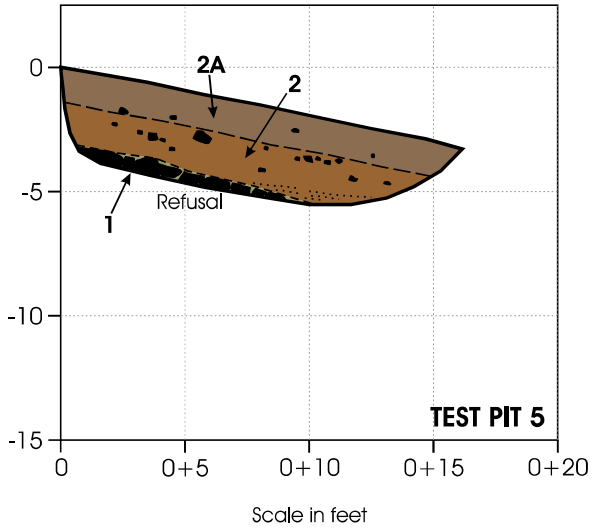
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Walls Logged, East to West

Test pits logged by Bill D Black, P.G. on July 13, 2015
Reviewed by Craig V Nelson, P.G.

TEST PIT LOGS, SHEET 2

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 5B

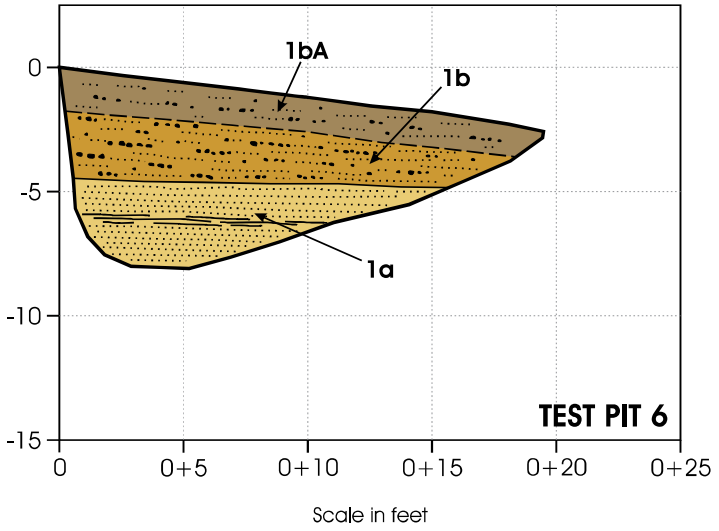


UNIT DESCRIPTIONS

Unit 1. *Fractured Precambrian Farmington Canyon Complex Schist Bedrock.*

Unit 2. *Slope colluvium* - Brown to brownish-gray, low to moderate density, generally massive, gravelly sand (SW) with trace cobbles; poorly bedded basal sand in south test pit end; clasts subround to subangular with stage II carbonate.

2A. Modern A-horizon soil formed in unit 2.



UNIT DESCRIPTIONS

Unit 1. *Lacustrine sediments related to the transgressive stage of Lake Bonneville* - coarsening-upward sequence of sand and gravel.

1a. Pale brown, low to moderate density, poorly to well bedded, sand with pea gravel (SW) and thin silt interbeds.

1b. Brown, low to moderate density, poorly bedded, gravelly sand (SW) with rounded small clasts up to 3".

1bA. Paleosol A horizon formed in unit 1b.

WESTERN GEOLOGIC

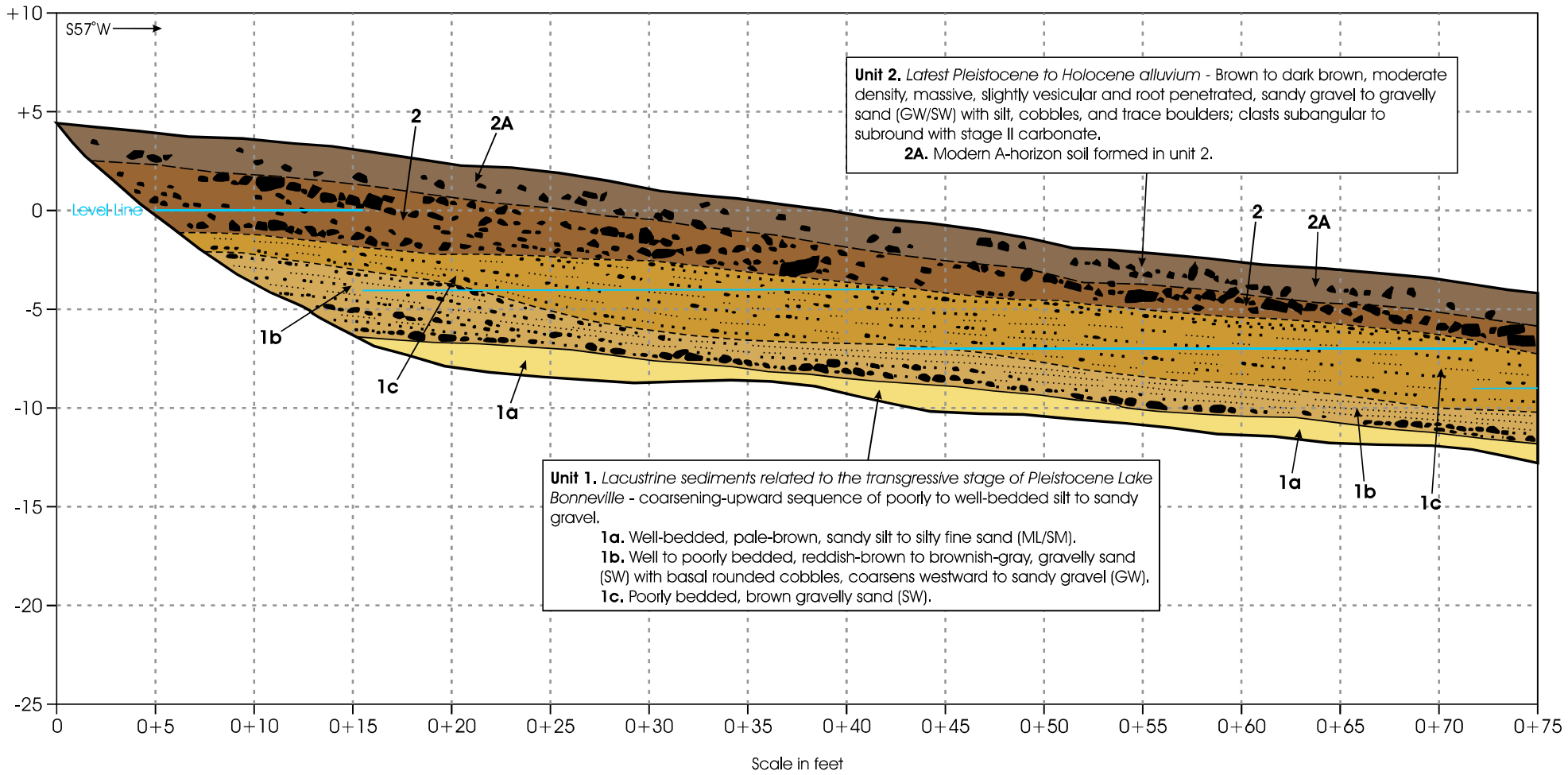
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Walls Logged, East to West

Test pits logged by Bill D Black, P.G. on July 13, 2015
Reviewed by Craig V Nelson, P.G.

TEST PIT LOGS, SHEET 3

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 5C



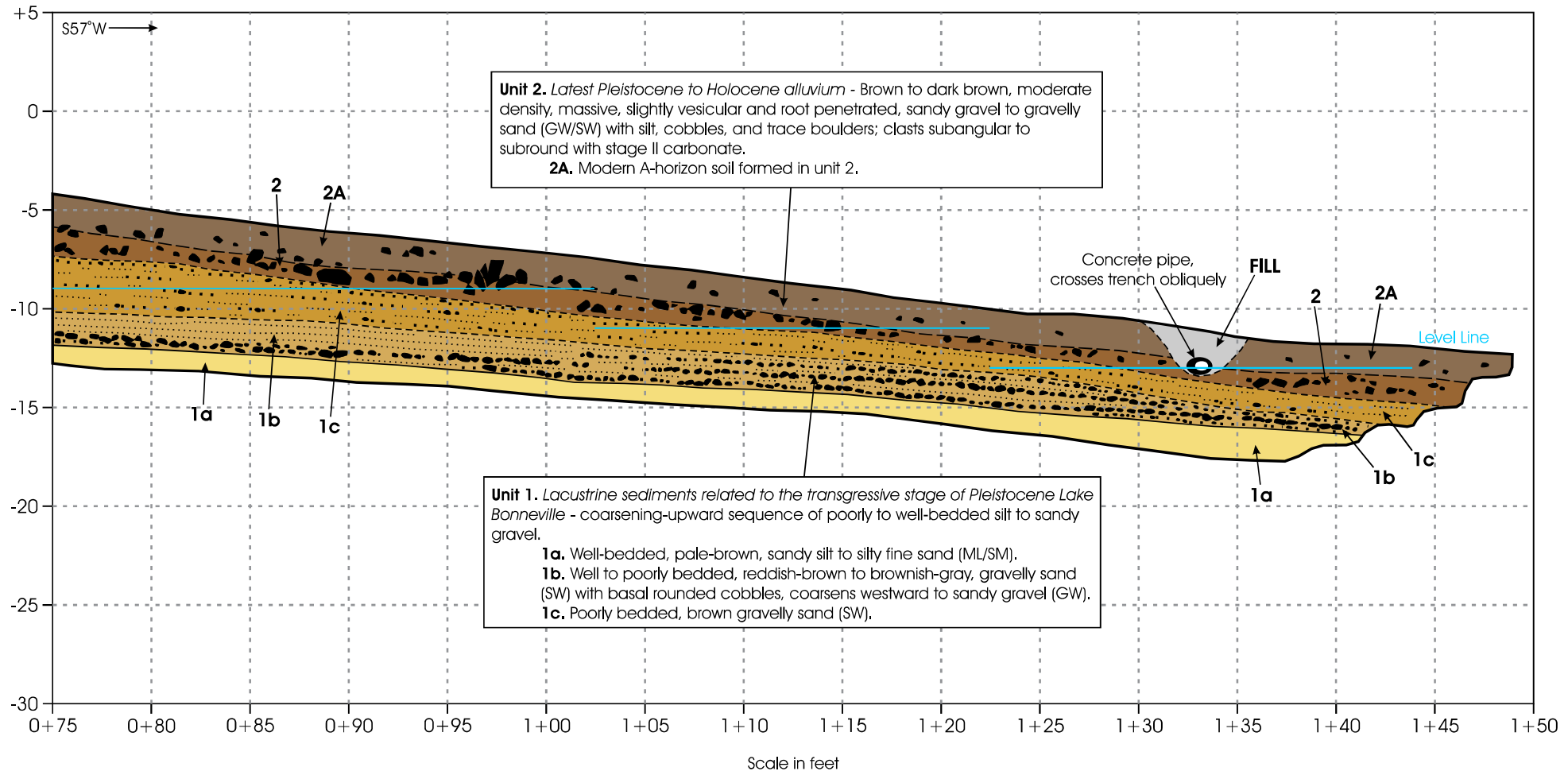
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Wall Logged, East to West

Trench logged by Bill D Black, P.G. on
July 14, 2015
Reviewed by Craig V Nelson, P.G.

TRENCH 1 LOG, SHEET 1

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 6A



SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Wall Logged, East to West

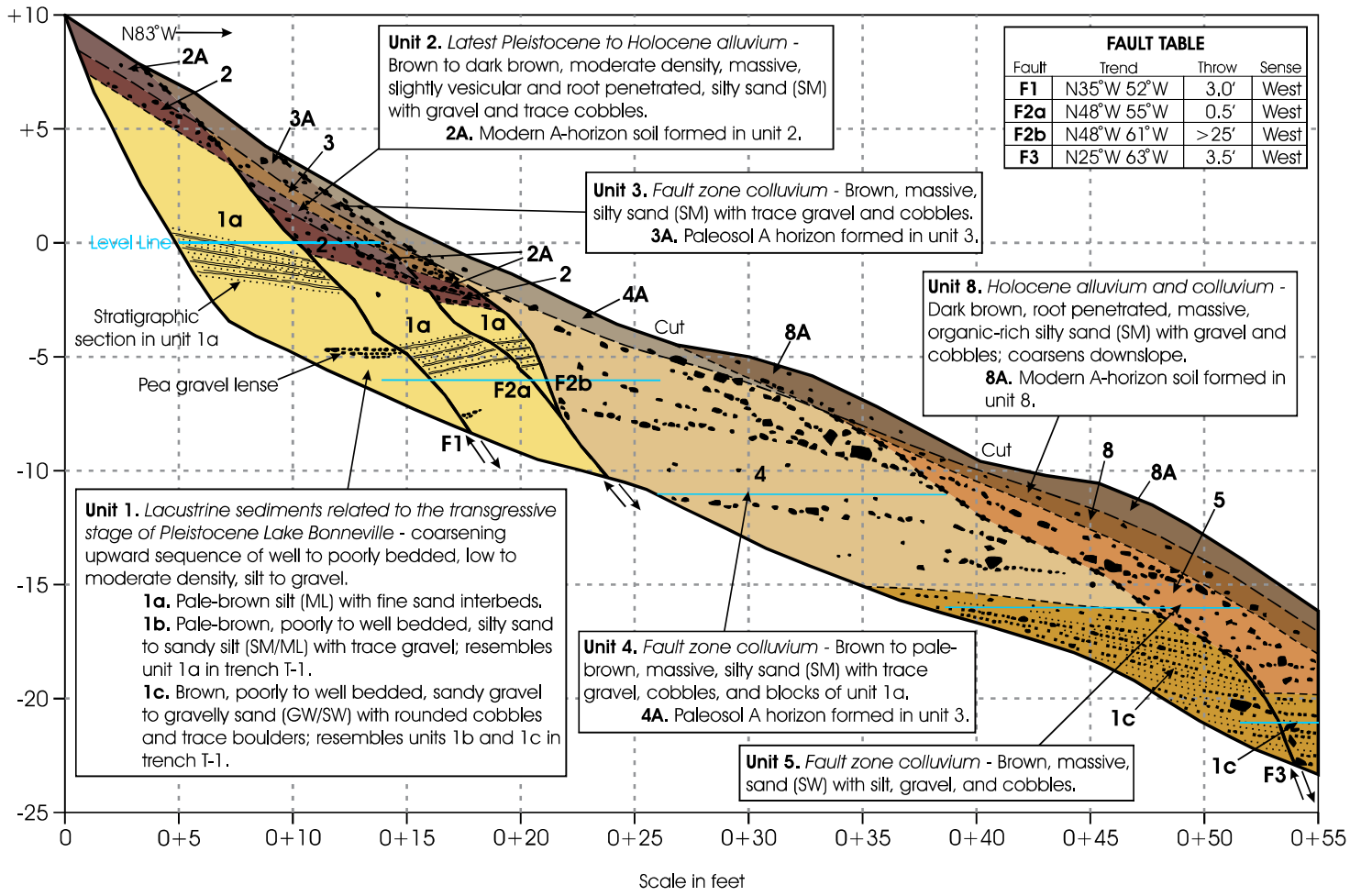
Trench logged by Bill D Black, P.G. on
July 14, 2015
Reviewed by Craig V Nelson, P.G.

WESTERN GEOLOGIC

TRENCH 1 LOG, SHEET 2

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 6B



FAULT TABLE

Fault	Trend	Throw	Sense
F1	N35°W 52°W	3.0'	West
F2a	N48°W 55°W	0.5'	West
F2b	N48°W 61°W	>25'	West
F3	N25°W 63°W	3.5'	West

SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Wall Logged, East to West

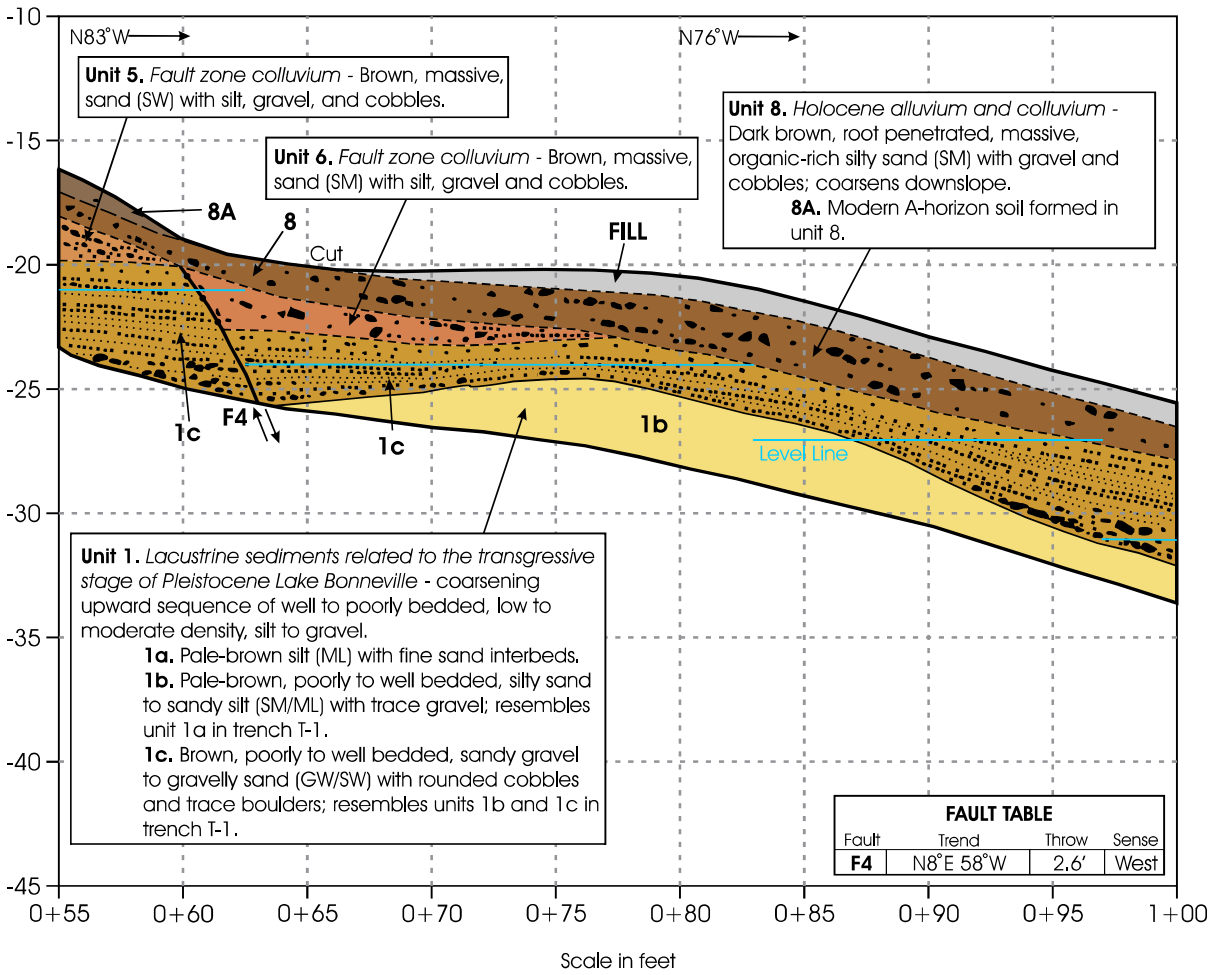
Trench logged by Bill D Black, P.G. on
July 15, 2015
Reviewed by Craig V Nelson, P.G.

WESTERN GEOLOGIC

TRENCH 2 LOG, SHEET 1

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 7A



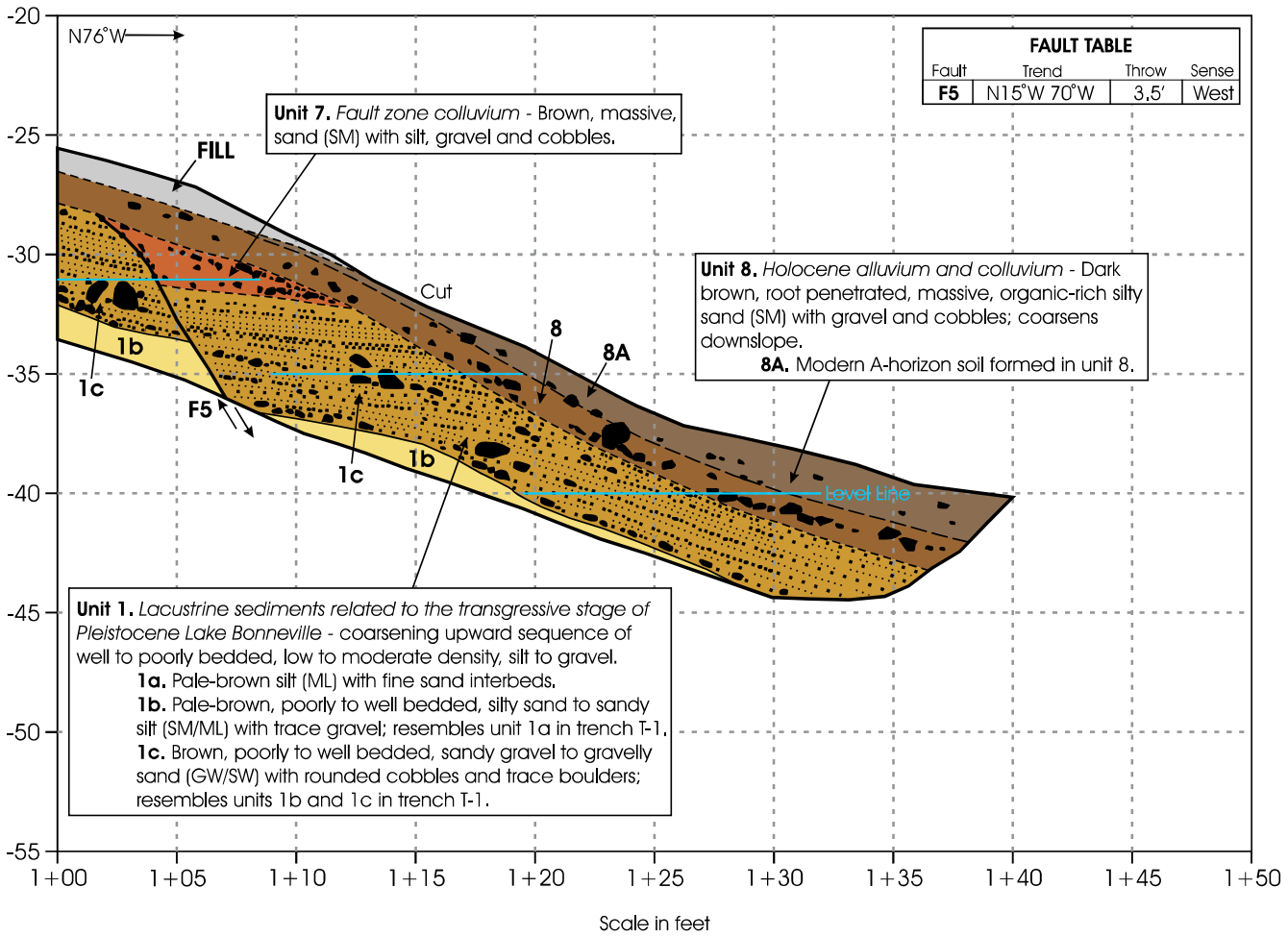
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Wall Logged, East to West

Trench logged by Bill D Black, P.G. on
July 15, 2015
Reviewed by Craig V Nelson, P.G.

TRENCH 2 LOG, SHEET 2

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 7B



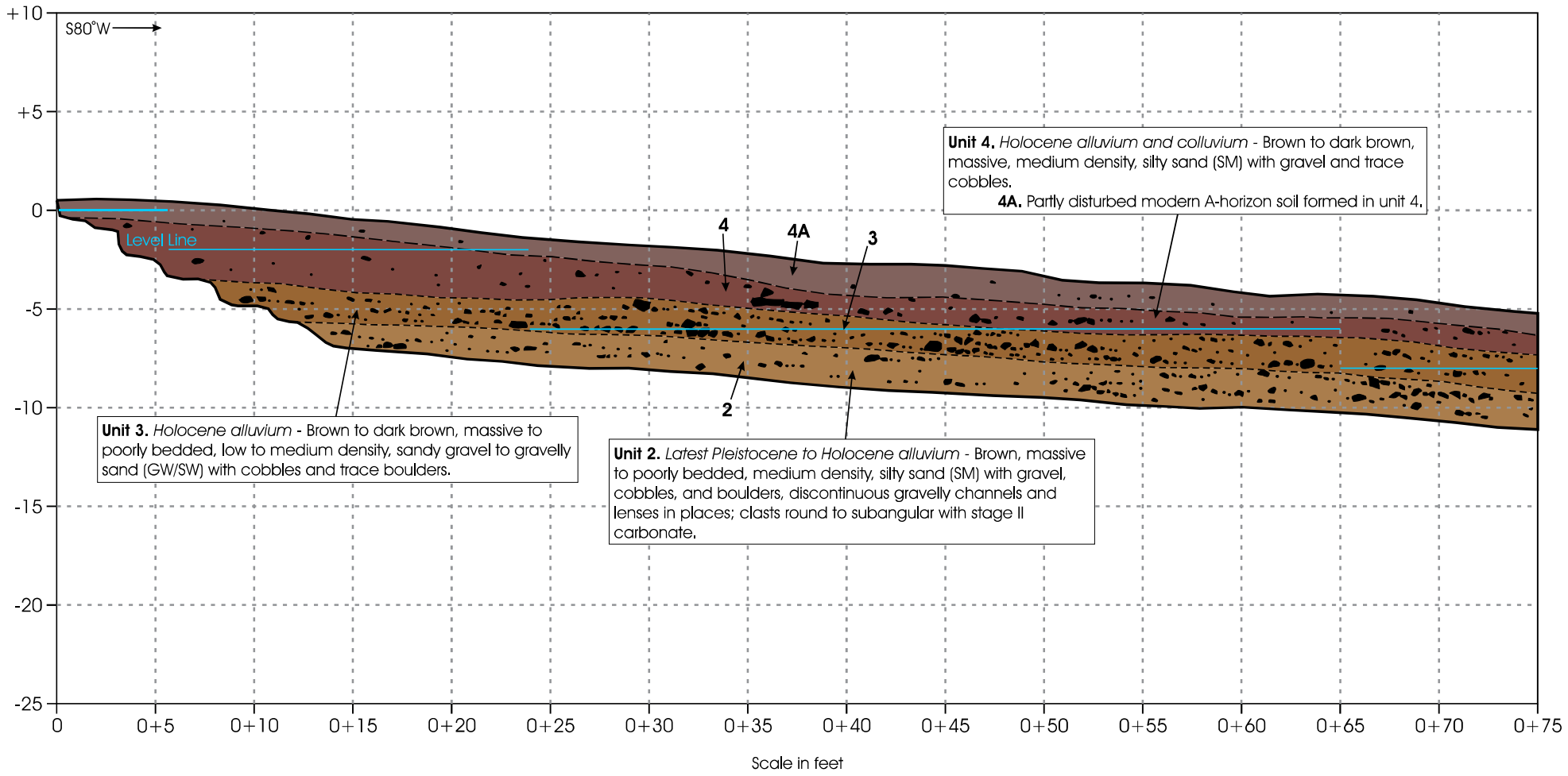
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Wall Logged, East to West

Trench logged by Bill D Black, P.G. on
July 15, 2015
Reviewed by Craig V Nelson, P.G.

TRENCH 2 LOG, SHEET 3

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 7C



WESTERN GEOLOGIC

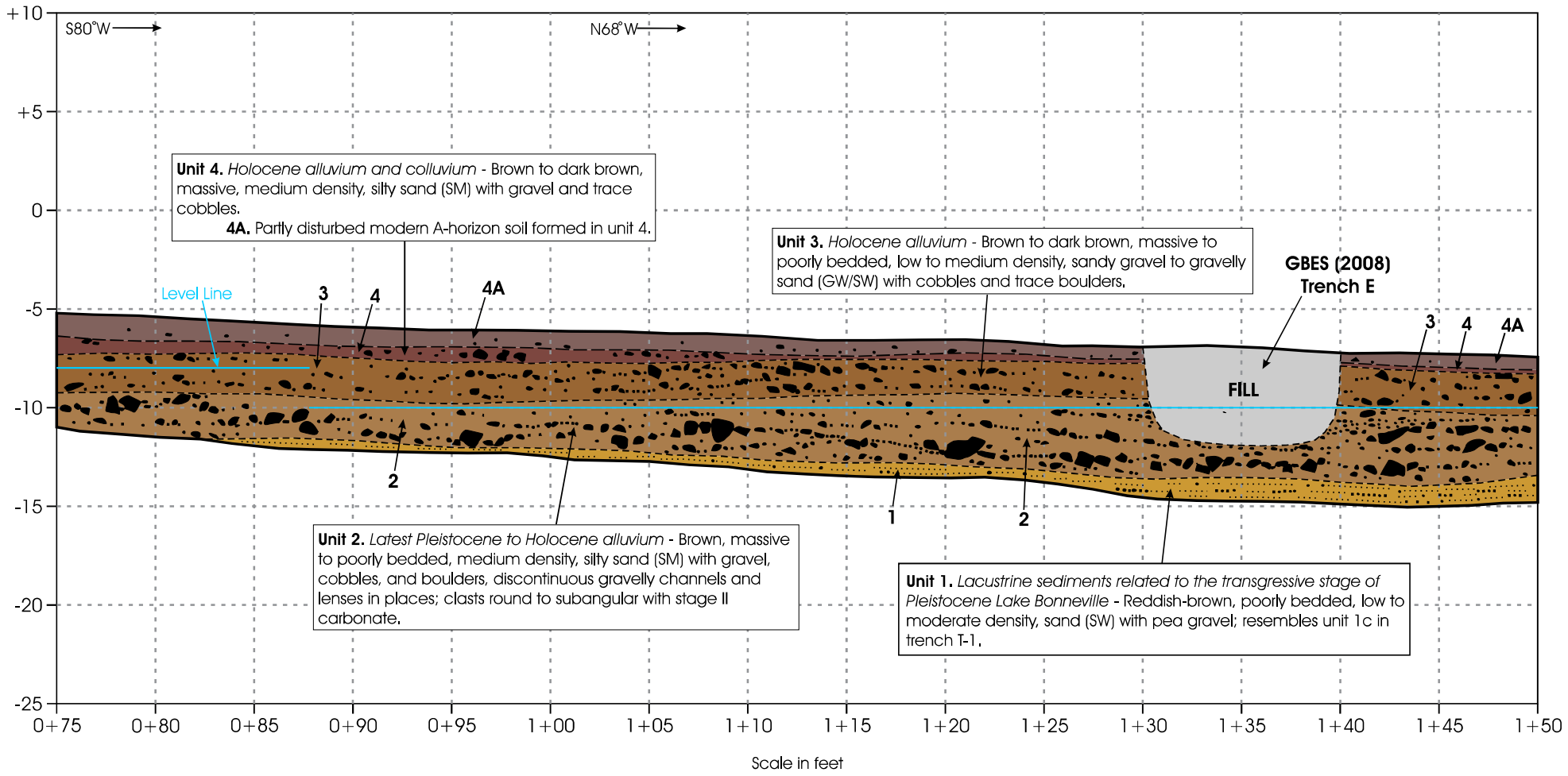
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Wall Logged, East to West

Trench logged by Bill D Black, P.G. on
July 16-17, 2015
Reviewed by Craig V Nelson, P.G.

TRENCH 3 LOG, SHEET 1

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 8A



WESTERN GEOLOGIC

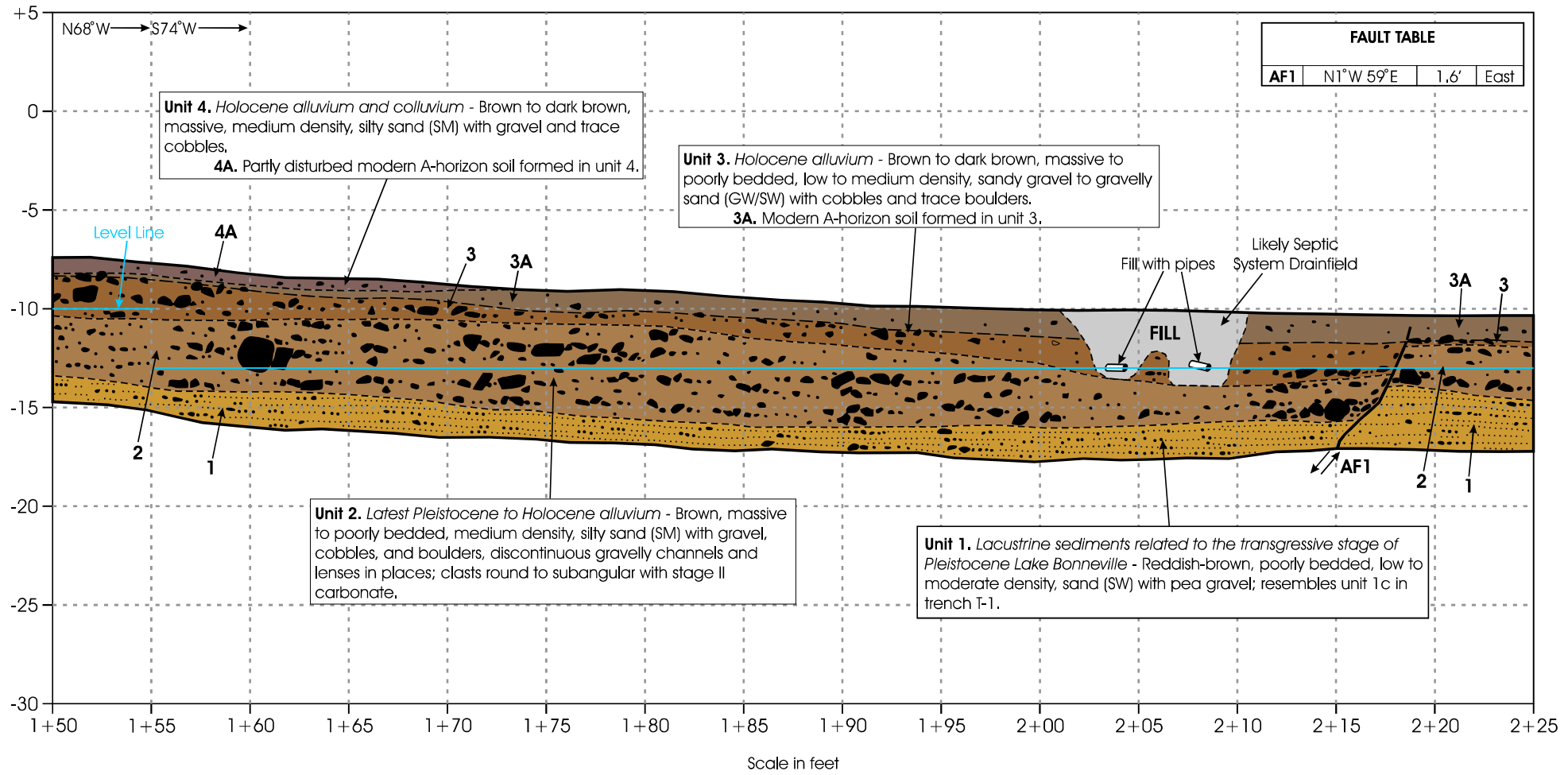
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Wall Logged, East to West

Trench logged by Bill D Black, P.G. on
July 16-17, 2015
Reviewed by Craig V Nelson, P.G.

TRENCH 3 LOG, SHEET 2

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 8B



SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Wall Logged, East to West

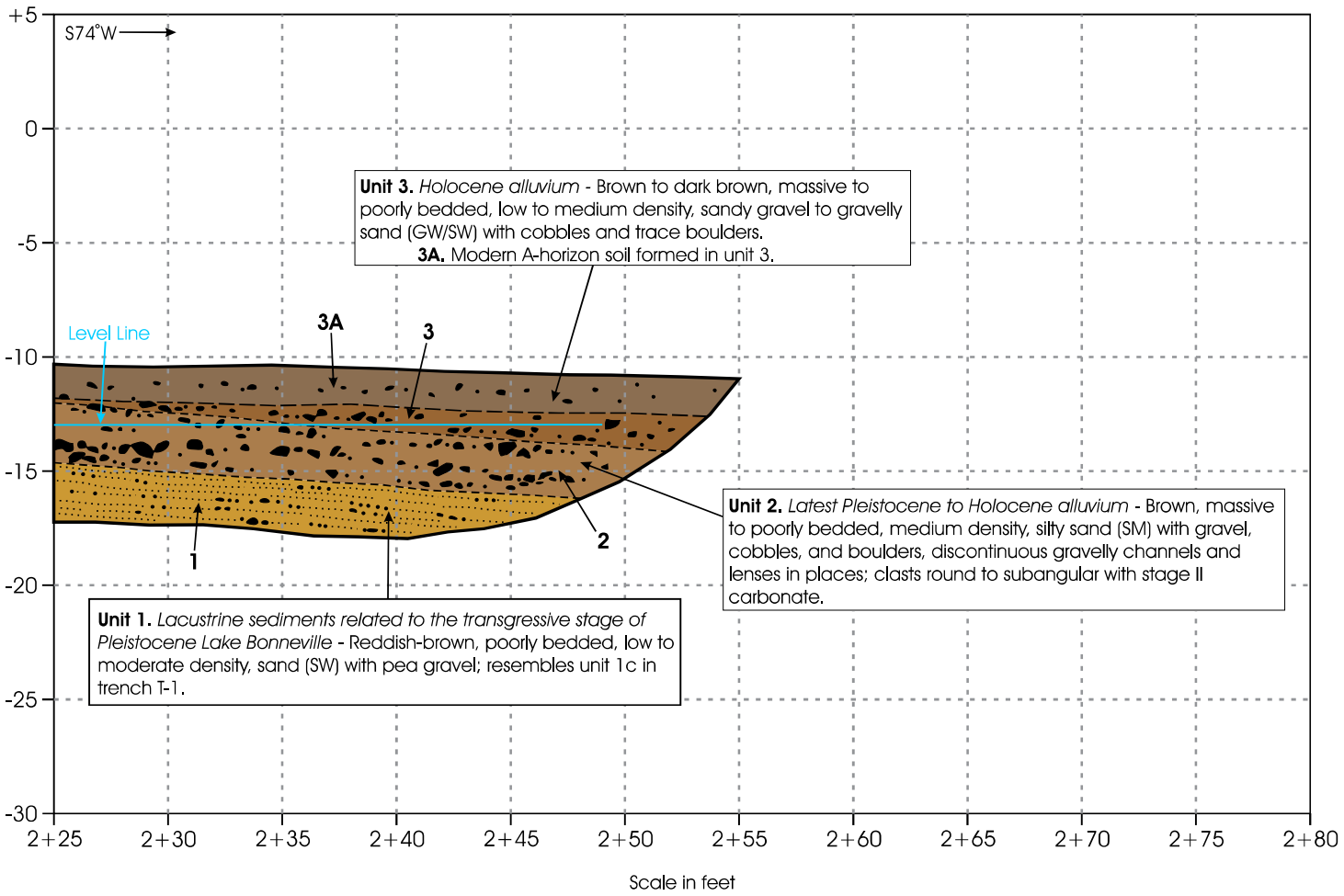
Trench logged by Bill D Black, P.G. on
July 16-17, 2015
Reviewed by Craig V Nelson, P.G.

WESTERN GEOLOGIC

TRENCH 3 LOG, SHEET 3

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 8C



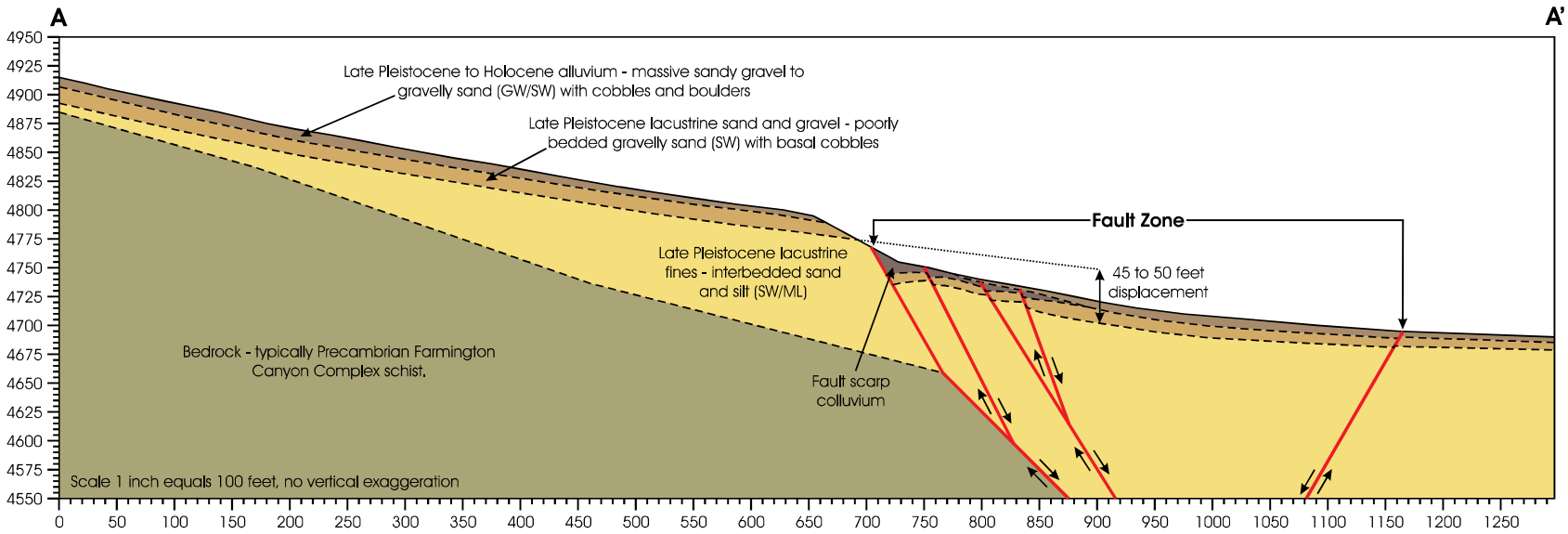
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Wall Logged, East to West

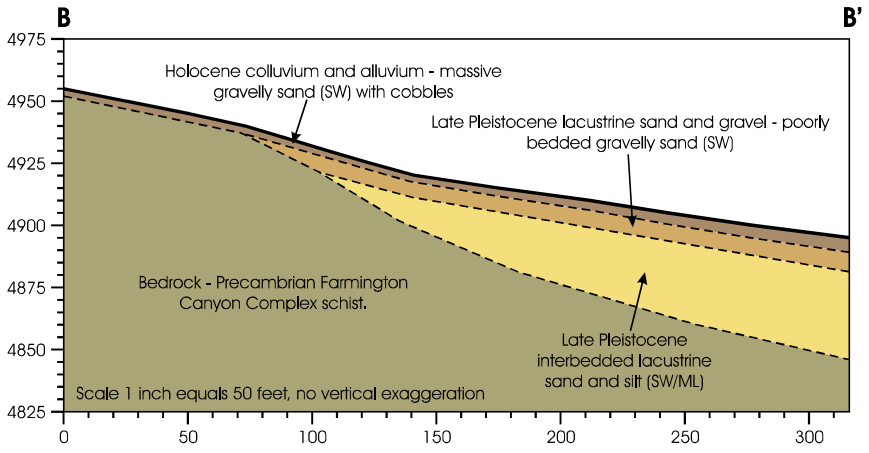
Trench logged by Bill D Black, P.G. on
July 16-17, 2015
Reviewed by Craig V Nelson, P.G.

TRENCH 3 LOG, SHEET 4

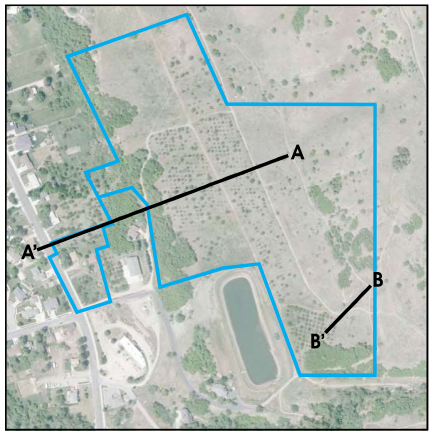
GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 8D





PROFILE LOCATIONS
(See also Figure 4)



WESTERN GEOLOGIC

CROSS SECTION

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 9

REPORT

GEOLOGIC HAZARDS EVALUATION

RAYMOND PROPERTY

ABOUT 175 SOUTH MOUNTAIN ROAD

FRUIT HEIGHTS, UTAH



Prepared for

CMT Engineering Laboratories
707 24th Street, Suite 1A
Ogden, UT 84401

April 3, 2017

Prepared by

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Salt Lake City, Utah 84106

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April 3, 2017

Andrew M. Harris, PE
Senior Geotechnical Engineer
CMT Engineering Laboratories
707 24th Street, Suite 1A
Ogden, UT 84401

SUBJECT: Geologic Hazards Evaluation
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

Dear Mr. Harris:

This report presents results of a reconnaissance-level engineering geology and geologic hazards review and evaluation conducted by Western GeoLogic, LLC (Western GeoLogic) for the Raymond Property in Fruit Heights, Davis County, Utah (Figure 1 – Project Location). The site is northwest of the mouth of Bair Canyon in foothills at the western base of the Wasatch Range in Section 36, Township 4 North, Range 1 West (Salt Lake Base Line and Meridian). Elevation of the site ranges from 4,690 feet to about 5,100 feet above sea level. Based on a Reeves & Associates, Inc. preliminary plan dated May 12, 2016, the intended site use is for development of a 53-lot residential development. The development will reportedly be conducted in three phases denoted by lots numbered in the 100s, 200s, and 300s. It is unconfirmed if these phases will be developed in sequential order. The development is currently identified as the Rock Loft Estates Subdivision on the preliminary plan.

PURPOSE AND SCOPE

The purpose and scope of this investigation is to identify and interpret evident surficial geologic conditions at the site and identify potential risk from geologic hazards to the Project. This investigation is intended to: (1) provide 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 to provide.

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;
- Excavation and logging of six test pits and three trenches at the property between July 13-17, 2015 to evaluate subsurface conditions and potential risk from surface faulting;
- Review of readily-available geologic maps, reports, and air photos; 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 following generally accepted professional engineering geologic principles and practice in Utah; the Guidelines for Preparing Engineering Geologic reports in Utah (Utah Section of the Association of Engineering Geologists, 1986); and in accordance with Fruit Heights City Municipal Code Title 10, Chapter 10 Sensitive Lands Evaluation & Development Standards.

Western GeoLogic previously prepared a geologic hazards evaluation report for the site dated August 15, 2015, in conjunction with a geotechnical evaluation conducted by GSH Geotechnical. At the request of Fruit Heights City, GeoStrata (2016) conducted a review of our 2015 report (Appendix). GeoStrata (2016) generally found the work to be complete, but recommended additional work with regard to landslide and debris flow hazards. Two meetings were held at the Fruit Heights City offices on August 9 and August 15, 2016 to discuss GeoStrata's (2016) review and the recommended additional work. This report updates and replaces our prior report to incorporate additional items from the meeting discussions.

HYDROLOGY

The U.S. Geological Survey (USGS) topographic map of the Kaysville Quadrangle shows no natural springs in the Project vicinity, and no active streams are mapped crossing the site. Bair Creek flows west from the Wasatch Range about 125 feet south of the southeast site corner (Figure 1) and was active at the time of our field program. The creek appears downcut about 30 feet in the area. Several inactive drainages enter the eastern boundary of the property from higher slopes, some of which continue across the upper part of the site to the southwest, although none of these drainages are mapped on Figure 1.

The subsurface hydrology in the area is dominated by the East Shore aquifer system. This aquifer system is comprised of a shallow, unconfined water table zone, and the deeper, often confined, Sunset and Delta aquifers (Feth and others, 1966). The depth to the shallow unconfined aquifer varies somewhat depending on topography and climatic and seasonal fluctuations. It is influenced by seepage from irrigation systems, and infiltration from

precipitation and urban runoff. The Sunset aquifer (typical depth 250-400 feet) and Delta aquifer (typical depth 500-700 feet) provide water that generally meets the standards for public drinking water supply. Based on topography, the local groundwater flow is expected to be to the west-southwest toward Farmington Bay.

Elevation of the shallow aquifer varies somewhat based on seasonal and climatic fluctuations. Perched and confined groundwater conditions are found at the site above and below (respectively) less permeable, fine-grained lacustrine layers commonly found in the area. No groundwater was encountered in any of the borings conducted by GSH at the site to depths of from 33 to 50 feet. No groundwater was also encountered in any of the test pits or trenches conducted for this report. We anticipate the depth to groundwater to be greater than 50 feet in the area in the upper part of the site, and between 30 and 50 feet in the lower part. However, it is possible that small seasonal seeps may be found in steep slope areas that are from perched conditions.

GEOLOGY

Seismotectonic Setting

The property is located along the western base of the Wasatch Range, a major north-south trending mountain range marking the eastern boundary 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 (WFZ) at the base of the Wasatch Range. Late Cenozoic normal faulting, a characteristic of the Basin and Range, began between about 17 and 10 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 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 segments, each of which rupture independently and are capable of generating large-magnitude surface-faulting earthquakes (Machette and others, 1992). The central five segments 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 Project straddles the main trace of the Weber segment, which divides the upper and lower parts of the site.

The Weber segment of the WFZ extends for about 35 miles from the southern edge of the Plain View salient near North Ogden to the northern edge of the Salt Lake salient near North Salt Lake (Machette and others, 1992). Previous paleoseismic studies indicate four large-magnitude surface-faulting earthquakes have occurred on the Weber segment since mid-Holocene time. Nelson and others (2006) report finding evidence for four events at the Garner Canyon and East Ogden sites, including what they infer was a partial segment rupture (with 1.6 feet of displacement) around 500 years ago. This partial segment rupture was not evident at the Kaysville site of McCalpin and others (1994), although chronologic intervals for the remaining three earthquakes were similar. DuRoss and others (2009) report paleoseismic data from the 2007 Rice Creek site support a preferred scenario of six surface-faulting earthquakes in Holocene time, with four events since about 5,400 years ago, and confirm Nelson and others' (2006) partial segment rupture timing.

Lund (2005) indicates preferred earthquake timing for the last four surface-faulting earthquakes on the Weber segment is: (1) an event Z between 200 to 800 years ago (partial segment rupture) and/or between 500 and 1,400 years ago (complete segment rupture), (2) an event Y between 2,300 and 3,700 years ago, (3) an event X between 3,800 and 5,200 years ago, and (4) an event W between 5,400 and 6,800 years ago. The consensus preferred recurrence interval for the Weber segment, as determined by the Utah Quaternary Fault Working Group, is 1,400 years for the past four surface-faulting earthquakes (Lund, 2005).

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_S 7.5 event in 1959 near Hebgen Lake, Montana. However, 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 of these events was the 1934 Hansel Valley (M_S 6.6) event north of the Great Salt Lake.

Surficial Geology

The site is located within the Wasatch Front Valley System, a deep sediment-filled, structural basin flanked by the Wasatch Range to the east and the Lakeside Mountains to the west. The Project is below the Bonneville shoreline of Lake Bonneville, but straddles the Provo shoreline. Surficial geology of the site is mapped by Solomon (2007) as mainly lacustrine sand, silt, and gravel from Pleistocene Lake Bonneville (units Qlsp and Qlgb) and post-lake coalesced alluvial-fan deposits (units Qaf1, Qafy, and Qafp; Figure 2). Small areas of Holocene to upper Pleistocene alluvium and colluvium (unit Qac) and landslide deposits (units Qms1 and Qmsy) are also mapped along the main trace of the Weber segment and extending into the southeast boundary of the site (Figure 2). Solomon (2007) maps the main, west-dipping trace, and an east-dipping antithetic trace of the Weber segment dividing the proposed development (Figure 2). Most of the development is on the upthrown side of the main fault trace; two lots and a proposed stormwater detention basin are on the downthrown fault side.

Solomon (2007) describes surficial units in the site vicinity, from youngest to oldest in age, as follows:

Qms1 -- Landslide deposits, unit 1 (Historical to Middle Holocene). Very poorly sorted clay, silt, sand, and minor gravel; grain size and texture varies with the nature of the deposits in the source area; unit 1 landslides occur in bluffs bordering the Weber River flood plain and north of Kaysville along steep bluffs incised by drainages following regression of Lake Bonneville; characterized by moderately fresh scarps and hummocky topography, with freshest scarps in areas of historical movement. Most unit 1 landslides along the Weber River originate in sandy Lake Bonneville deltaic deposits, form gently sloping alluvial-fan shaped deposits overlying unit 2 landslide deposits on river terraces that fringe the bluffs, and apparently result from flow failure, perhaps liquefaction induced, of the bluff face; one landslide along the Weber River, the Cedar Bench landslide, is an earth slide resulting from fill failure on the slope bordering an irrigation pond (Solomon, 1999). Landslides along other drainages are found at and slightly below the gradational contact between overlying Lake Bonneville sand and silt and underlying silt and clay and are the result of a combination of rotational and translational earth slides; parts of several of these landslides have been historically reactivated, resulting in damage to roads and houses. Thickness of the deposits is highly variable.

Qmsy – Younger landslide deposits, undivided (Historical to Upper Pleistocene). Very poorly sorted, clay to boulder-size gravel in a matrix of silt, sand, clay, and pebbles; grain size and texture varies with the nature of the deposits in the source area. Younger landslide deposits are found near Adams, Hobbs, and Holmes Reservoirs and along the Wasatch Range front; deposits along the steep bluffs near the reservoirs are derived from Lake Bonneville deposits incised by drainages following regression of Lake Bonneville and resemble unit 1 landslide deposits but lack relatively fresh scarps; younger landslide deposits along the range front are derived from the Farmington Canyon Complex (Xfc) and either result from incision of drainages after Lake Bonneville regression, obscure the Bonneville shoreline, or overlie Lake Bonneville deposits; younger landslide deposits may include unit 1 and unit 2 landslide deposits that are undifferentiated because units complexly overlap; thickness of the deposits is highly variable.

Qaf1 – Modern (level 1) alluvial-fan deposits (upper Holocene). Poorly to moderately sorted, weakly to non-stratified, silt- to boulder-size sediment deposited principally by debris flows and debris floods at the mouths of mountain-front canyons; upper parts characterized by abundant boulders and debris-flow levees that radiate away from the fan apex; small, finer grained alluvial-fan deposits are found on gentle slopes in the northern part of the quadrangle, deposited by shallow, dispersive flows; equivalent to the younger part of Qafy but differentiated where deposits can be mapped separately; commonly overlies lacustrine and older alluvial-fan deposits; debris-flow and debris-flood hazards are highest on modem alluvial fans, their undivided equivalents within younger alluvial fans (Qafy), and debris-flow deposits (Qmf); less than 30 feet (9 m) thick.

Qac – Alluvial and colluvial deposits, undifferentiated (Holocene to upper Pleistocene). Poorly to moderately sorted, generally poorly stratified, clay- to boulder-size, locally derived sediment deposited in drainages along the Wasatch Range front by fluvial, rockfall, slopewash, and creep processes; generally less than 30 feet (9 m) thick.

Qc – Colluvial deposits (Holocene to upper Pleistocene). Poorly to moderately sorted, angular, clay- to boulder-size, locally derived sediment deposited by rock fall, slopewash, and soil creep on moderate to steep mountain-front spurs above the Bonneville shoreline; most bedrock is covered by at least a thin veneer of colluvium, and only the larger, thicker deposits are mapped; maximum thickness about 20 feet (6 m).

Qafy – Younger alluvial-fan deposits, undivided (Holocene to upper Pleistocene). Poorly to moderately sorted, weakly to non-stratified, silt- to boulder-size sediment deposited principally by debris flows, debris floods, and streams; equivalent to modern and level 2 alluvial-fan deposits (Qaf1, and Qaf2.) that are undifferentiated because units are complexly overlapping or too small to show separately; also mapped where the age of Holocene alluvial-fan deposits is uncertain; upper parts of fans are locally deeply incised; occurs near canyon mouths along the mountain front; probably less than 40 feet (12 m) thick.

Qafp – Alluvial-fan deposits related to the Provo (regressive) phase of the Bonneville lake cycle (upper Pleistocene). Poorly to moderately sorted, silt- to cobble-size sediment, with local boulders, deposited principally by debris flows whose surfaces are graded to the Provo shoreline at the mouth of Bair Canyon; incised by active streams; probably less than about 40 feet (12 m) thick.

Qlsp - Lacustrine sand and silt related to the Provo (regressive) phase of the Bonneville lake cycle (upper Pleistocene). Fine- to coarse-grained lacustrine sand and silt with minor gravel deposited at and below the Provo shoreline; locally overlain by a thin veneer of eolian sand; grades downslope into fine-grained Lake Bonneville deposits and laterally northward to sandy deltaic deposits (Qldp); typically thick bedded and well sorted; gastropods locally common; deposited in relatively shallow water in beaches and, at the mouths of larger canyons, in deltas that no longer retain distinctive morphology; as much as 50 feet (15 m) thick.

Qlgb – Lacustrine gravel and sand related to the Bonneville (transgressive) phase of the Bonneville Lake cycle (upper Pleistocene). Moderately to well-sorted, moderately to well-rounded, clast-supported, pebble to cobble gravel interbedded with lesser amounts of pebbly sand and clean sand, deposited between the Bonneville and Provo shorelines; thin to thick bedded; gastropods locally common in sandy lenses; locally partly cemented with calcium carbonate; typically forms a bench along the Bonneville shoreline near the base of the Wasatch Range; locally as much as 200 feet (60 m) thick.

Xfc – Farmington Canyon Complex (Early Proterozoic). Metamorphic and igneous rocks, not mapped in detail for this study; Bryant (1984) concluded that these rocks in the Kaysville quadrangle record a Late Archean metamorphic event, but Barnett and others (1993) and Nelson and others (2002) found evidence that metamorphism was Early Proterozoic, affecting rocks either initially deposited as Archean sediment or Early Proterozoic sediment incorporating material eroded from nearby Archean crust; Yonkee and Lowe (2004) supported this interpretation and provided further interpretations based on detailed mapping of the Farmington Canyon Complex in the adjacent Ogden 7.5-minute quadrangle.

References included in the above unit descriptions are not provided in this report, but are provided in Solomon (2007).

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 underlie the site and site vicinity.

Timing of events related to the transgression and regression of Lake Bonneville is indicated by calendar age estimates of significant radiocarbon dates in the Bonneville Basin (Donald Currey, University of Utah; written communication to the Utah Geological Survey, 1996; and verbal communication to the Utah Quaternary Fault Parameters Working Group, 2004). Approximately 32,500 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, after about 18,000 years ago. During the transgression and highstand, major drainages that emanate from within the Wasatch Range (such as the Weber River) formed large deltaic complexes in the lake at their canyon mouths. The lake remained at its highest level until 16,500 years ago, when headward erosion of the Snake River-Bonneville basin drainage divide caused a catastrophic incision of the threshold and the lake level lowered by roughly 360 feet in fewer than two months (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. 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. Drainages that fed Lake Bonneville began

downcutting through stranded deltaic complexes and near-shore deposits as the lake receded from the Provo shoreline, including the alluvial fan at the mouth of Bair Canyon. Great Salt Lake 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).

SITE CHARACTERIZATION

Air Photo Observations

A 1937 aerial photograph available from the U.S. Geological Survey (frame 10-AAK3-41, Figure 3) was reviewed to obtain information about the geomorphology of the site and surrounding area. Figure 4 also shows a site plan at a scale of 1 inch equals 100 feet on a 2012 Utah AGRC high-resolution digital orthophoto. The main trace of the Weber segment of the WFZ is evident on Figure 3 crossing the western part of the property, corresponding to the steep escarpment dividing the upper and lower portions of the site. Trenching evidence (discussed below) indicates that this escarpment includes several west-dipping faults, which appear to converge northward and southward (Figure 3). A weak lineament is also evident on Figure 3 that corresponds to the antithetic fault observed in trench 3. We note that Solomon (2007; Figure 2) maps the main fault further west in the area south of trench 2, and the antithetic fault is mapped further east than found in trench 3. Solomon's (2007) fault locations do not appear to correspond to any topographic features. We also note that the slope failure east of the site (discussed above and mapped on Figure 2) is not evident on the 1937 air photo. This suggests that the failure is historical (post-1937).

Several small inactive drainage swales are also evident on Figure 3 entering the eastern part of the property. The two northernmost drainages head slightly above the Bonneville shoreline and have small associated alluvial fans at their mouths (labeled 1 and 2, Figure 3). A third drainage swale is to the south that heads below the shoreline and also has a small alluvial fan (labeled 3, Figure 3). The fan size and morphology for all three drainages decreases southward. Further southward, the remaining drainages entering the site appear to be small erosional gullies that head below the Bonneville shoreline (Figure 3). These drainages have no associated alluvial fans, and many continue across the site. All the ephemeral drainages entering the site are likely only active following cloudburst storms and spring snowmelt, and may not be consistently active. As recommended by GeoStrata (2016), drainage courses evident on the 1937 air photo (Figure 3) have been added to Figure 4, although we note that the drainage pattern has been substantially altered and will likely be further altered by the proposed development.

Empirical Observations

On July 14, 2015, Mr. Bill D. Black of Western GeoLogic conducted a reconnaissance of the property and immediate vicinity. Weather at the time of the site reconnaissance was clear with temperatures in the 90's (°F). The site is on west-facing slopes at the base of the Wasatch Range northwest of the mouth of Bair Canyon. Vegetation at the site consists of fruit orchards and native trees, scrub oak, sage brush, and grasses.

The property is generally on two gently sloping surfaces separated by a steep west-dipping escarpment formed by displacement along the Weber segment of the WFZ; Figure 4 shows most of the proposed development is on the upper surface east of the escarpment except for a non-contiguous area on the west. Slope gradients vary across the site and exceed 30 percent in some areas, including along the fault escarpment and slopes bordering the Project to the east (as marked by the slope boundary line on Figure 4). The escarpment north of trench 1 showed evidence for rotational slumping in interbedded lacustrine sand and silt deposits, and Solomon (2007) maps a small landslide further north in the escarpment on Figure 2 in the northwest corner of the property. We consider the entire escarpment to be an active mass-wasting zone mitigated only by heavy tree cover. Solomon (2007) also maps a landslide northeast of the southeast site corner. A low arcuate scarp from this failure was evident at the surface. The landslide appeared to be a broad, shallow rotational failure sourced in lacustrine deposits overlying weathered schist. We do not anticipate this landslide will have any direct impacts on slope stability at the property given that deposits from the failure (Figure 4) do not appear to extend into the site.

No active streams cross the site, but two drainage swales were observed east of the site that appeared capable of generating small debris flows. Such flows would likely initiate at or above the Bonneville shoreline and entrain lacustrine deposits below the shoreline. Some large boulders up to about three feet across were evident embedded in the surficial sediment veneer, but no evidence of debris flow levees or recent flows was observed at the drainage mouths. Scattered weathered outcrops of schist of the Precambrian Farmington Canyon Complex were also observed in slopes east of the property and in one area in the southeast part of the site near test pit 5. No boulders from rock falls were observed at the property, and upslope bedrock outcrops appeared to have low relief and likely incapable of generating large rock fall boulders due to their composition. No evidence of other geologic hazards was observed.

Prior Evaluations

Great Basin Earth Science (GBES) conducted a previous geologic hazards evaluation for the proposed Cherry Ridge development in 2008. The present site constitutes part of the former Cherry Ridge development. The GBES (2008) study included excavation and logging of twelve test pits and twelve trenches at the site to evaluate subsurface conditions and risk from surface faulting, as well as surficial observations and calculations of potential debris flow sediment volumes.

With regard to surface faulting, GBES (2008) found no evidence for active faults in the trenches excavated at the site. GBES (2008) trench locations are shown on Figure 4. GBES (2008) inferred the main trace of the Weber segment was located in the lower half of the west-facing escarpment, although no trenching was conducted to confirm its location. GBES (2008) instead recommended a broad setback area extending east and west from the escarpment toe and crest. Six trenches (A through F, Figure 4) were excavated to expose the antithetic fault to the west, but found no evidence for the fault. However, we note that: (1) the antithetic fault was found in our trench 3 further west than

mapped, and (2) GBES' (2008) trench E encountered fill from an onsite septic system in this area that would have complicated recognition of the antithetic fault. The antithetic fault was also not recognized in GBES' (2008) trench D, but may correspond to the abrupt down-to-the-east inflection in the unit II upper contact between stations 80 and 85 feet.

With regard to landslide hazards, GBES (2008) identified three areas of landsliding in the fault escarpment. Two of these areas correspond to our observations north of trench 1, whereas the third was queried and located in the area of our trench 2. This latter failure does not exist given the lack of evidence for landsliding in trench 2. GBES (2008) did not believe that the slope failure east of the site mapped by Solomon (2007) existed.

With regard to debris flows, GBES (2008) concluded a debris flow hazard existed on their mapped alluvial fans A, B, and E. They indicate the remaining fans showed no evidence for Holocene activity, based on surface observations and test pit data. Fans A and B are north of the Project and do not impact this site, whereas fan E corresponds to our alluvial fan 2 on Figure 4. GBES (2008) estimated a design magnitude debris volume of 1,400 cubic yards for the fan E [2] drainage using empirical bulking.

With regard to rock fall hazards, GBES (2008) did not identify any potential hazard areas within the Project boundaries. Two areas were identified north of the Project that do not impact the site.

As noted in the Purpose and Scope Section above, Western GeoLogic also conducted a geologic hazards evaluation for the property in August 2015 that was reviewed by GeoStrata (2016). This report updates and replaces our prior report to satisfy GeoStrata's (2016) review comments based on discussions in two post-review development meetings. GeoStrata (2016) also recommended a future subsurface investigation be conducted with regard to landsliding along the fault zone, but it is our understanding that this investigation will be conducted at a later date as needed.

Subsurface Investigation

On July 13, 2015 six test pits were excavated at the site with a large trackhoe to evaluate subsurface conditions. Test pit locations are shown on Figure 4. Logs of the test pits at a scale of 1 inch equals 5 feet are shown on Figures 5A through 5C. Test pit 1 was excavated within alluvial fan 1 (Figure 4) and found evidence for a sequence of post-Lake Bonneville debris flow and flood deposits displaying two paleosol A horizons (units 2 through 4, and units 2A and 3A; Figure 5A). Unit 4 was overlain by fill from prior site disturbance. Discounting surface fill, the alluvial units showed thicknesses ranging between 0.4 to 2.3 feet, with an average thickness of about 1.6 feet. This evidence confirms a debris flow hazard exists on alluvial fan 1 (Figures 3 and 4). Test pits 2 through 4 exposed similar post-lake alluvium overlying Lake Bonneville sand and gravel deposits, but no evidence for paleosols or discrete debris flows (Figures 5A-B). Test pit 5 exposed about 3 feet of sandy slope colluvium overlying fractured schist bedrock (Figure 5C). Shallow bedrock in this test pit was anticipated given an outcrop is located nearby to the west (Figure 4). Test pit 6 exposed only lacustrine sand and gravel overlying interbedded sand and silt (Figure 5C).

To evaluate the hazard from surface faulting, three trenches were also excavated at the Project using a large trackhoe. Trench locations are shown on Figure 4. Trench 1 extended across lot 2 an overall S57°W for a distance of 149 feet, and generally covered the area between the crest of the steep escarpment to the east edge of the Surface Fault Rupture Special Study Area (SFRSSA). Trench 2 was located about 140 feet south of trench 1, and extended across the escarpment an overall S80°W for a distance of 140 feet. Trench 3 was located about 270 feet south of trench 2, and extended sinuously across lot 50 for a total distance of 255 feet. Trench 3 was excavated in this fashion to avoid following GBES'(2008) trench E and encountering fill from the onsite septic system exposed in that trench. The trenches were excavated to a depth sufficient to expose evidence for active faulting. Trench logging followed methodology in McAlpin (1996). Trench and active fault locations were surveyed by Reeves & Associates on July 17, 2015. The trenches were reviewed by Tim Thompson, P.G. of GeoStrata, Fruit Heights City contract geologic reviewer, also on July 17, 2015. No significant complications were encountered that impacted our findings.

Trench 1 exposed a coarsening-upward sequence of lacustrine silt, sand, and gravel (units 1a, 1b, and 1c), overlain by gravelly to sandy post-lake alluvium (unit 2) in which the modern A-horizon soil was forming (Figures 6A-B). Unit 1b was observed to contain a basal cobbly zone that coarsened westward. The stratigraphic sequence exposed in the trench appeared similar to that exposed in test pits 2 and 3 (Figures 5A-B), with the addition of unit 1a underlying the lacustrine sand and gravel (Figures 6A-B). No evidence for active faulting was exposed in the trench.

Trench 2 exposed lacustrine silt with thin fine sand interbeds (unit 1a) in the east trench end overlain by a thin veneer of alluvium (unit 2), both of which are displaced down-to-the-west across a series of three faults (F1, F2a, and F2b; Figure 7A). Fault F1 displays 3.0 feet of offset in correlative bedding, fault F2a displays 0.5 feet of offset, and fault F2b displays more than 25 feet of offset. No correlative stratigraphy was observed across fault F2b, which we infer is the main trace of the Weber segment of the WFZ. Unit 1a lies beneath the lacustrine sequence exposed in trench 1. However, the sequence exposed in trench 1 is eroded and not present in the footwall of the main fault. We infer units 1a-1c in trench 1 to correlate to units 1b and 1c in trench 2, and unit 2 is correlative between the trenches. Unit 1a in the footwall of trench 2 is at depth and no longer exposed in the hanging wall, but the overlying sequence is exposed (Figure 7A). Further west the lacustrine sequence is displaced down-to-the-west across three additional faults: (1) fault F3 with 3.5 feet of offset, (2) fault F4 with 2.6 feet of offset, and (3) fault F5 with 3.5 feet of offset (Figures 7B-C). Five colluvial wedges from fault scarp degradation are exposed on the downthrown sides of faults F1, F2a, F3, F4, and F5 (units 3 through 7; Figures 7A-C). The fault scarp colluvium is overlain by alluvium and colluvium in which the modern A-horizon soil is forming (unit 8 and 8A, Figures 7A-C).

Trench 3 exposed lacustrine sand with pea gravel (unit 1) resembling unit 1c in trench 1 west of station 0+82 feet (Figures 8B-8D). This unit is overlain by a sequence of three Holocene alluvial deposits (units 2 through 4, Figures 8A-8D). Backfill from GBES' (2008) trench E was encountered between stations 1+30 and 1+40 feet, and an area of fill with pipes was encountered at the location of the onsite septic system. However, west of this area the stratigraphic sequence is displaced 1.6 feet down to the east across fault AF1. Recognition of this fault would have been difficult in GBES' (2008) trench E due to the onsite septic system.

No other evidence for faulting was observed in the trenches excavated for this study, though the exposed sediments were of sufficient age to show evidence for active faulting.

Cross Sections
Figure 9 shows two cross sections across the slopes in the central and south parts of the site. Cross section A-A' is at a scale of 1 inch equals 100 feet, whereas B-B' is at a scale of 1 inch equals 50 feet. The cross sections have no vertical exaggeration. Cross section locations are shown on Figure 4. Unit contacts and dips are inferred from the test pit and trenching data, a review of boring logs conducted by GSH, and site observations.

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 to the proposed development. A "moderate" hazard rating (M) indicates a hazard that poses an equivocal risk or only impacts a small portion of the development. 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. High and moderate-risk hazards may require further studies or mitigation, whereas 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 for the Raymond Property.*

Hazard	H	M	L	…Hazard Rating
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**	
Radon	**X**			
Problem Soil			**X**	

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. Mapped active faults within this distance include: the East and West Cache fault zones; the Brigham City, Weber, Salt Lake, and Provo segments of the Wasatch fault zone; the East Great Salt Lake fault zone; the Morgan fault; the West Valley fault zone; the Oquirrh fault zone; and the Bear River fault zone (Black and others, 2003).

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 (Costa and Baker, 1981). Peak ground, 0.2 second spectral, and 1.0 second spectral accelerations (percent of gravity, %g) at the site with 10% and 2% probabilities of exceedance in 50 years are estimated in Frankel and others (2002) as follows:

41.033483° N, -111.899899° W	**10% PE in 50yr**	**2% PE in 50yr**
PGA	20.32	61.71
0.2 sec SA	49.42	142.85
1.0 sec SA	17.50	59.27

Given the above information, earthquake ground shaking is a high risk to the site. The hazard from earthquake ground shaking can be adequately mitigated by prudent design and construction.

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.

Trench 2 at the site exposed evidence for five west-dipping traces associated with faulting on the Weber segment of the Wasatch fault zone. Trench 3 also exposed one east-dipping antithetic fault trace. Fault locations are shown on Figure 4, based on trenching and air photo evidence. We infer total displacement across faults F1 through F5, based on relative elevations of the upper contact of the lacustrine sequence in trenches 1 and 2 (4,785 feet to 4,735 feet), to be a total of about 50 feet. Net displacement across the entire fault zone is between 45 and 50 feet (Figure 9, cross section A-A'). The fault displacement likely represents multiple post-Lake Bonneville events. DuRoss and others (2009) prefer a scenario of six Holocene events, which would give an average per-event displacement of about 8 feet.

Given the above, the risk from surface faulting is high at the site. Based on the results of this investigation and our current understanding that surface fault rupture and deformation tend to follow past patterns, we recommend a non-buildable (setback) zone around the fault traces as shown on Figure 4 (shaded in red). Table 2 shows relevant data and calculated setbacks for each fault. Setback distances are based on guidelines in Christenson and others (2003).

The fault setback for the downthrown block side of the fault is calculated using the following formula:

$$S = U (2D + F/\tan\Theta)$$

where:

S = Setback distance from active faults

U = Criticality factor (2.0 for IBC class R residential structures)

D = Fault displacement per event in feet (assumed to be 8 feet if not measurable)

F = Maximum depth of footing or subgrade portion of the building (assumed to be 10 feet)

Θ = Dip of the fault

The fault setback for the upthrown block side of the fault was calculated using:

$$S = U (2D)$$

Table 2. *Fault data and setback calculations.*

Trench	Fault	Station	Dip	Offset (feet)	Dip Direction	Calculated Downthrown Setback	Calculated Upthrown Setback	Downthrown Setback Distance	Upthrown Setback Distance
2	F1	0+7.2	52	3.0	West	20.7	9.0	20.7	15.0
2	F2a	0+15.8	55	0.5	West	12.0	1.5	15.0	15.0
2	F2b	0+17.7	61	8.0	West	32.3	24.0	32.3	24.0
2	F3	0+51.3	63	3.5	West	18.1	10.5	18.1	15.0
2	F4	0+61.6	58	2.6	West	17.2	7.8	17.2	15.0
2	F5	1+4.3	70	3.5	West	16.0	10.5	16.0	15.0
3	AF1	2+18.7	59	1.6	East	13.8	4.8	15.0	15.0

Minimum fault setback is 15 feet where calculated distances are less than 15 feet (Table 2). No structure intended for human occupancy should be located in the setback zones 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.

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 (earthquake ground shaking) accelerations, groundwater conditions, and presence of susceptible soils.

Sandy lacustrine deposits possibly susceptible to liquefaction are present in the upper 30 feet of the site subsurface, and the site is in an area of potentially strong shaking (as discussed in the Earthquake Ground Shaking Section above). However, groundwater at the site appears to be greater than at least 30 feet deep, and possibly more than 50 feet deep in the upper portion of the development. Based on the above, we rate the hazard from liquefaction as low, although the risk may vary locally depending on factors such as perched groundwater.

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 a normal fault. The main trace of the Weber segment of the WFZ divides the upper and lower parts of the site. Most of the lots on Figure 4 are on the upthrown fault side and therefore have a low risk from tectonic deformation. However, lots 101 and 102 are adjacent to the downthrown fault side and could experience a few degrees of tilting in a large-magnitude surface-faulting earthquake on the Weber segment. Tectonic deformation is typically not a life safety issue, but could cause damage that requires expensive repairs such as tilted foundations and altered sewer and water-line gradients.

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, the risk to the subject property from seismic seiches is rated as low.

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 cross the site or were evident during our reconnaissance. However, several ephemeral drainages enter the site from higher slopes to the east, some of which continue across the property to the southwest (Figure 4). These drainages are likely only active following cloudburst storms and spring snowmelt and may not be consistently active, but could cause produce small flash floods. We therefore rate the hazard from stream flooding as moderate. Although the drainage pattern has been altered by man-made activities since 1937, site hydrology and runoff should be addressed in the civil engineering design and grading plan for the Project.

Shallow Groundwater
No springs are shown on the topographic map for the Kaysville quadrangle at the site and no springs were observed during our site reconnaissance. No groundwater was encountered in any of the borings conducted by GSH at the site to depths of from 33 to 50 feet. No groundwater was also encountered in any of the test pits or trenches conducted for this report. We anticipate the depth to groundwater to be greater than 50 feet in the area in the upper part of the site, and between 30 and 50 feet in the lower part. However, it is possible that small seasonal seeps may be found in steep slope areas that are from perched conditions. Given the above, we rate the risk from shallow groundwater as low, although the risk may vary locally depending on factors such as perched groundwater.

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.

The steep west-facing escarpment formed by faulting along the Weber segment of the WFZ, which divides the proposed development, is an active mass-wasting zone prone to landsliding. Three scarp degradation-related landslides are north of trench 1 in this area (Figure 4, in gold). A possibly historical slide is also east of the site, as mapped on Figure 2 and discussed above. Although this offsite failure is not expected to have a direct slope stability impact (because it does not appear to extend into the site), it does suggest a similar potential for slope instability. Steep slope areas at the site prone to potential instability are demarked by a thick, black, dashed line on Figure 4. The steep slope areas impact lots 103 through 109, and 201 through 203 on the west; and lots 314 through 324 and lot 211 on the east.

Given all of the above, we rate the risk from landsliding at the site as high in areas in or adjacent to landslide-prone steep slopes (Figure 4, discussed above). We recommend stability of these slopes be evaluated in a geotechnical engineering evaluation prior to building based on site specific data and subsurface information included in this report. Recommendations for reducing the risk from landsliding should be provided if factors of safety are determined to be unsuitable, including setbacks from the steep slope areas as determined from the stability evaluation. Care should also be taken that site grading does not destabilize slopes in this area without prior geotechnical analysis and grading plans, that proper drainage is maintained, and no non-engineered cuts are made in slope toes. A future subsurface investigation may be needed to evaluate the landslides along the fault zone if existing data are deemed insufficient to evaluate slope stability.

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.

Several small inactive drainages enter the eastern part of the property. The two northernmost drainages head slightly above the Bonneville shoreline and have small associated alluvial fans at their mouths (labeled 1 and 2, Figures 3 and 4). A third drainage swale is to the south that heads below the shoreline and also has a small alluvial fan (labeled 3, Figures 3 and 4). The fan size and morphology for all three drainages decreases southward. Further southward, the remaining drainages entering the site appear to be small erosional gullies that head below the Bonneville shoreline (Figures 3 and 4). These drainages have no associated alluvial fans, and some continue to the southwest across the property. Based on surface observations and test pit data, GBES (2008) concluded a debris flow hazard existed at the current proposed development only on corresponding alluvial fan 2 on Figure 4. Our surface observations and test pit data also indicate a hazard also exists on alluvial fan 1. Alluvial fan 3 does not appear to have any evidence for Holocene activity, and is the smallest of the three with the weakest morphology. We note that evidence for discrete flows was only observed in test pit 1 near the distal end of alluvial fan 1 (Figures 4 and 5A).

Figure 1 shows the drainage basin areas and estimated drainages for alluvial fans 1 through 3, along with their estimated area and drainage lengths. Drainages 1 and 2 are assumed to head at the Bonneville shoreline for purposes of calculating the drainage length. Drainage basins 1 and 2 are 6.4 and 12.6 acres in size (Figure 1), which would be below the statistical threshold for volume estimates using VanDine's (1996) drainage basin area correlation. However, Hungr and others (1984) provides a correlation to estimate design magnitude debris flow volumes based on drainage length (empirical bulking). Based on our observations, the drainages are in loose lacustrine sediments and soil over weathered schist bedrock, which would be a Hunger and others' (1984) Channel Type B drainage with a corresponding sediment bulking factors of 2-4 yd^3/ft. Giraud (2005) confirms bulking rates for intermittent and ephemeral streams are generally lower than rates for perennial streams. Similar ephemeral drainages have showed bulking rates

of 1.5 to 5 yd^3/ft (Mulvey and Lowe, 1992; McDonald and Giraud, 2002). We believe an average bulking rate of 3 yd^3/ft is appropriate. Drainage lengths for basins 1 and 2 (Figure 1) are 596 feet and 669 feet, respectively. At a bulking rate of 3 yd^3/ft, the drainages would have design magnitude debris flow volumes of 1,788 and 2,007 cubic yards. GBES (2008) estimated a design magnitude debris flow volume for alluvial fan 2 of 1,400 cubic yards based on a drainage length of 700 feet and a bulking rate of 2 yd^3/ft.

Based on the above, we rate the hazard from debris flows as moderate. Lots 206 through 208, 211, and 322 through 324 have the highest risk. However, given the small calculated volumes and assuming a design flow is further attenuated by deposition in other areas, it appears likely that only a small amount of debris and muddy water would reach these lots. All the fan apexes are also offsite to the east, which may make offsite mitigation unfeasible. In lieu of offsite mitigation, we therefore recommend raising the building pads for the high-risk lots (noted above) by at least 3 feet (roughly twice the average past flow thickness observed in test pit 1). Alternatively, a combination of raised building pads, deflection berms, and routing channels may be employed as long as such measures are synergistic and achieve an equivalent total. We further recommend that: (1) homes on high-risk lots not have any upslope-facing, subgrade entryways that could pose pathways for debris and water to enter basement areas; (2) deflection berms (if employed) not have penetrations (such as driveways) that would render them ineffective; and (3) roads on the east side of the property be designed by the Project civil engineer to efficiently route water and debris away from homes and into the storm drain system. Care should be taken that surface debris and floodwater is not directed offsite or into other areas lacking protective measures.

Rock Fall

Scattered weathered outcrops of schist of the Precambrian Farmington Canyon Complex were observed in slopes east of the property and in one area in the southeast part of the site near test pit 5. However, no boulders from rock falls were observed at the site and upslope outcrops appeared to have low relief and likely incapable of generating large rock fall boulders due to their composition. Based on the above, we rate the hazard from rock falls as low. GBES (2008) also did not identify any potential rock fall hazard areas within the Project boundaries.

Radon

Radon comes from the natural (radioactive) breakdown of uranium in soil, rock, and water and can seep into homes through cracks in floor slabs or other openings. The site is located in areas of "Moderate" and "High" radon-hazard potential (Black and Solomon, 1996). A moderate hazard potential indicates geologic factors are favorable for indoor radon concentrations between 2 and 4 picocuries per liter of air, whereas a high hazard potential indicates indoor-radon concentrations may exceed 4. Actual indoor radon levels can be affected by non-geologic factors such as building construction, maintenance, and weather. Long-term indoor testing following construction is the best method to characterize the radon hazard and determine if mitigation measures are required.

Swelling and Collapsible Soils
Surficial soils that contain certain clays can swell or collapse when wet. Given the subsurface soil conditions observed at the site, we do not anticipate that any soils susceptible to swelling or collapse will be present. However, a geotechnical engineering evaluation should be performed to address soil conditions and provide specific recommendations for site grading, subgrade preparation, and footing and foundation design.

CONCLUSIONS AND RECOMMENDATIONS

Geologic hazards posing a high relative risk to the site are earthquake ground shaking, surface fault rupture, landslides, and radon. Moderate-risk hazards include tectonic deformation, stream flooding, and debris flows. The following recommendations are provided to address these hazards:

- Proposed homes should be designed and constructed to current seismic standards to reduce the potential ground-shaking hazard.

- No structures intended for human occupancy should be located in the fault setback zones delineated 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.

- A design-level geotechnical engineering study should be conducted prior to construction to: (1) address soil conditions at the site for use in foundation design, site grading, and drainage; (2) provide recommendations regarding building design to reduce risk from seismic acceleration; and (3) evaluate stability of slopes at the site, including providing recommendations for reducing the risk of landsliding if the factors of safety are deemed unsuitable. A future subsurface investigation should be conducted prior to building on

- The building pads for lots 206 through 208, 211, and 322 through 324 should be raised by at least 3 feet to reduce risk from debris flows and floods, and homes on these lots should include no upslope-facing, subgrade entryways. Deflection berms and/or routing channels may also be employed, but care should be taken that all combination measures are synergistic and achieve the same level of risk reduction as provided by the raised building pads.

- Site grading and drainage should be addressed in the civil engineering design for the development. Roads should be designed to route water and debris away from homes and into the storm drain system. Routing channels and berms, if employed, should be designed by the Project civil engineer, based on data included in this report, and shown on the site grading plan. All raised building pads should also be shown on the grading plan. We recommend that we review the final grading plan once it has been prepared.

- A future subsurface investigation will be needed if additional data is needed to evaluate the landslides along the fault zone north of trench 1.

The site appears suitable for the proposed development given the scope of this report and findings herein, and assuming our recommendations provided above are followed.

Availability of Report

The report should be made available to architects, building contractors, and in the event of a future property sale, real estate agents and potential buyers. This 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.

LIMITATIONS

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.

It has been a pleasure working with you on this project. Should you have any questions, please call.

Sincerely,
Western GeoLogic, LLC



Bill. D. Black, P.G.
Senior Engineering Geologist

Reviewed by:



Craig V. Nelson, P.G.
Principal Engineering Geologist

ATTACHMENTS

Figure 1. Location Map (8.5”x11”)
Figure 2. Geologic Map (8.5”x11”)
Figure 3. 1937 Air Photo (11”x17”)
Figure 4. Site Plan (22”x25.5”)
Figures 5A-C. Test pit Logs (three 11”x17” sheets)
Figures 6A-B. Trench 1 Log (two 11”x17” sheets)
Figures 7A-C. Trench 2 Log (three 11”x17” sheets)
Figures 8A-D. Trench 3 Log (four 11”x17” sheets)
Figure 9. Cross Sections (one 11”x17” sheet)
Appendix. GeoStrata (2016) review letter

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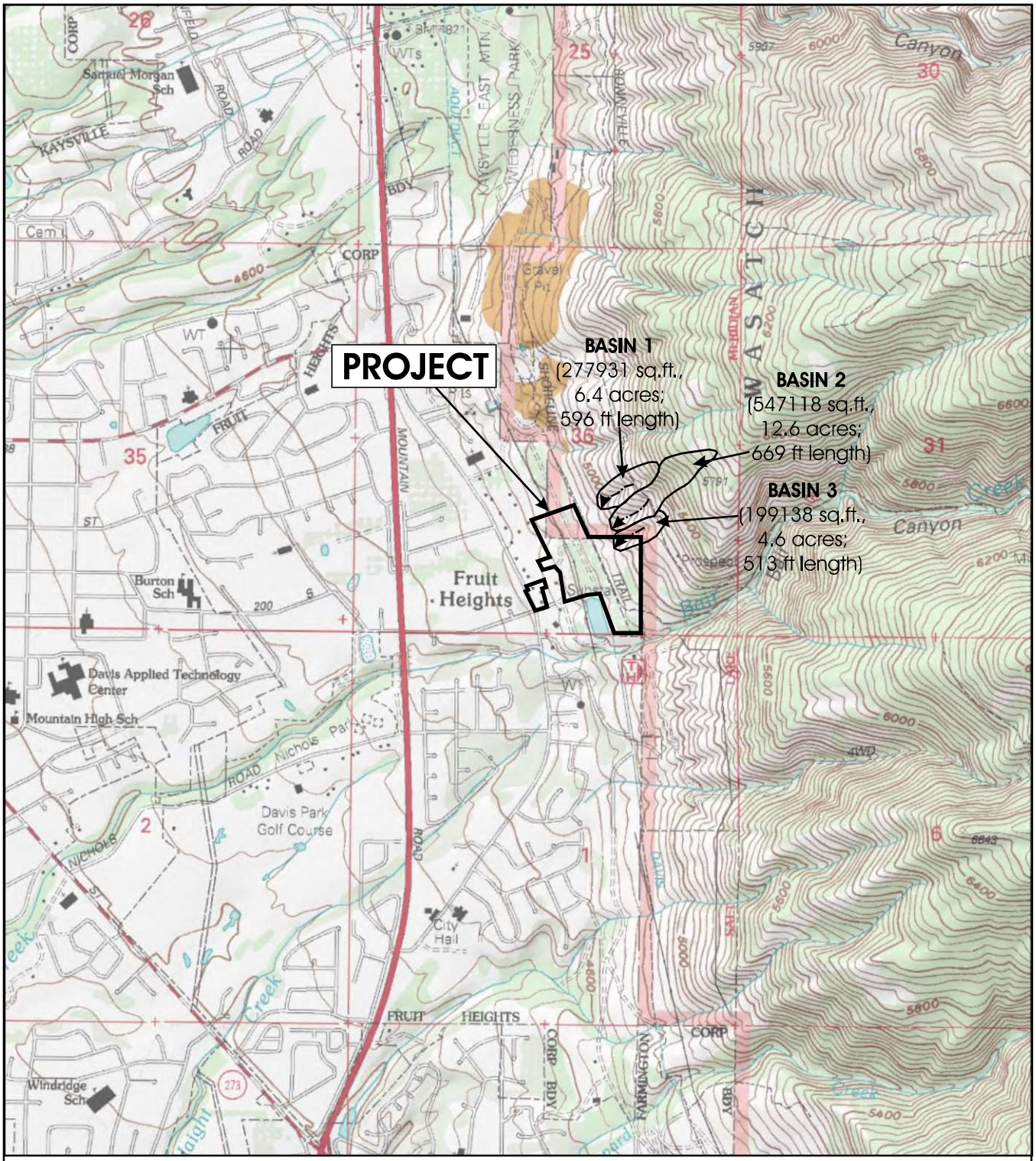
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Source: U.S. Geological Survey 7.5 Minute Series Topographic Maps, UT - Kaysville, 1998.

WESTERN GEOLOGIC

N
W E
S

0 1000 2000 feet

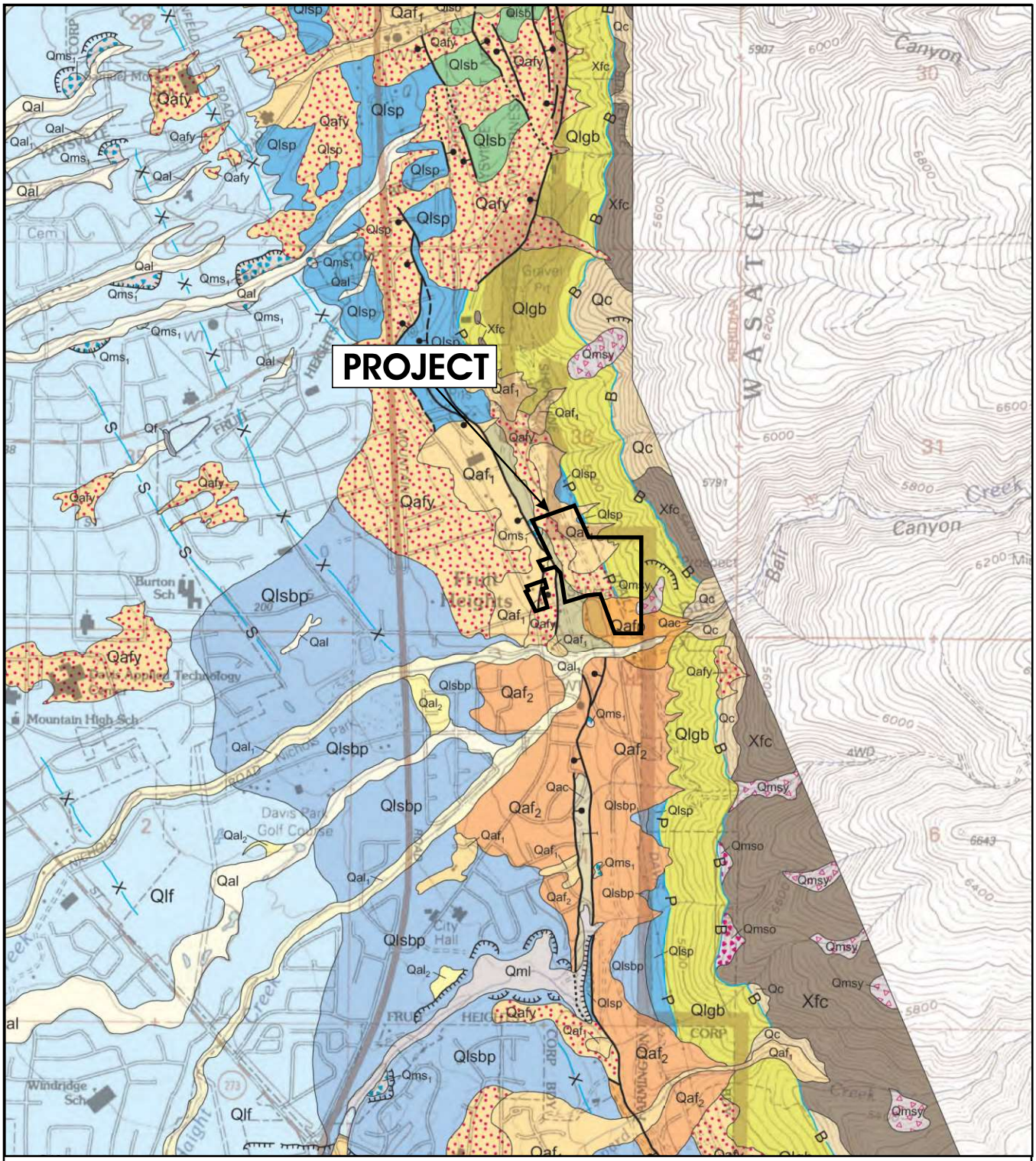
Scale 1:24,000
(1 inch = 2000 feet)

LOCATION MAP

GEOLOGIC HAZARDS EVALUATION

Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 1



Source: Solomon, 2007.

N
W E
S

0 1000 2000 feet

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

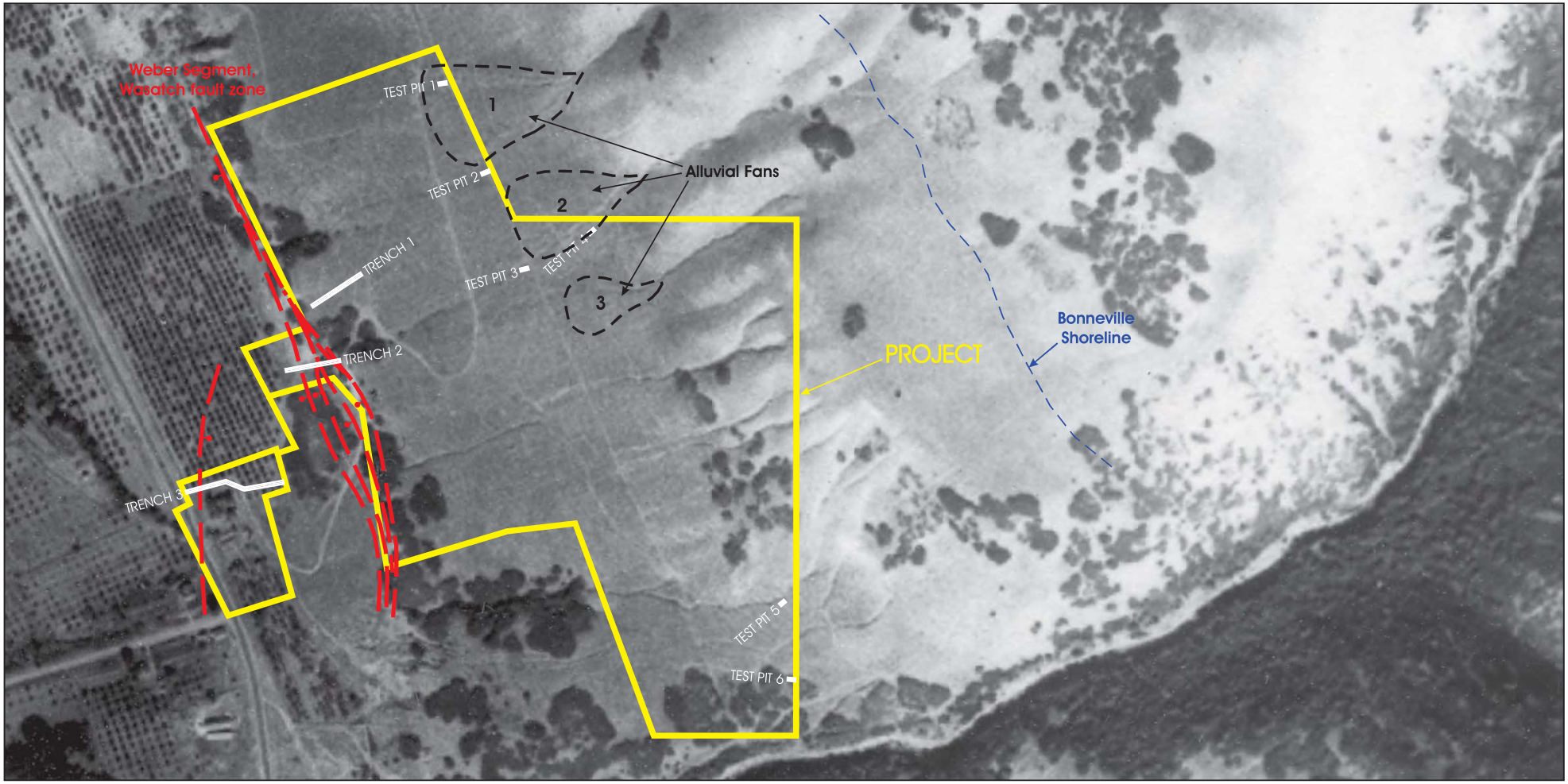
WESTERN GEOLOGIC

GEOLOGIC MAP

GEOLOGIC HAZARDS EVALUATION

Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 2



Air photo from U.S. Geological Survey, frame 10-AAK3-41, original scale 1:20,000.

N
W E
S

0 125 250 feet

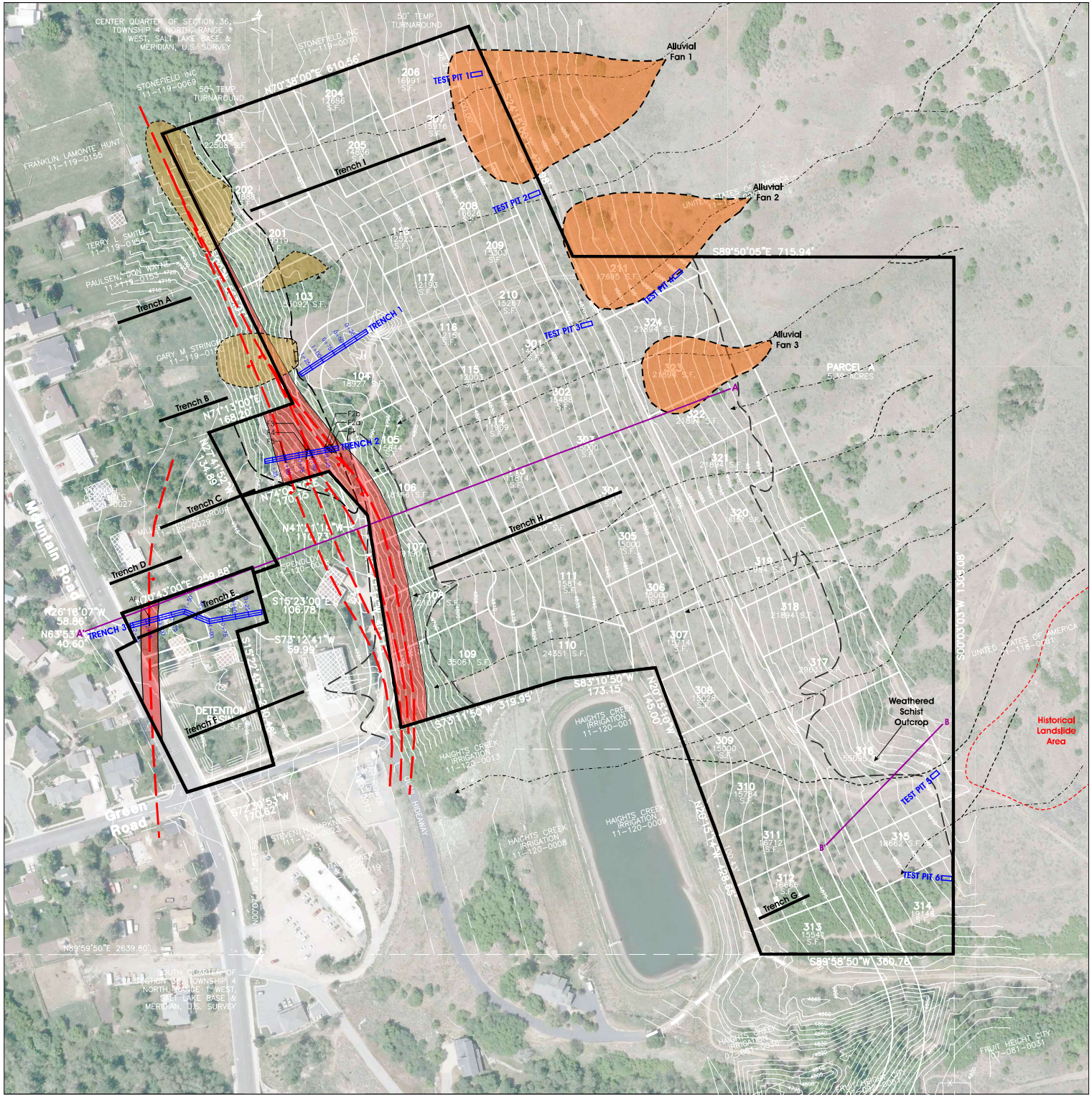
Scale 1:3,000
(1 inch = 250 feet)

WESTERN GEOLOGIC

1937 AIR PHOTO

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 3



EXPLANATION

- Fault, bar and ball on downthrown side, dashed where approximately located.
- Setback zone. No structures designed for human occupancy.
- Great Basin Earth Science (GBES, 2008) trench locations.
- Slope failures along fault scarp.
- Alluvial fans.
- Trenches and test pits, this study.
- 1937 ephemeral drainage courses.
- A—A' Slope cross section.
- Steep slope boundary.

N W E S

0 50 100 feet

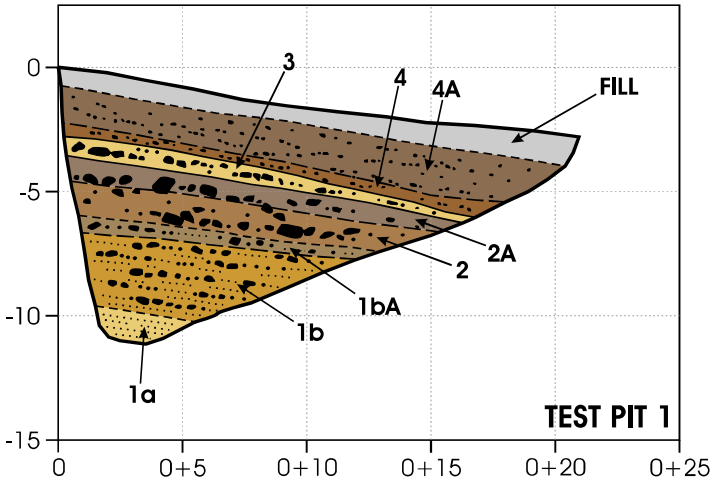
Scale 1:1,200
(1 inch = 100 feet)

Air photo base, site plan, topography, and surveyed trench and fault locations from Reeve & Associates, Inc.; Rock Loft Estates Subdivision preliminary plan dated May 12, 2016.

SITE PLAN

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 4



Scale in feet

UNIT DESCRIPTIONS

Unit 1. *Lacustrine sediments related to the transgressive stage of Lake Bonneville* - coarsening-upward sequence of sand and gravel.

1a. Pale brown, low to moderate density, poorly bedded, sand with gravel (SW) and trace cobbles.

1b. Brown, low to moderate density, poorly bedded, gravelly sand to sandy gravel (SW/GW) with trace rounded cobbles.

1bA. Paleosol A horizon formed in unit 1.

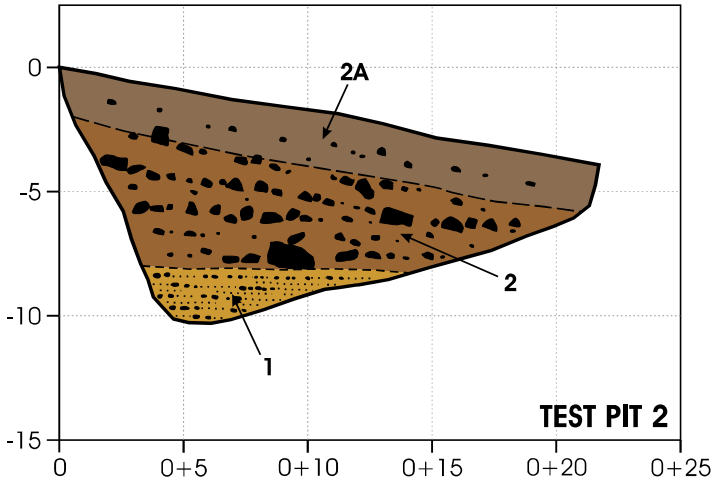
Unit 2. *Latest Pleistocene to Holocene alluvium* - Brown, moderate density, massive, sandy gravel (GW) with silt and cobbles; clasts subround to subangular with stage II carbonate.

2A. Paleosol A horizon formed in unit 2.

Unit 3. *Holocene debris flood deposit* - Pale brown, moderate density, sandy gravel (GW) with silt and cobbles.

Unit 4. *Holocene alluvium* - Brown to dark brown, low to moderate density, massive, root-penetrated, organic-enriched, gravelly sand (GW) with trace cobbles.

4A. Disturbed modern A-horizon soil, overlain by plowed zone.



Scale in feet

UNIT DESCRIPTIONS

Unit 1. *Lacustrine sand and gravel related to the transgressive stage of Lake Bonneville* - Brown, moderate density, poorly bedded, sandy gravel to gravelly sand (SW/GW) with trace rounded cobbles.

Unit 2. *Latest Pleistocene to Holocene alluvium* - Brown to dark brown, moderate density, massive, slightly root penetrated, slightly vesicular, sandy gravel to gravelly sand (GW/SW) with silt, cobbles, and boulders up to 3 feet across; clasts subangular to rounded with stage II carbonate.

2A. Modern A-horizon soil formed in unit 2.

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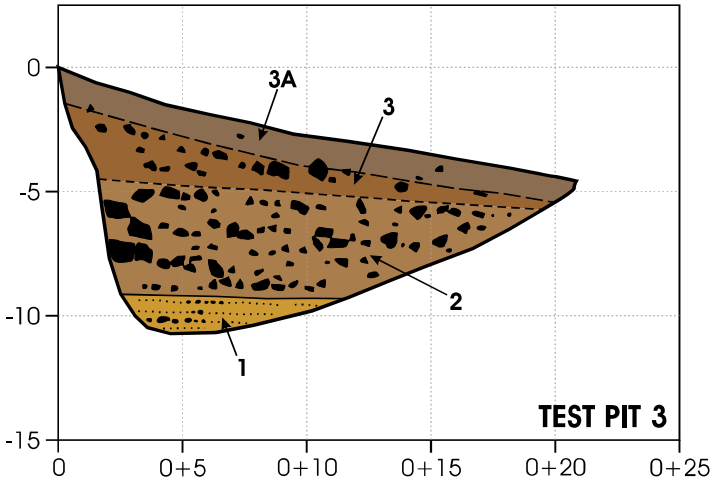
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Walls Logged, East to West

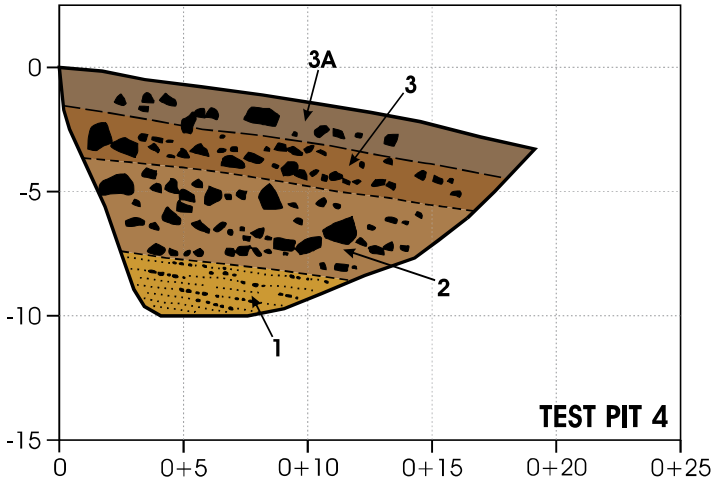
Test pits logged by Bill D Black, P.G. on
July 13, 2015
Reviewed by Craig V Nelson, P.G.

TEST PIT LOGS, SHEET 1

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 5A





UNIT DESCRIPTIONS

Unit 1. *Lacustrine sand and gravel related to the transgressive stage of Lake Bonneville* - Brown, low to moderate density, poorly bedded, gravelly sand (SW) with trace rounded cobbles.

Unit 2. *Latest Pleistocene to Holocene alluvium* - Brown, moderate density, massive, gravelly sand (SW) with cobbles and boulders; clasts subangular to subround with stage II carbonate.

Unit 3. *Holocene alluvium* - Brown to dark brown, moderate density, massive, sandy gravel (GW) with cobbles.

3A. Modern A-horizon soil formed in unit 3.

UNIT DESCRIPTIONS

Unit 1. *Lacustrine sand and gravel related to the transgressive stage of Lake Bonneville* - Brown, low to moderate density, poorly bedded, gravelly sand to sandy gravel (SW/GW).

Unit 2. *Latest Pleistocene to Holocene alluvium* - Brown, low to moderate density, massive, gravelly sand to sandy gravel (SW/GW) with silt, cobbles, and boulders up to 4 feet across; clasts subangular to subround with stage II carbonate.

Unit 3. *Holocene alluvium* - Brown to dark brown, low to moderate density, massive, slightly root penetrated, sandy gravel (GW) with cobbles.

3A. Modern A-horizon soil formed in unit 3.

WESTERN GEOLOGIC

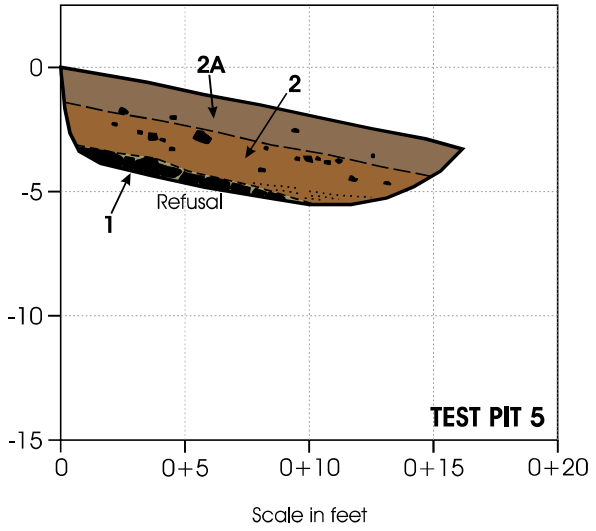
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Walls Logged, East to West

Test pits logged by Bill D Black, P.G. on July 13, 2015
Reviewed by Craig V Nelson, P.G.

TEST PIT LOGS, SHEET 2

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 5B

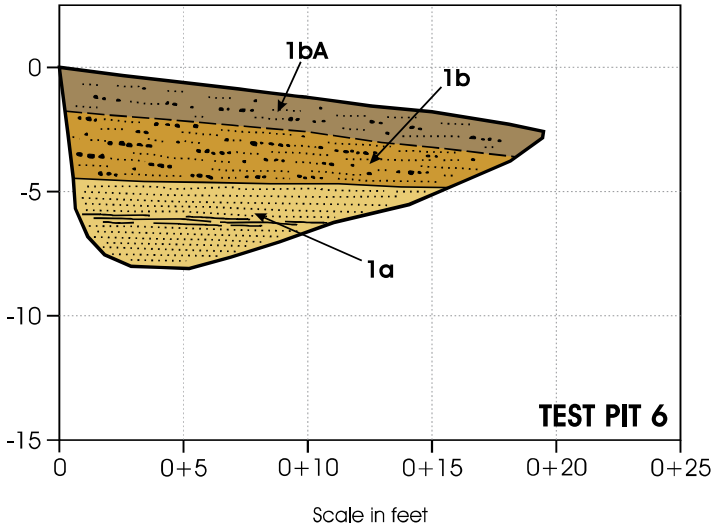


UNIT DESCRIPTIONS

Unit 1. *Fractured Precambrian Farmington Canyon Complex Schist Bedrock.*

Unit 2. *Slope colluvium* - Brown to brownish-gray, low to moderate density, generally massive, gravelly sand (SW) with trace cobbles; poorly bedded basal sand in south test pit end; clasts subround to subangular with stage II carbonate.

2A. Modern A-horizon soil formed in unit 2.



UNIT DESCRIPTIONS

Unit 1. *Lacustrine sediments related to the transgressive stage of Lake Bonneville* - coarsening-upward sequence of sand and gravel.

1a. Pale brown, low to moderate density, poorly to well bedded, sand with pea gravel (SW) and thin silt interbeds.

1b. Brown, low to moderate density, poorly bedded, gravelly sand (SW) with rounded small clasts up to 3".

1bA. Paleosol A horizon formed in unit 1b.

WESTERN GEOLOGIC

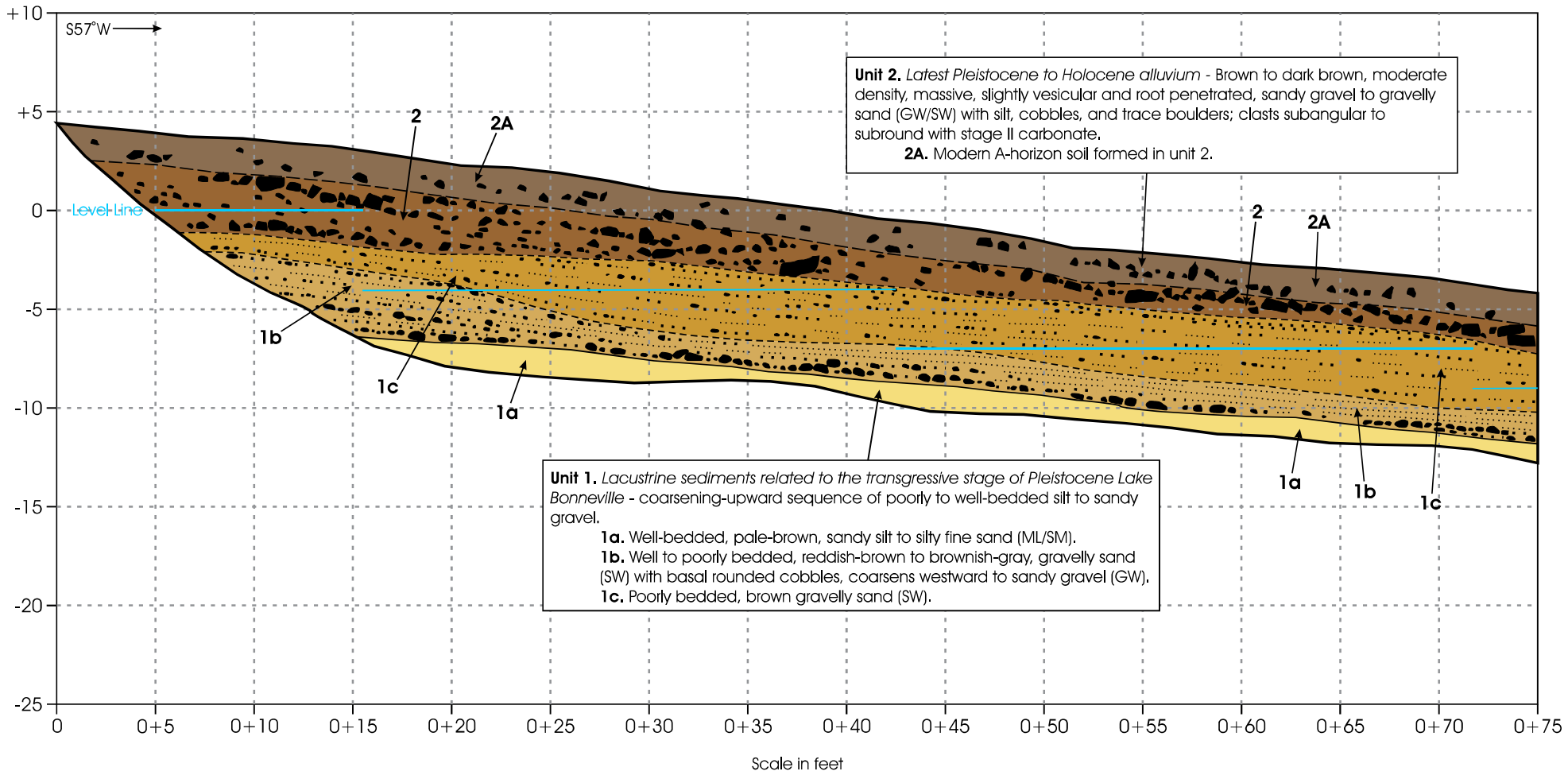
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Walls Logged, East to West

Test pits logged by Bill D Black, P.G. on July 13, 2015
Reviewed by Craig V Nelson, P.G.

TEST PIT LOGS, SHEET 3

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 5C



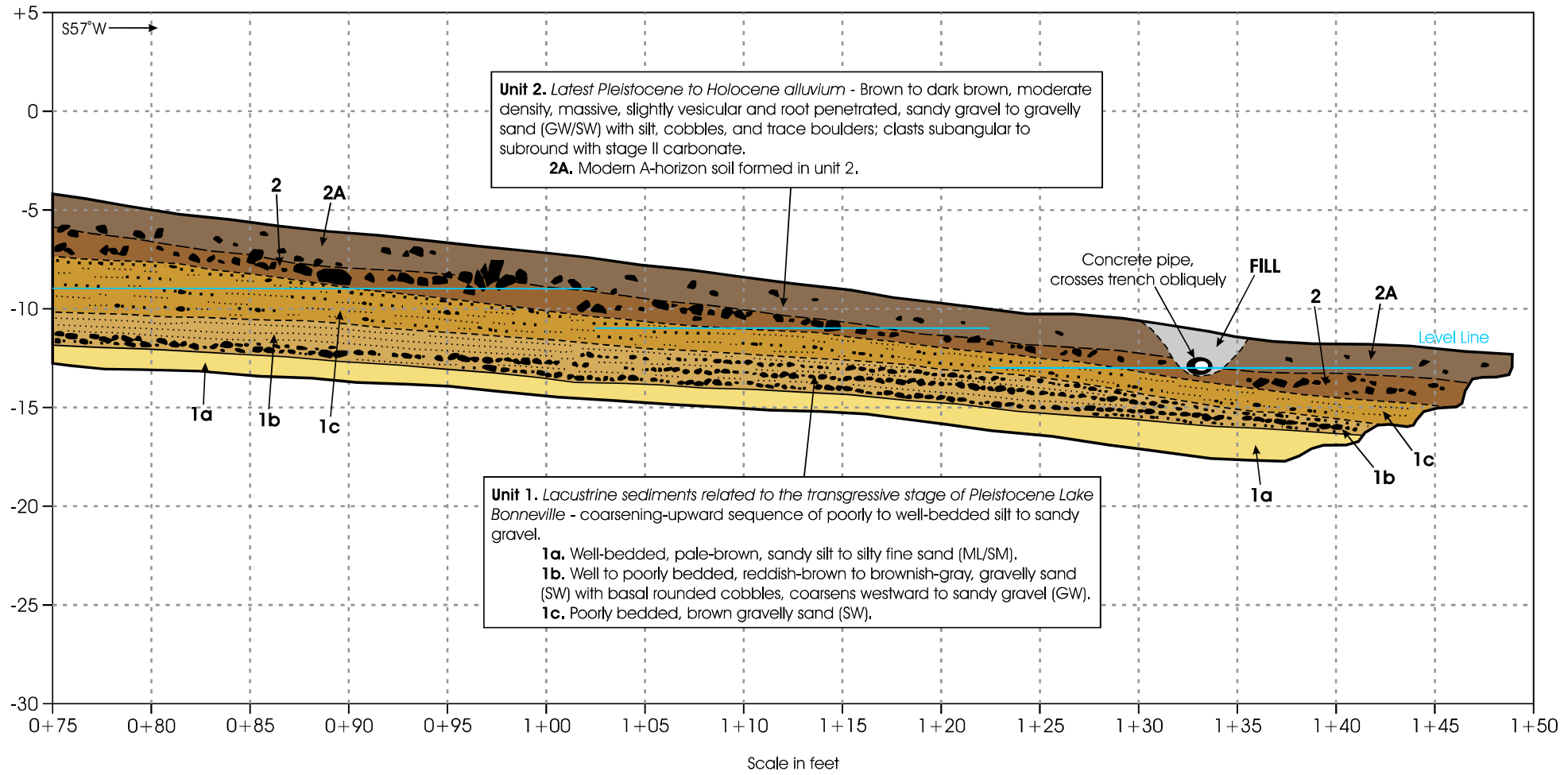
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Wall Logged, East to West

Trench logged by Bill D Black, P.G. on
July 14, 2015
Reviewed by Craig V Nelson, P.G.

TRENCH 1 LOG, SHEET 1

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 6A



SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Wall Logged, East to West

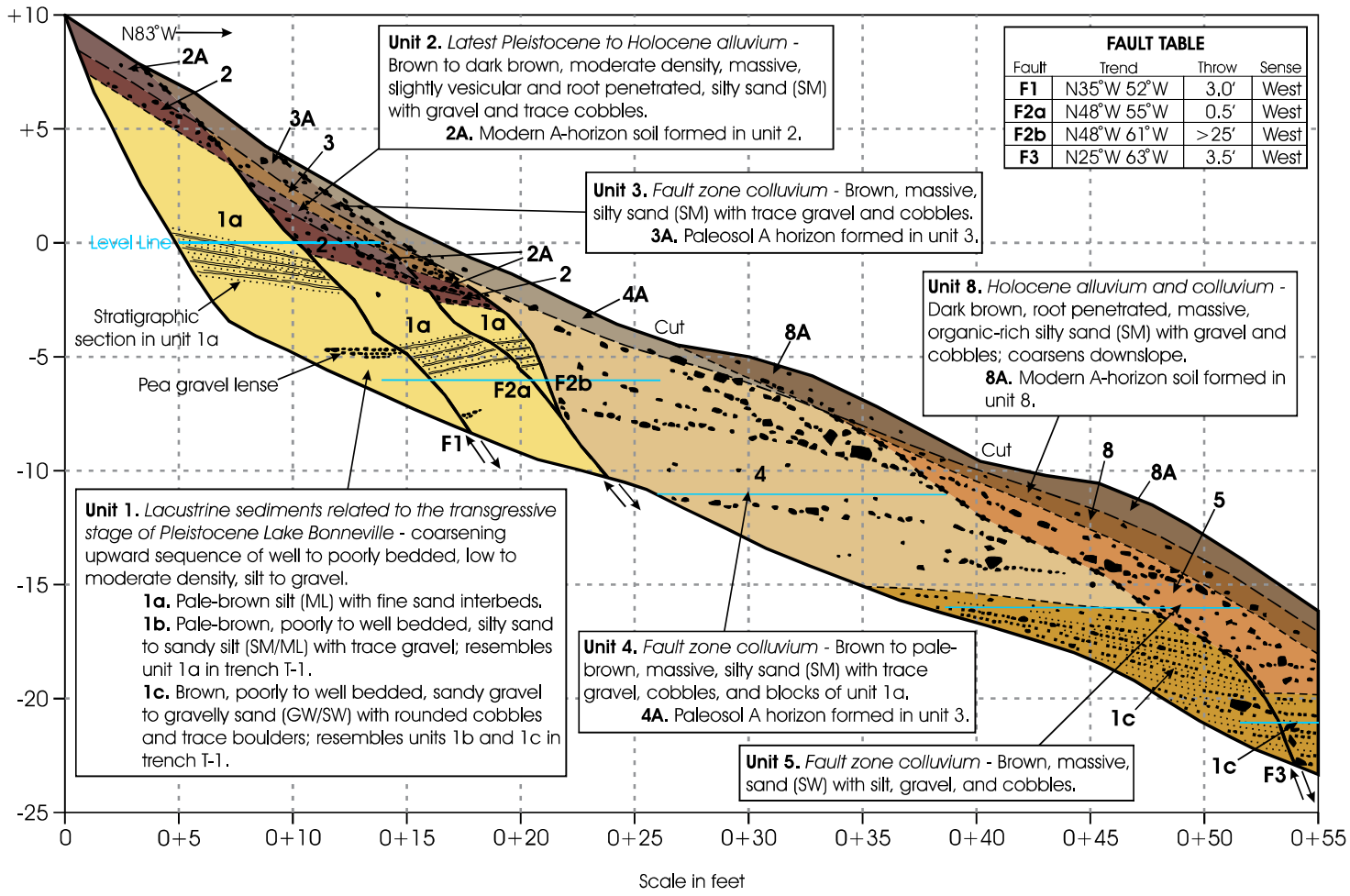
Trench logged by Bill D Black, P.G. on
July 14, 2015
Reviewed by Craig V Nelson, P.G.

WESTERN GEOLOGIC

TRENCH 1 LOG, SHEET 2

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 6B



FAULT TABLE

Fault	Trend	Throw	Sense
F1	N35°W 52°W	3.0'	West
F2a	N48°W 55°W	0.5'	West
F2b	N48°W 61°W	>25'	West
F3	N25°W 63°W	3.5'	West

SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Wall Logged, East to West

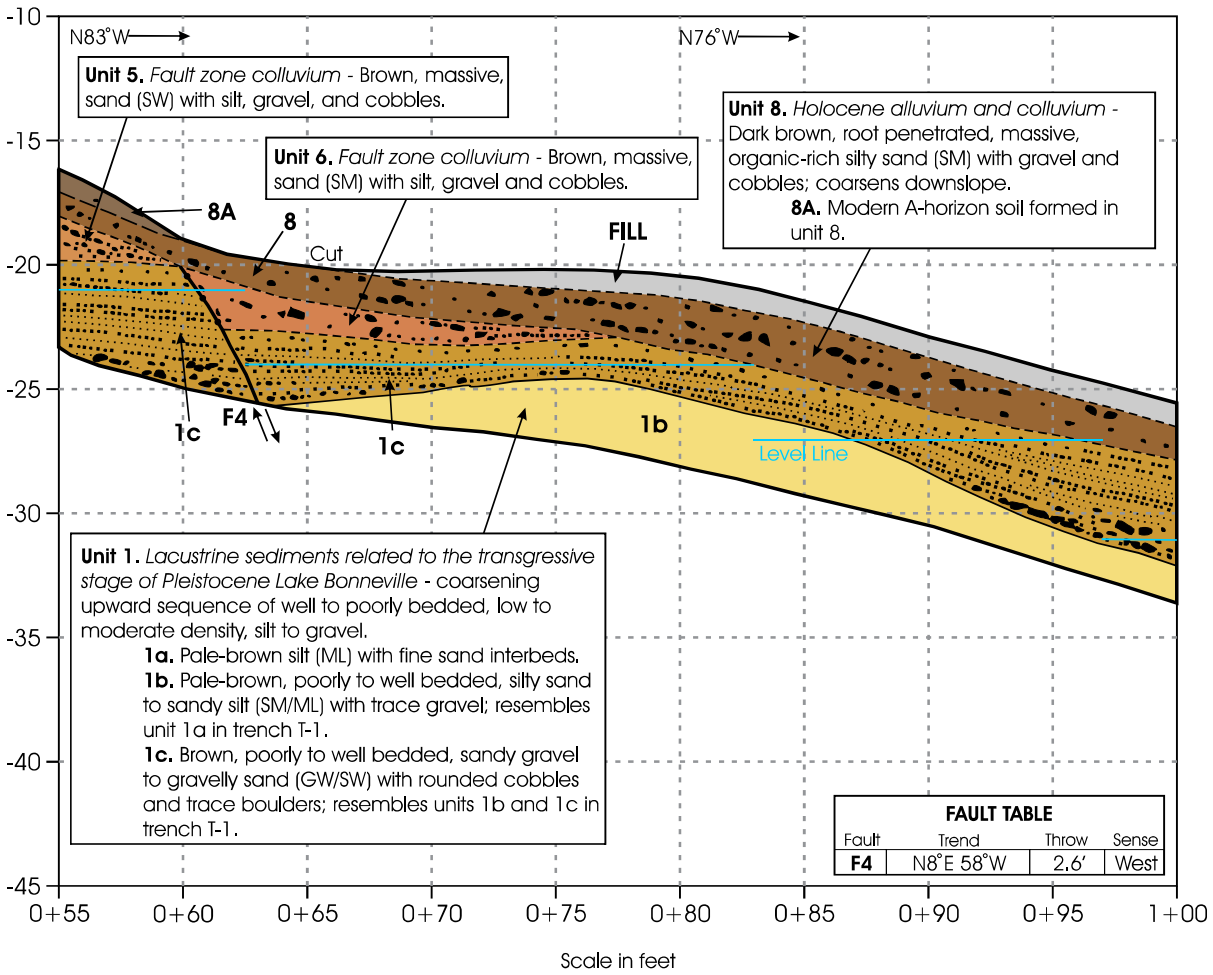
Trench logged by Bill D Black, P.G. on
July 15, 2015
Reviewed by Craig V Nelson, P.G.

WESTERN GEOLOGIC

TRENCH 2 LOG, SHEET 1

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 7A



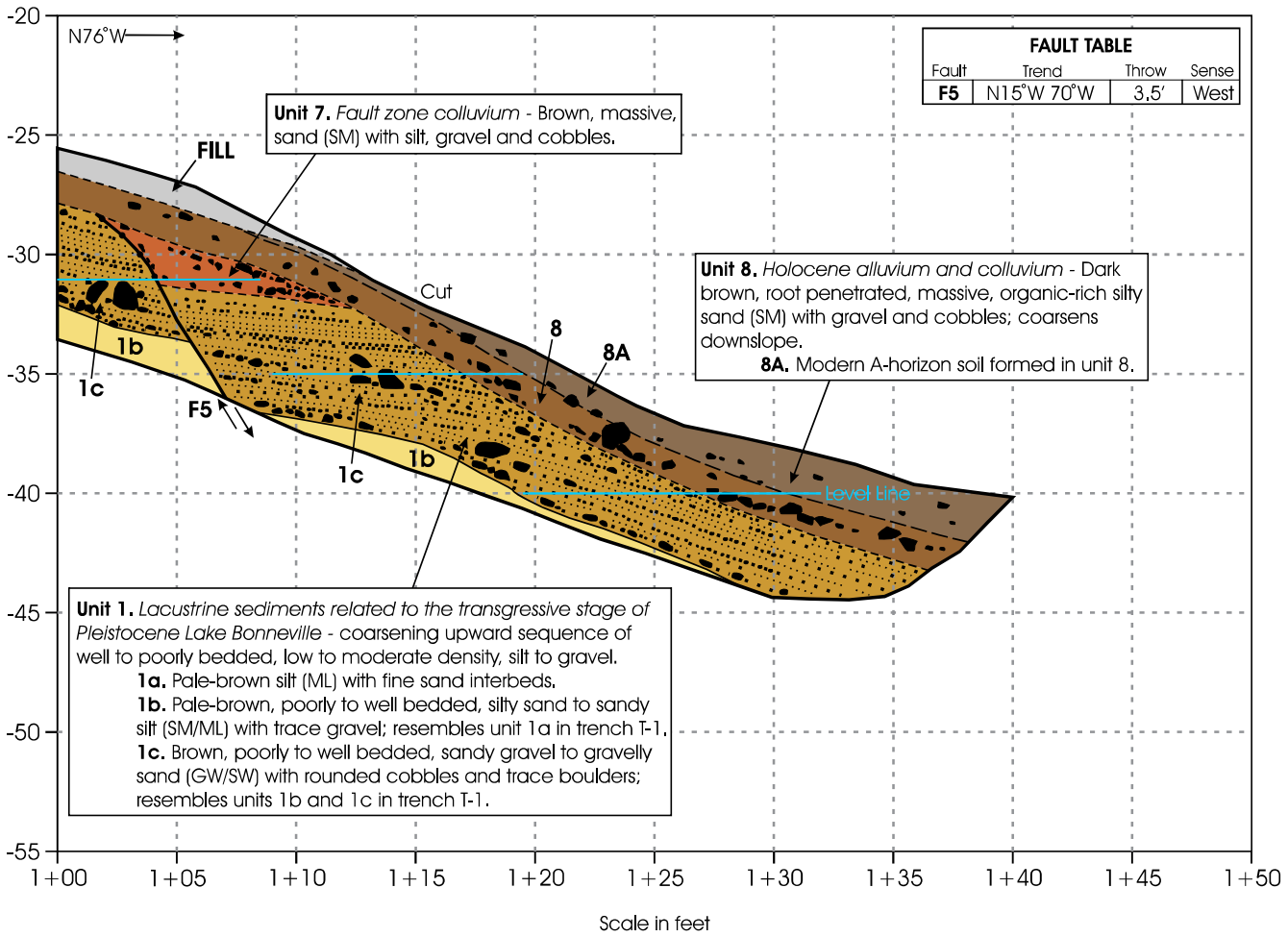
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Wall Logged, East to West

Trench logged by Bill D Black, P.G. on
July 15, 2015
Reviewed by Craig V Nelson, P.G.

TRENCH 2 LOG, SHEET 2

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 7B



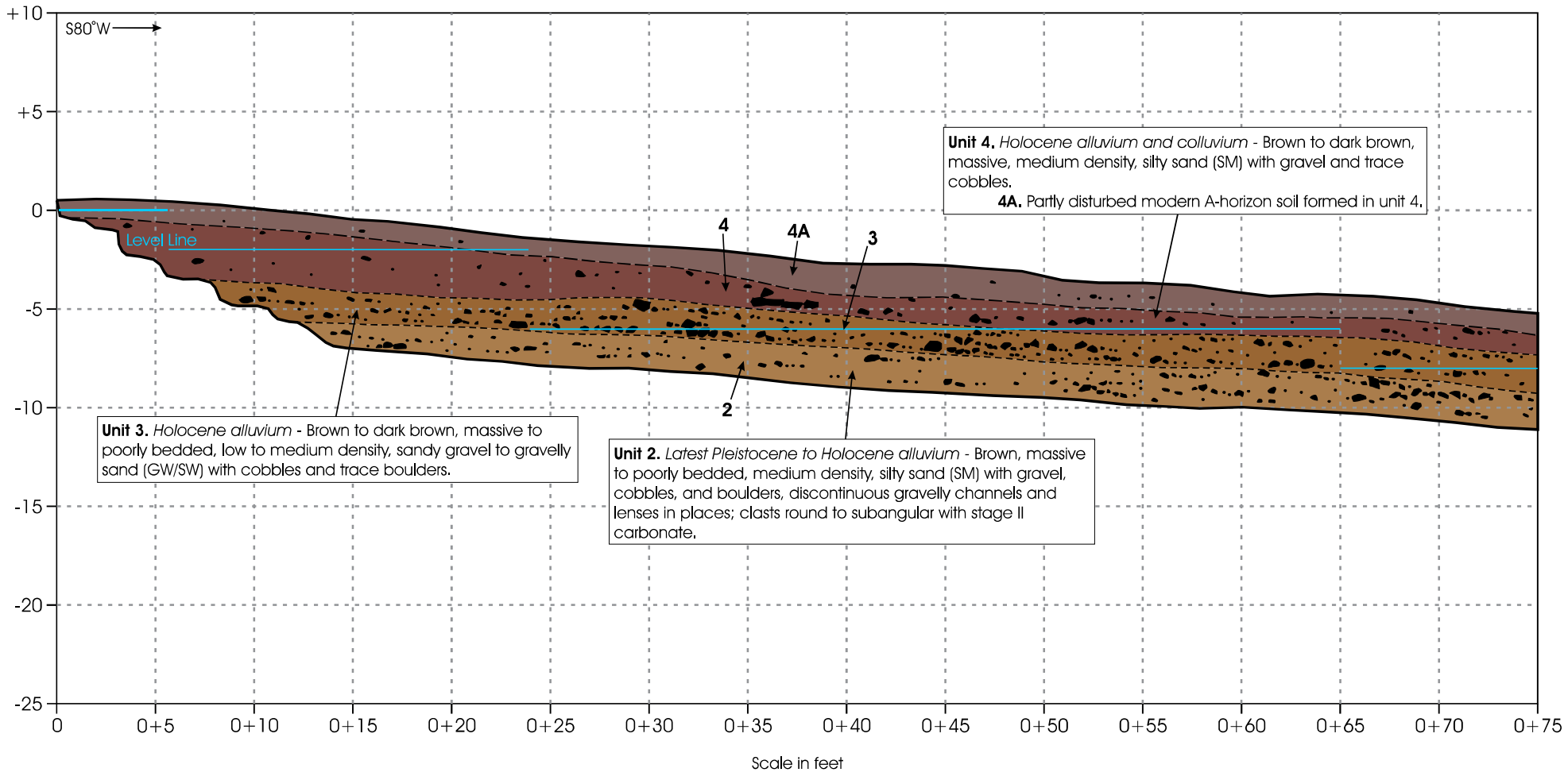
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Wall Logged, East to West

Trench logged by Bill D Black, P.G. on
July 15, 2015
Reviewed by Craig V Nelson, P.G.

TRENCH 2 LOG, SHEET 3

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 7C



WESTERN GEOLOGIC

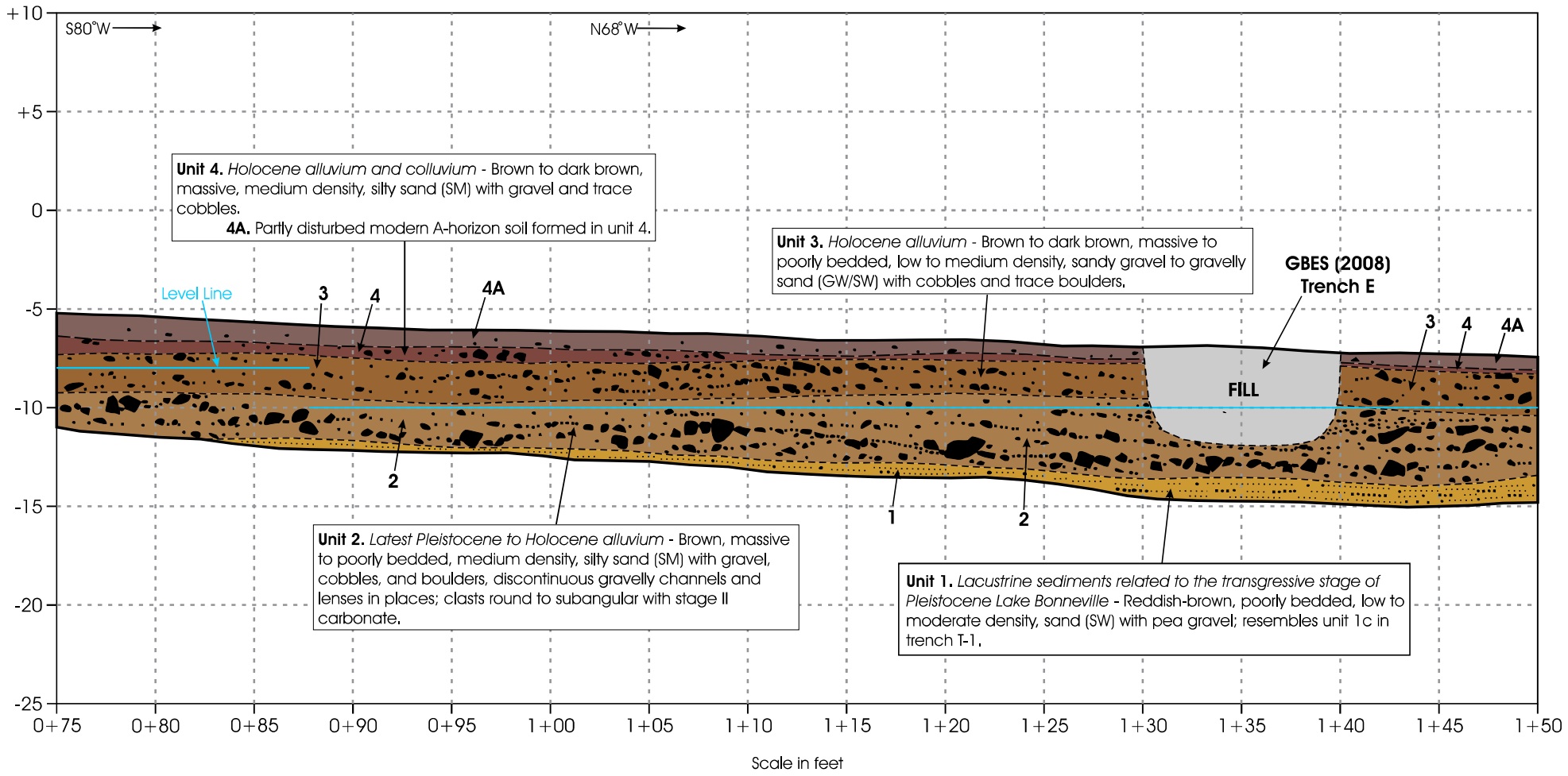
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Wall Logged, East to West

Trench logged by Bill D Black, P.G. on
July 16-17, 2015
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TRENCH 3 LOG, SHEET 1

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 8A



WESTERN GEOLOGIC

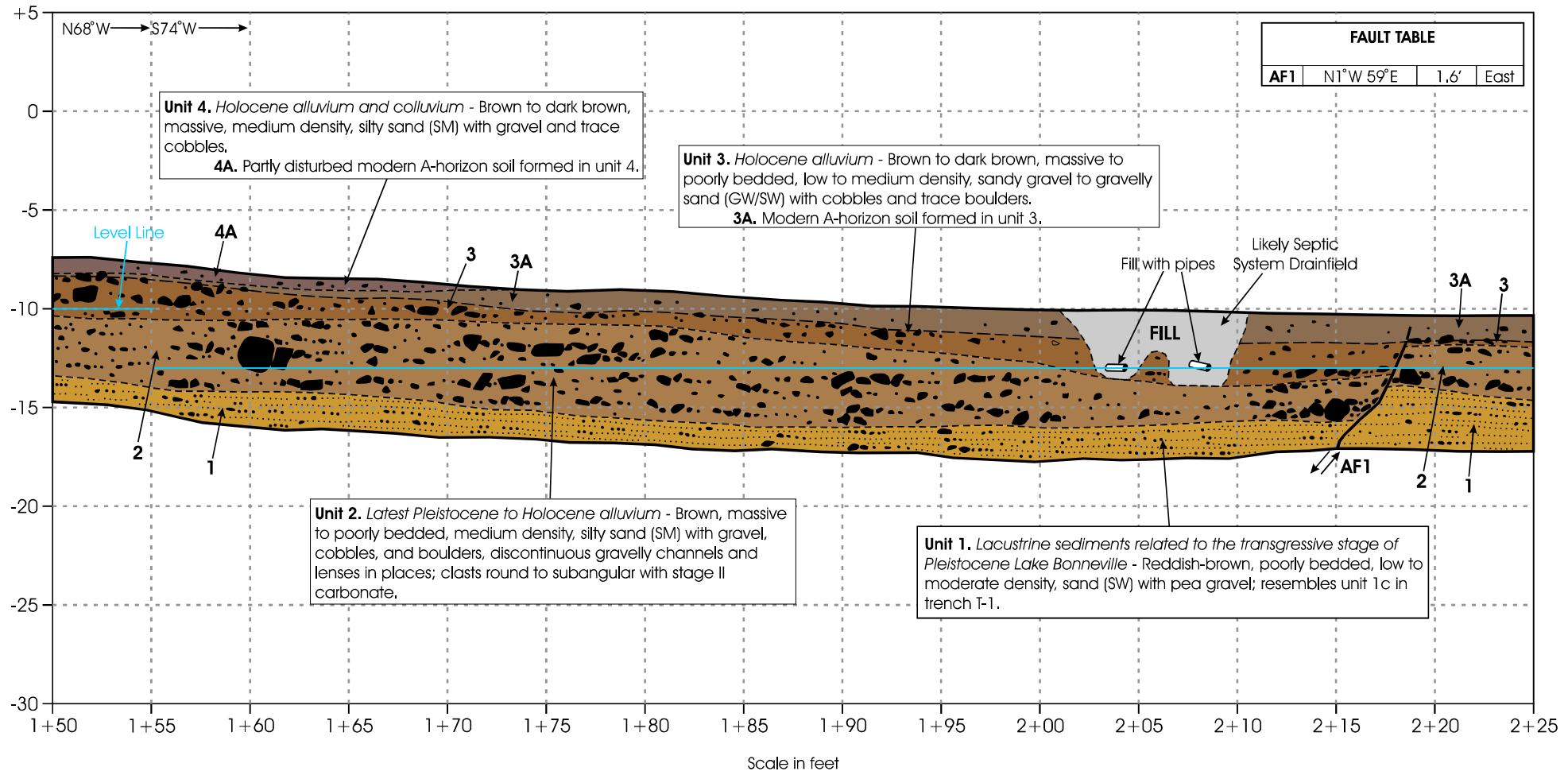
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Wall Logged, East to West

Trench logged by Bill D Black, P.G. on
July 16-17, 2015
Reviewed by Craig V Nelson, P.G.

TRENCH 3 LOG, SHEET 2

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 8B



SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Wall Logged, East to West

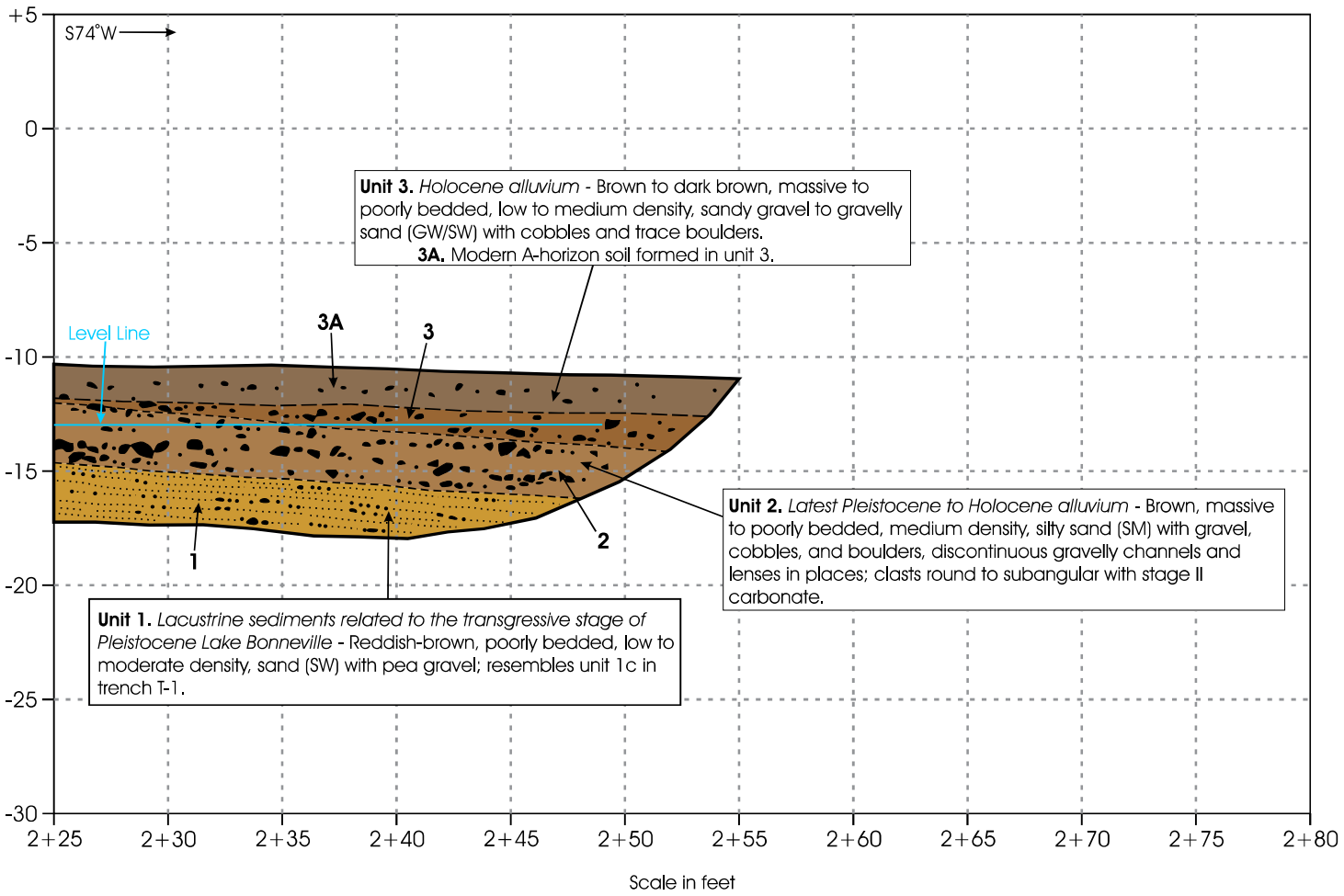
Trench logged by Bill D Black, P.G. on
July 16-17, 2015
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WESTERN GEOLOGIC

TRENCH 3 LOG, SHEET 3

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 8C



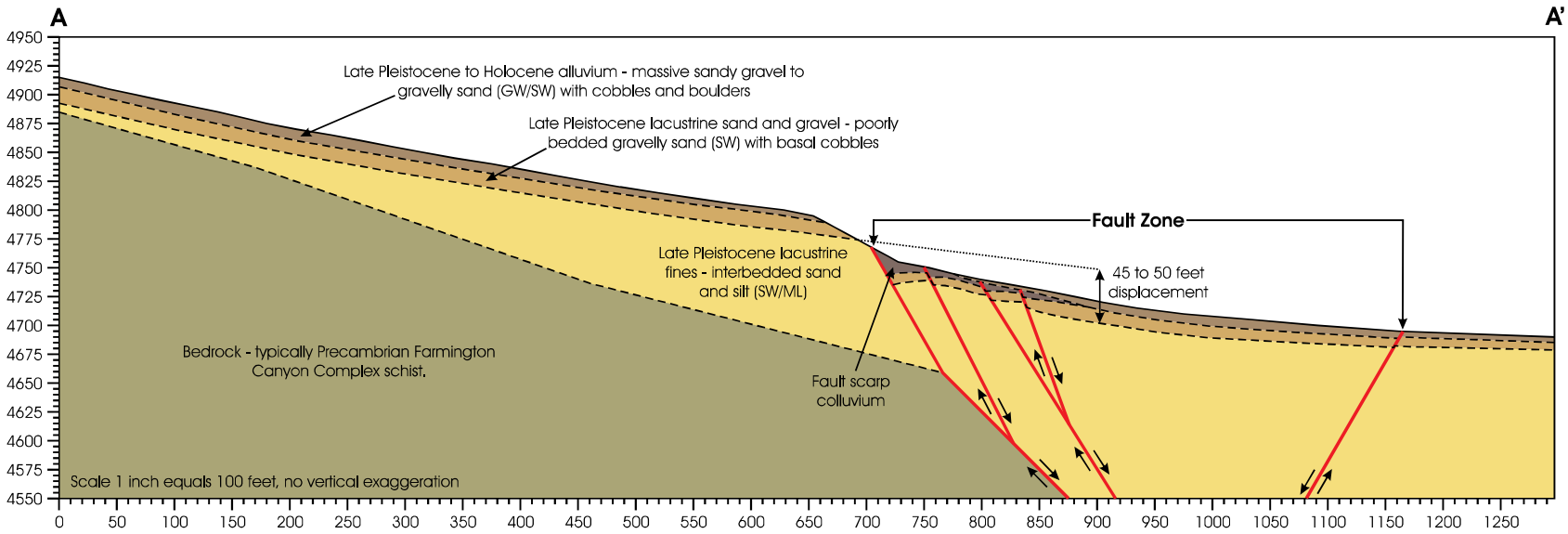
SCALE: 1 inch = 5 feet
(no vertical exaggeration)
South Wall Logged, East to West

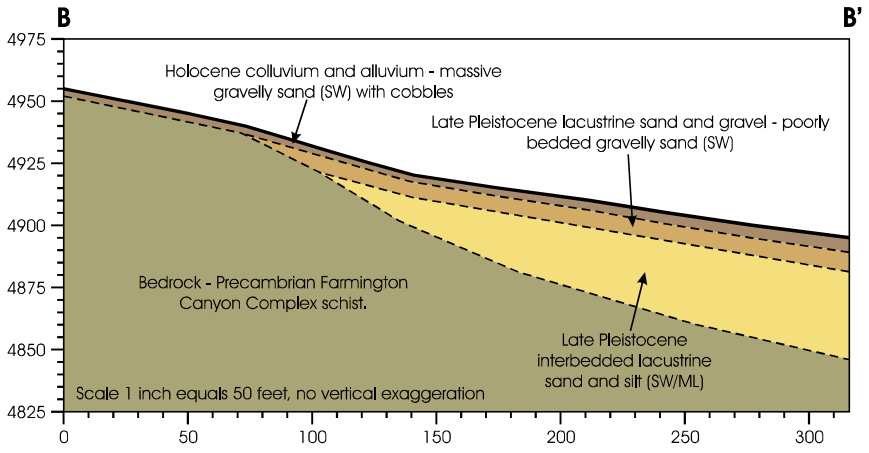
Trench logged by Bill D Black, P.G. on
July 16-17, 2015
Reviewed by Craig V Nelson, P.G.

TRENCH 3 LOG, SHEET 4

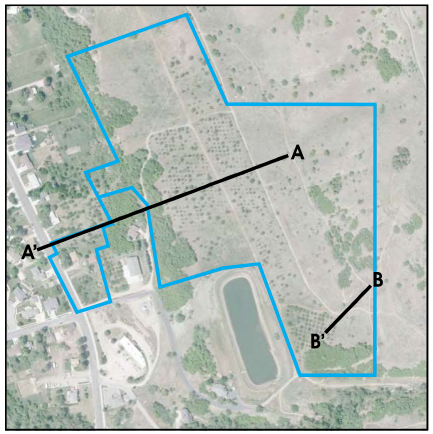
GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 8D





PROFILE LOCATIONS
(See also Figure 4)



WESTERN GEOLOGIC

CROSS SECTION

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 9

GeoStrata 14425 South Center Point Way, Bluffdale, Utah 84065
T: (801) 501-0583 ~ F: (801) 501-0584

To: **Mr. Brandon Green P.E.**
Fruit Heights City Engineer
910 South Mountain Road
Fruit Heights, Utah 84037



From: **GeoStrata, Timothy J. Thompson, PG**

Date: **March 17, 2016**

Subject: **Review of Response Geologic Hazards Evaluation Raymond Property About 175 South Mountain Road Fruit Heights, Utah: dated August 15, 2016**

At the request of Fruit Heights City, GeoStrata reviewed the above referenced geologic hazards evaluation report prepared by Western GeoLogic, for GSH Geotechnical (GSH), and was submitted to Fruit Heights City for the proposed Raymond Property development for review. Our review of the Western GeoLogic geologic hazards evaluation report was conducted on behalf of Fruit Heights City to assist the city in protecting public health, safety, and welfare, and to reduce risks to future property owners. The purposes of our review are to assess whether or not the report adequately addresses the geologic hazards at the site consistent with reasonable standards of practice and in accordance with Fruit Heights City's Sensitive Lands Evaluation & Development Standards (Municipal Code Title 10, Chapter 10.10A). GeoStrata participated in the field review of the site on July 15, 2015.

General

Western GeoLogic states: *"The purpose and scope of the investigation is to identify and interpret evident surficial geologic conditions at the site and identify potential risk form geologic hazards to the project. This investigation is intended to: (1) provide 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 to provide."*

Western GeoLogic also states: *"The engineering geology section of this report has been prepared following the generally accepted professional engineering geologic principles and practices in Utah; the Guidelines for Preparing Engineering Geologic reports in Utah (Utah Section of the Association of Engineering Geologists, 1986); and in accordance with Fruit Heights City Municipal Code Title 10, Chapter 10 Sensitive Lands Evaluation & Development Standards,"*

Comments and Recommendations

The Western GeoLogic geologic hazards evaluation report identifies and addresses twelve potential geologic hazards as a part of their investigation of the Raymond Property. Their evaluation of these twelve geologic hazards will be discussed below.

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1. Earthquake Ground Shaking

   Western GeoLogic reports that *"..., earthquake ground shaking is a high risk to the site. The hazard from earthquake ground shaking can be adequately mitigated by prudent design and construction."*

   The earthquake ground shaking hazard for the Raymond Property site is adequately addressed in the Western GeoLogic report. We concur that the earthquake ground shaking hazard at the site should be mitigated through proper engineering of the planned residential structures which takes into account the seismic loading parameters provided in the Western GeoLogic geologic hazards evaluation report and the GSH geotechnical report.

2. Surface Fault Rupture

   Western GeoLogic reports that *"..., the risk from surface faulting is high at the site. Based on the results of this investigation and our current understanding that surface fault rupture and deformation tend to follow past patterns, we recommend a non-buildable (setback) zone around the fault traces as shown on Figure 4 (shaded in red). Table 2 shows relevant data and calculated setbacks for each fault... No structure intended for human occupancy should be located in the setback zones 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."*

   The surface fault rupture hazard for the Raymond Property site is adequately addressed in the Western GeoLogic report. We concur with the non-buildable setback zones as shown on Figure 4 of the report. The identified faults and calculated setback zones associated with each fault as shown on Figure 4 should be clearly labeled on the subdivision plat and all design plans and drawings. The restrictions on development within the setback zones, as described in the report, should be clearly noted on the subdivision plat and all design plans and drawings.

3. Liquefaction and Lateral-Spread Ground Failure

   Western GeoLogic reports that *"..., we rate the hazard from liquefaction as low, although the risk may vary locally depending on factors such as perched groundwater."*

   The liquefaction hazard for the Raymond Property site is adequately addressed in the Western GeoLogic report. We concur that the liquefaction hazard at the site is likely low, however we recommend that GSH address the liquefaction hazard for the site in their geotechnical report and make any engineering recommendations if they feel any recommendations are needed.

4. Tectonic Deformation

   The tectonic deformation hazard for the Raymond Property site is adequately addressed in the Western GeoLogic report.

5. Seismic Seiche and Storm Surge

   The seismic seiche and storm surge hazard for the Raymond Property site is adequately addressed in the Western GeoLogic report.

6. Stream Flooding

   Western GeoLogic reports that *"..., several ephemeral drainages enter the site from higher slopes to the east, some of which continue across the property to the southwest. These drainages are likely only active following cloudburst storms and spring snowmelt and may not be consistently active, but could cause produce small flash floods. We therefore rate the hazard from stream flooding as moderate. Site hydrology and runoff should be therefore addressed in the civil engineering design and grading plan for the project.*

   The stream flooding hazard for the Raymond Property site is adequately addressed in the Western GeoLogic report. We concur that the site hydrology and runoff should be therefore addressed in the civil engineering design and grading plan for the project. It appears that the ephemeral drainages described by Western GeoLogic can be seen on the 1937 air photograph presented on Figure 3. We would recommend that these drainage channels be delineated on the site plan presented on Figure 4 in an easy to distinguish color and described in the legend of Figure 4. This will illustrate what proposed lots are currently affected by the described moderate stream flooding hazard.

   GSH should also make any prudent geotechnical recommendations regarding site drainage and surface water flows within the subject site in their geotechnical report for the site. The civil designer will also need to independently assess the site drainage and surface water flow potential across the site as part of their civil site design, however Figure 4 of the Western GeoLogic geologic hazards evaluation report and the geotechnical recommendations regarding the site drainage and surface water flow potential in the GSH geotechnical report can be utilized by the civil designer as a reference during their engineering of site grading and drainage plans.

7. Shallow Groundwater

   Western GeoLogic reports that *"..., we rate the risk from shallow groundwater as low, although the risk may vary locally depending on factors such as perched groundwater."*

   The stream flooding hazard for the Raymond Property site is adequately addressed in the Western GeoLogic report. We concur that the risk from shallow groundwater at the site is likely low, however we recommend that GSH address the shallow groundwater hazard for the site in their geotechnical report and make any engineering recommendations as needed.

8. Landslides and Slope Failures

   Western GeoLogic reports that *"..., we rate the risk from landslide at the site as high. We recommend stability of the slopes be evaluated in a geotechnical engineering evaluation prior to building based on site specific data and subsurface information included in this report. Recommendations for reducing the risk from landsliding should be provided if factors of safety are determined to be unsuitable. Care should also be taken that site grading does not destabilize slopes in this area without prior geotechnical analysis and grading plans, that proper drainage is maintained, and no non-engineered cuts are made in slope toes."*

   The landslide and slope failures hazard for the Raymond Property site is adequately addressed in

the Western GeoLogic report. Figure 4 in the Western GeoLogic geologic hazards evaluation report shows the location of three mapped landslide deposits along the steep slope located on the western property boundary of the site. The report also described a mapped and observed landslide deposit east of the subject site, which is not shown on Figure 4. Western GeoLogic considers the landslide east of the subject site to be a historical landslide. The Western GeoLogic report also describes slopes in the area of Lots 2 through 6, 46 through 49 and 18 through 27 as bordering or containing landslide-prone slopes. The three landslides mapped on the current Figure 4 are difficult to see. We recommend that these landslide deposits be shown on Figure 4 in an easy to distinguish color and described in the legend of Figure 4 and that the landslide east of the site also be shown on the figure. We also recommend that the landslide-prone slopes be shown on Figure 4. If the landslide deposits and landslide-prone slopes are too much information to clearly show on Figure 4 then a separate figure could be prepared that shows landslide and slope failure hazards in easy to distinguish colors and described in the legend of the Figure. Presenting the landslide deposits and the landslide-prone slopes on Figure 4 or a separate landslide and slope failure hazards figure will illustrate what proposed lots are currently effected by the described high landslide and slope failure hazards related to the landslide deposits and landslide-prone slopes. This figure should be utilized by GSH to aid them in locating cross sections for their slope stability modeling.

We concur that the stability of all slopes within and adjacent to the subject site should be evaluated in the GSH geotechnical report. Special consideration should be given to slopes identified by Western GeoLogic as being landslide-prone slopes as described in their report and that will be shown on Figure 4 or on a separate figure prepared for landslide and slope failure hazards as discussed above. Site specific data and subsurface information included in the Western GeoLogic geologic hazards evaluation report should be utilized to construct the geologic cross sections used for the slope stability modeling. GSH should have a licensed professional geologist prepare the geologic cross sections that are used in the slope stability modeling. Western GeoLogic should be consulted during the preparation of the geologic cross sections to assess whether the geologic cross sections accurately reflect their observations and include all of the site specific data and subsurface information included in the Western GeoLogic geologic hazards evaluation report.

9. Debris Flows

Western GeoLogic reports that *"..., Several small inactive drainages enter the eastern part of the property. The two northernmost drainages head slightly above the Bonneville shoreline and have small associated alluvial fans at their mouths (labeled 1 and 2, Figures 3 and 4). A third drainage swale is to the south that heads below the shoreline and also has a small alluvial fan (labeled 3, Figures 3 and 4). The fan size and morphology for all three drainages decreases southward. Further Southward, the remaining drainages entering the site appear to be small erosional gullies that head below the Bonneville shoreline (Figures 3 and 4). These drainages have no associated alluvial fans, and some continue to the southwest across the property..."*

The debris flow hazard for the Raymond Property site is generally adequately addressed in the Western GeoLogic report. Western GeoLogis states that their *"investigation is intended to: (1) provide 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" It is our opinion that the Western GeoLogic report has adequately identified a potential debris flow hazard associated with at least two of the drainage basins (Drainage Basins 1 and 2) east of the site that drain to the west onto the subject site and have associated alluvial fan deposits at their mouths. A third alluvial fan associated with Drainage Basin 3 was not considered by Western GeoLogic to have been active in Holocene time. Figure 4 in the Western GeoLogic geologic hazards evaluation report shows the location of the three mapped alluvial fan deposits along the base of the steep slope located on the eastern property boundary of the site. The three alluvial fans are difficult to see on the current Figure 4. We recommend that these alluvial fan deposits be shown on Figure 4 in an easy to distinguish color and described in the legend of Figure 4 and that the drainage basins east of the site also be shown on the figure.

A quantitative assessment of the potential debris flow and alluvial fan flooding hazards associated with these drainage basins was provided in the Western GeoLogic report. This assessment was provided utilizing published bulking factors from studies performed on drainage basins in the general area of the subject site and on correlations of estimated design magnitude debris flow volumes based on drainage lengths.

Western GeoLogic also reports that *"..., we rate the hazard from debris flows as moderate. Given the small calculated volumes and assuming a design flow is further attenuated by deposition in other areas, it appears likely that only a small amount of debris and muddy water would reach Lots 14 through 17. Lot 18 appears to have the highest risk since it occupies a substantial area of alluvial fan 2 and is closer to the fan mouth. Recommendations to reduce the hazard from debris flows to specific lots at the site should be provided in the civil engineering design for the development, and may include raising building pads, eliminating upslope below grade entryways, grading routing channels and berms to direct debris and water away from homes, or a combination of the above"*

GeoStrata recommends that an additional site and debris flow/alluvial fan flooding hazard-specific study be performed for the drainages basins east of the Raymond Property. It is understood that this type of assessment was out of the stated scope of the Western GeoLogic geologic hazards evaluation report. We recommend that a drainage specific debris flow hazard assessment be performed for each drainage basin east of the site that drains onto the site and has an associated alluvial fan. The site specific study should report the assessed peak flows and total debris and water volumes associated with each drainage basin. The findings of this site specific debris flow/alluvial fan flooding hazard assessment should be utilized by the civil designer to design appropriate mitigation structures.

10. <u>Rock Fall</u>
Western GeoLogic reports that *"..., we rate the hazard from rock falls as low.*

The rock fall hazard for the Raymond Property site is adequately addressed in the Western GeoLogic report.

11. Radon

    The Radon hazard for the Raymond Property site is adequately addressed in the Western GeoLogic report.

12. Swelling and Collapsible Soils

    Western GeoLogic reports that *"..., we do not anticipate that any soils susceptible to swelling or collapse will be present. However, a geotechnical engineering evaluation should be performed to address soil conditions and provide specific recommendations for site grading, subgrade preparation, and footing and foundation design."*

    We concur that the swelling and collapsible soils hazard at the site should be addressed by GSH in their geotechnical report. We recommend that GSH perform all necessary soil sampling and testing to adequately assess these hazards and provide all engineering recommendations required to meet the current standard of practice for identification and mitigation of swelling and collapsible soil hazards.

Summary of Recommendations

The following recommendations are presented in our discussion above but are presented here as a simple summary. Each numbered recommendation corresponds to the numbered geologic hazard discussed in the previous section.

1. The work completed by Western GeoLogic is considered complete with respect to earthquake ground shaking hazards. We recommend that GSH address the earthquake ground shaking hazard at the site in their geotechnical report and make any engineering recommendations as needed.
2. We recommend that Western GeoLogic be provided with a final versions of the subdivision plat and all design plans and drawings for their review to confirm that all data presented in their report regarding surface fault rupture hazards have been adequately incorporated into these documents. Western GeoLogic should confirm that the restrictions on development within the setback zones, as described in their report, have be clearly noted on the subdivision plat and all design plans and drawings. We recommend that Western GeoLogic stamp and sign the subdivision plat and all design plans and drawings if these documents are found to meet their recommendations.
3. The work completed by Western GeoLogic is considered complete with respect to liquefaction hazards. We recommend that GSH address the liquefaction hazard for the site in their geotechnical report and make any engineering recommendations if they feel any recommendations are needed.
4. The work completed by Western GeoLogic is considered complete with respect to tectonic deformation hazards.
5. The work completed by Western GeoLogic is considered complete with respect to seismic seiche and storm surge hazard.
6. We recommend that the several ephemeral drainages that Western GeoLogic identifies as entering the site from higher slopes to the east of the Raymond Property, some of which continue across the property to the southwest, be delineated on the site plan presented on Figure 4 in an easy to distinguish color and described in the legend of Figure 4.

7. The work completed by Western GeoLogic is considered complete with respect to stream flooding hazard. We recommend that GSH address the shallow groundwater hazard for the site in their geotechnical report and make any engineering recommendations as needed.
8. We recommend that the three mapped landslide deposits along the steep slope located on the western property boundary of the site and the mapped and observed landslide deposit Western GeoLogic reports east of the subject site be shown on Figure 4 in an easy to distinguish color and described in the legend of Figure 4. We also recommend that the landslide-prone slopes be shown on Figure 4. If the landslide deposits and landslide-prone slopes are too much information to clearly show on Figure 4, then a separate figure could be prepared that shows landslide and slope failure hazards in easy to distinguish colors and described in the legend of the Figure.

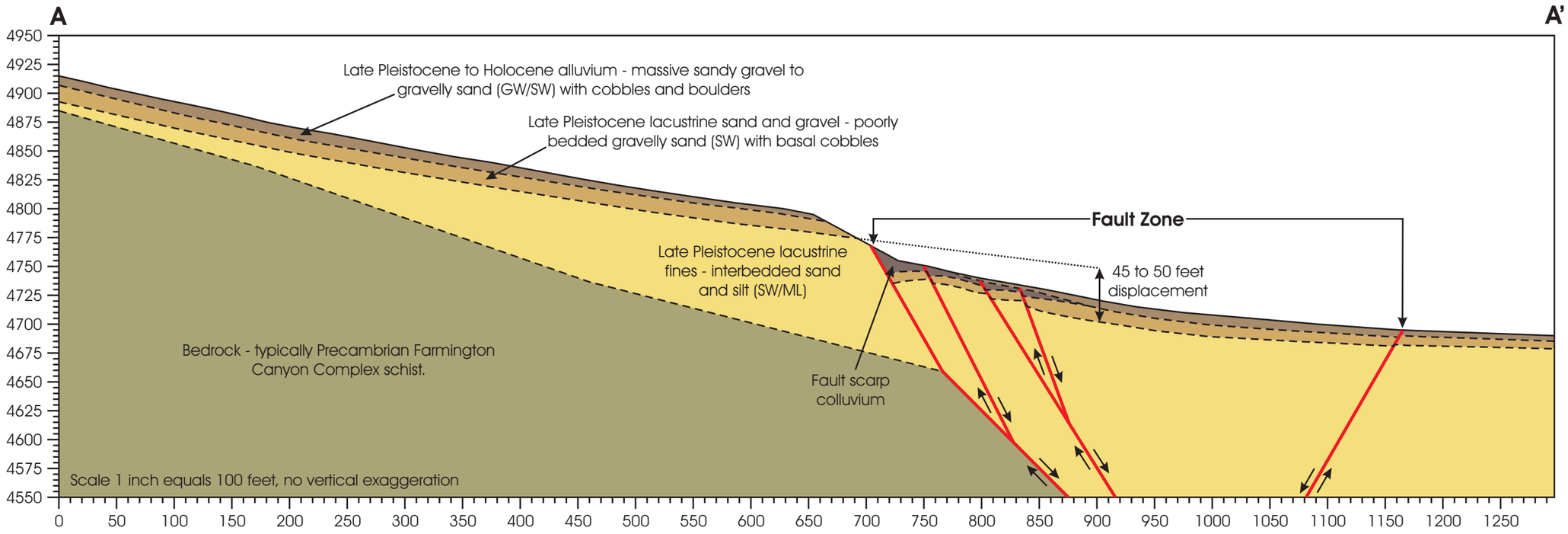
   We recommend that a licensed professional geologist from Western GeoLogic assist in the preparation of the geologic cross sections that are used in the slope stability modeling performed by GSH to assess whether the geologic cross sections accurately reflect their observations and include all of the site specific data and subsurface information included in the Western GeoLogic geologic hazards evaluation report.
9. We recommend that an additional site and debris flow/alluvial fan flooding hazard-specific study be performed for the drainages basins east of the Raymond Property. It is understood that this type of assessment was out of the stated scope of the Western GeoLogic geologic hazards evaluation report. We recommend that a drainage specific debris flow hazard assessment be performed for each drainage basin east of the site that drains onto the site and has an associated alluvial fan.
10. The work completed by Western GeoLogic is considered complete with respect to rock fall hazards.
11. The work completed by Western GeoLogic is considered complete with respect to Radon hazards. We recommend that GSH address the Radon hazard for the site in their geotechnical report and make any engineering recommendations as needed.
12. The work completed by Western GeoLogic is considered complete with respect to swelling and collapsible soils hazards. We concur that the swelling and collapsible soils hazard at the site should be addressed by GSH in their geotechnical report.

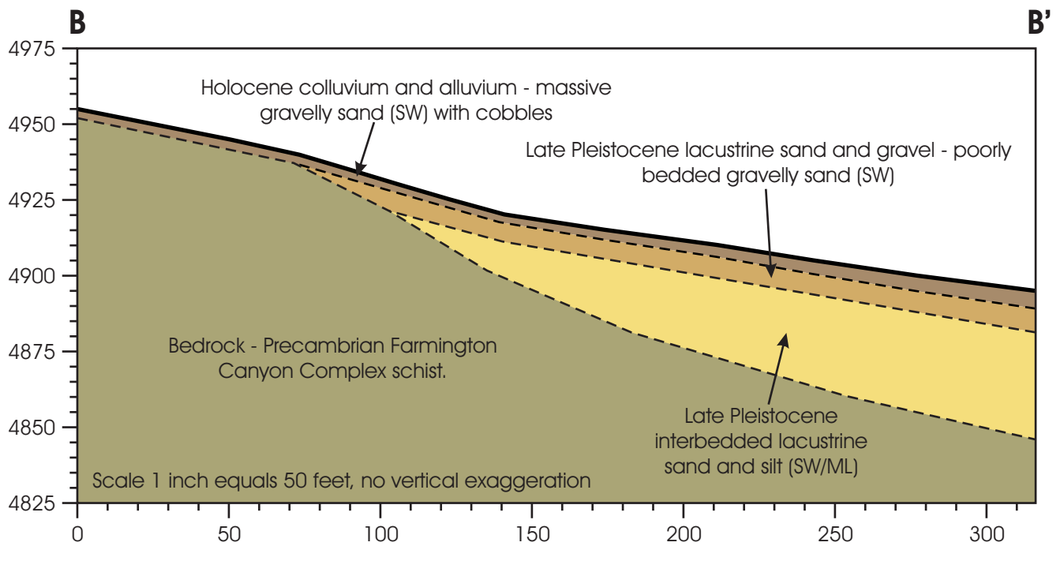
Closure

This letter is issued in response to the consultant's assessment of the above referenced site. Comments and recommendations in this review letter are based on field data presented by the Consultant. GeoStrata has not performed an independent site assessment. GeoStrata has relied on the Consultant's reports in performing its services. Consequently, it does not represent or warrant that the Consultant's report contain accurate data or proper recommendations. Recommendations and Comments presented in this review letter are provided to Fruit Heights City to aid in reducing risks from geologic hazards. GeoStrata makes no warranty; either expressed or implied and shall not be liable for any direct, special, incidental, or consequential damages with respect to claims by users of this review.

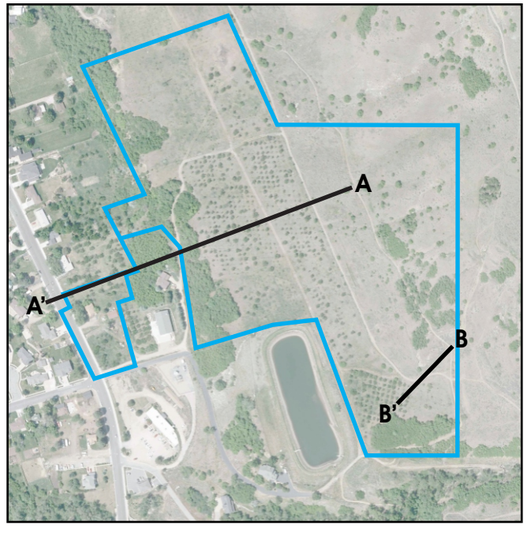
All services performed by GeoStrata for this review were provided for the exclusive use and benefit of Fruit Heights City. No other person or entity is entitled to rely on GeoStrata's services or use the

information contained in this letter without the express written consent of GeoStrata.





PROFILE LOCATIONS
(See also Figure 4)



WESTERN GEOLOGIC

CROSS SECTION

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 9

14425 South Center Point Way Bluffdale, Utah 84065
Phone (801) 501-0583 | Fax (801) 501-0584

Geologic Hazards Screening Assessment
Undeveloped Area East of 20 North Mountain Rd
Fruit Heights, Utah

GeoStrata Job No. 1058-004

June 28, 2018

Prepared for:

City of Fruit Heights
c/o Brandon Green
City Manager/Recorder
910 South Mountain Rd
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Geologic Hazards Screening Assessment
Undeveloped Area East of 20 North Mountain Rd
Fruit Heights, Utah

GeoStrata Job No. 1058-004

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June 28, 2018

TABLE OF CONTENTS

1.0 EXECUTIVE SUMMARY 1
2.0 INTRODUCTION 6
2.1 PURPOSE AND SCOPE OF WORK 6
2.2 PROJECT DESCRIPTION 6
3.0 METHODS OF STUDY 8
3.1 OFFICE INVESTIGATION 8
3.2 FIELD INVESTIGATION 8
4.0 GEOLOGIC CONDITIONS 9
4.1 GEOLOGIC SETTING 9
4.2 SITE GEOLOGY 9
5.0 GENERALIZED SITE CONDITIONS 10
5.1 SURFACE CONDITIONS 10
6.0 GEOLOGIC HAZARDS 11
6.1 EARTHQUAKE GROUND SHAKING HAZARD 12
6.2 SURFACE FAULT RUPTURE HAZARD 13
6.3 TECTONIC DEFORMATION 14
6.4 LIQUEFACTION 14
6.5 ROCKFALL AND TOPPLE 15
6.11 CANAL FLOODING 20
6.12 DAM FAILURE 20
6.13 PROBLEM SOILS 21
6.14 RADON 21
6.15 KARST AND SINK HOLES 22
7.0 CLOSURE 25
7.1 LIMITATIONS 25
8.0 REFERENCES CITED 26

APPENDICES

Appendix Plate 1 – Site Vicinity Map
Plate 2 – Topographic Map
Plate 3 – Hillshade Map
Plate 4 – Site Vicinity Geologic Map
Plate 5 – Site Vicinity 30’ X 60’ Geologic Map

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Plate 6 – Quaternary Fault Map
Plate 7 – Landslide Hazard Map
Plate 8 – Drainage Map

1.0 EXECUTIVE SUMMARY

As requested by the City of Fruit Heights, the purpose of this investigation and report is to assess the scarp along a hillside in the area located east of Mountain Road in Fruit Heights, Utah and the impacts it may have for current residential structures and any future development above and below the scarp. GeoStrata also assessed the area for the presence of all other geologic hazards that may impact any future development above and below the scarp.

The study area is located east of Mountain Road and between Green Road and Baer Canyon Road in Fruit Heights, Utah. The hillside in the study area generally slopes gently to moderately to the west toward Salt Lake Valley. It is our understanding that the general area was first developed in the 1950's. The study area east of the scarp is approximately a 27-acre parcel used for agricultural purposes. The area to the south, east and west of the scarp are occupied by established residential building lots, some of which are large enough to be subdivided.

The earthquake ground shaking hazard that would potentially impact the study area was assessed as part of our study. Given our office investigations, it is the opinion of GeoStrata that the earthquake ground shaking hazard within the study area is considered high. It is the opinion of GeoStrata that earthquake ground shaking hazard should not preclude future planned development within the study area, however, GeoStrata recommends that the earthquake ground shaking hazard within the study area should be further evaluated by a professional engineer for any proposed development.

The surface fault rupture hazard that would potentially impact the study area was assessed as part of our study. The Weber Segment of the Wasatch Fault Zone is located trending northwest through the middle of the study area. Given our field and office investigations, GeoStrata recommends that a surface fault rupture special study is conducted for any development planned within the study area.

The tectonic deformation hazard that would potentially impact the site was assessed as part of our study. The Weber Segment of the Wasatch Fault Zone is mapped within the study area. It is the opinion of GeoStrata that the tectonic deformation hazard within the study area is considered high. GeoStrata recommends that the tectonic deformation hazard within the study area should be further evaluated by a professional engineer.

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The liquefaction hazard that would potentially impact the site was assessed as part of our study. The site is located in an area currently designated as having a “Very Low” liquefaction potential. The near-surface soils are not considered to be susceptible to liquefaction. It is the opinion of GeoStrata that liquefaction hazard should not preclude development at the study area.

The rockfall hazards within the study area were assessed as part of our study. No rockfall or talus deposits are located within or immediately adjacent to the study area. Our field investigation revealed no indications that the study area has been subjected to previous rockfall. Therefore, the rockfall hazard within the study area is considered low and it is considered unlikely that rockfall will impact the proposed development. It is the opinion of GeoStrata that rockfall hazard should not preclude development at the study area.

The landslide, slump and creep hazards that would potentially impact the site were assessed as part of this study. A landslide deposit is mapped within the study area. Additional landslide deposits were observed during our office and field investigations. It is the opinion of GeoStrata that the landslide hazard within the study area is considered high. GeoStrata recommends slopes within the study area be monitored for further landslide movement and failures. Given the observed recent slope failures, GeoStrata recommends that prior to any development below or above the hillside formed by the Wasatch fault scarp, a slope stability analysis be performed by a professional engineer as part of a comprehensive site specific geotechnical investigation to assess the potential for slope failures within the study area and to determine buildable areas and setback distances from the hillside. Slope stability modeling should take landslide deposits and potential landslide deposits into consideration. The slope stability hazards within the study area is considered high. GeoStrata recommends that slope stability analysis be performed by a professional engineer for all portions of the study area being considered for development to provide recommendations for construction that will aid in reducing the risk to structures posed by potential mass movement within the study area. It should be noted that grading or development adjacent to the top or toe of the existing fault scarp area could potentially impact the stability of the hillside within the study area. The geotechnical engineer should also provide recommendations to reduce the risk of planned grading and development activities within the site having a detrimental effect on the local slope stability within the study area.

It is the opinion of GeoStrata that landslide hazards should not preclude development at the study area as long as a slope stability analysis is conducted as a part of a comprehensive site specific geotechnical investigation for the site and that the findings of the geotechnical investigation

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indicate that the planned development will not be affected by potential slope failure within the site, that the planned grading and development activities within the site do not result in a detrimental effect on the local slope stability within the study area, and that all recommendations to reduce the risks of slope stability hazards contained in the site specific geotechnical report are followed. Saturated soil conditions and residual soil strengths should be considered in the slope stability analysis conducted as a part of a comprehensive site specific geotechnical investigation for the site.

Slope stability of the study area was not assessed as part of this geological hazard assessment. The study area was observed to be gently and steeply sloping to the west toward Salt Lake Valley (Plate 2 Topographic Map). The possibility that future development within the study area could negatively affect slope stability along the Wasatch fault scarp is increased if development is planned for areas with slopes steeper than approximately 3 horizontal: 1 vertical. If improvements are planned as a part of any proposed development that include cuts and fills, it is our opinion that the post development conditions and topography should be considered in the site specific geotechnical investigation which should include a detailed slope stability assessment Recommendations presented in the geotechnical report should be followed as a part of developing grading and drainage design for any future development.

The snow avalanche hazard that would potentially impact the site was assessed as part of this study. No evidence of prior snow avalanche was observed within the study area. It is the opinion of GeoStrata that the snow avalanche hazard within the study area is considered low and it is considered unlikely that this hazard will impact the proposed development. It is the opinion of GeoStrata that snow avalanche hazard should not preclude development at the study area.

The alluvial-fan flooding hazard that would potentially impact the site was assessed as part of this study. A series of Holocene age alluvial fan deposits are mapped as underlying the entire study area. Several small drainages are located on the slope east of the study area and could be a potential for debris flow. During our review of 2017 aerial imagery and our field observations, a minor debris flow deposit was identified. GeoStrata recommends that a debris flow study be conducted prior to any development within the study area.

The shallow groundwater hazard that would potentially impact the site was not assessed and is out of the scope of this study. Seasonal fluctuations in precipitation, surface runoff from adjacent properties, or other on or offsite sources may increase moisture conditions; groundwater conditions can be expected to rise several feet seasonally depending on the time of year. Since

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shallow groundwater was not assessed as part of this study, a separate geotechnical study including subsurface exploration would be needed to assess this hazard.

The stream flooding hazard that would potentially impact the site was assessed as part of this study. The stream flooding hazard within the is considered low and it is considered unlikely that stream flooding will impact the proposed development. It is the opinion of GeoStrata that stream flooding hazard should not preclude development at the study area. Proper site grading and drainage plans should be developed for the study area as a part of the civil engineering design for the site to mitigate the potential for stream flooding to impact and damage planned structures or other planned associated infrastructure.

The canal flooding hazard that would potentially impact the site was assessed as part of this study. No canals were observed or are mapped within or adjacent to the study area. Given our field and office investigations, the canal flooding hazard within the is considered low and it is considered unlikely that canal flooding will impact the proposed development. It is the opinion of GeoStrata that canal flooding hazard should not preclude development at the.

The dam failure hazard that would potentially impact the site was assessed as part of this study. No reservoir is located within or adjacent to the study area. Given our field and office investigations, the dam failure hazard within the study area is considered low and it is considered unlikely that dam failure will impact the proposed development. It is the opinion of GeoStrata that dam failure hazard should not preclude development within the study area.

The problem soils hazard is out of the scope of this study. Based on our review of published geologic maps and our field observations, the study area is underlain by fine-grained clayey soils with sand and gravel. No laboratory testing was performed on these soils as part of this study and therefore this hazard was not assessed as part of this study. A geotechnical study should be completed for any proposed development in order to understand soil properties for use in the design of footing, foundation elements and grading.

The radon gas hazard is out of the scope of this study. Based on our review of the *Radon Hazard Potential Map of the Ogden Valley, Weber County, Utah* compiled by Solomon, 1996, the radon gas hazard within the study area is considered high. A high radon gas potential indicates that the indoor radon gas levels are likely to be greater than 4 picocuries per liter (pCi/L). Indoor testing following construction is recommended for determining radon gas levels and mitigation methods needed.

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The karst and sink holes hazards is out of the scope of this study. The karst and sink holes hazards within the study area are considered low and it is unlikely that karst and sink holes hazards will impact the proposed development.

NOTICE: The scope of services provided within this report are limited to the assessment of the subsurface conditions for the proposed development. This executive summary is not intended to replace the report of which it is part and should not be used separately from the report. The executive summary is provided solely for purposes of overview. The executive summary omits a number of details, any one of which could be crucial to the proper application of this report.

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2.0 INTRODUCTION

2.1 PURPOSE AND SCOPE OF WORK

As requested by the City of Fruit Heights, the purpose of this investigation and report is to assess the scarp along a hillside in the area located east of Mountain Road in Fruit Heights, Utah and the impacts it may have for current residential structures and any future development above and below the scarp. GeoStrata also assessed the area for the presence of all other geologic hazards that may impact any future development above and below the scarp. The geologic hazards considered for this site are presented in Table 2 of this report. The work performed for this report was performed in accordance with our proposal, dated April 2, 2018. Our scope of services included the following:

- Review of available references and maps of the area.
- Stereographic aerial photograph interpretation of aerial photographs covering the site area.
- Review of 2013-2014 0.5 meter LiDAR obtained from the State of Utah AGRC.
- Geologic reconnaissance and field mapping of the study area by an engineering geologist to observe and document pertinent surface features indicative of geologic hazards.
- Evaluation of our observations combined with existing information and preparation of this written report with conclusions and recommendations regarding observed slope failures, possible surface rupture hazards, and any other geologic hazards observed to affect the study area.

The recommendations contained in this report are subject to the limitations presented in the Limitations section of this report.

2.2 PROJECT DESCRIPTION

The study area is located east of Mountain Road and between Green Road and Baer Canyon Road in Fruit Heights, Utah. The hillside in the study area generally slopes gently to moderately to the west toward Salt Lake Valley. It is our understanding that the general area was first developed in the 1950's. The study area east of the scarp is approximately a 27-acre parcel used for agricultural purposes. The area to the south, east and west of the scarp are occupied by established residential building lots, some of which are large enough to be subdivided. The study

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area is shown on the Site Vicinity Map and Topographic Map included in the Appendix of this report (Plate 1; Plate 2).

3.0 METHODS OF STUDY

3.1 OFFICE INVESTIGATION

To prepare for the investigation, GeoStrata reviewed pertinent literature and maps listed in the references section of this report, which provided background information on the local geologic history of the area and the locations of suspected or known geologic hazards (Elliot and Harty, 2010; Black and others, 2003; UGS, 2016, Coogan and King, 2016; Solomon, 2007). A stereographic aerial photograph interpretation was performed for the study area using two sets of stereo aerial photographs obtained from the UGS as shown in Table 1.

Source	Photo Number	Date	Scale
AMS	AMS_121-16-2963_B	August 10, 1953	64,000
AMS	AMS_121-9-1555	June 26, 1953	64,000

Table 1: Aerial Stereosets.

GeoStrata also conducted a review of hillshades derived from 2013-2014 0.5 meter LiDAR obtained from the State of Utah AGRC to assess the study area for visible alluvial fan deposits, scarps associated with landslide geomorphology and lineations related to stream flooding hazards or surface fault rupture related geomorphology. The LiDAR elevation data was used to create hillshade imagery that could be reviewed for assessment of geomorphic features related to geologic hazards (Plate 3 Hillshade Map).

3.2 FIELD INVESTIGATION

An engineering geologist investigated the geologic conditions within the general site area. A field geologic reconnaissance was conducted to observe existing geologic conditions and to assess existing geomorphology for surficial evidence of geologic hazards. During our fieldwork we conducted site observations to assess geologic hazards that might impact the study area. We used our field observations to confirm the observations made during our office research and to observe any evidence of geologic hazards that were not evident in our office research, but which could be observed in the field.

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4.0 GEOLOGIC CONDITIONS

4.1 GEOLOGIC SETTING

The site is located within the Salt Lake Valley along the foothills of the Wasatch Mountain Range in Fruit Heights, Utah at approximately 4,715 feet above sea level. The Salt Lake basin is a deep, sediment-filled structural basin of Cenozoic age flanked by the Wasatch Range and Wellsville Mountains to the east and the Promontory Mountains, the Spring Hills, and the West Hills to the west (Hintze, 1980). The southern portion of the Salt Lake Basin is bordered on the west by the east shore of the Great Salt Lake. The Wasatch Range is the easternmost expression of pronounced Basin and Range extension in north-central Utah (Stokes, 1986).

The near-surface geology of the Salt Lake Valley is dominated by lake sediments which were deposited within the last 30,000 years during the high stand of the Lake Bonneville Cycle (Scott and others, 1983; Hintze, 1993; Leggette and Taylor, 1937; King and others, 2008). As Lake Bonneville receded, streams began to incise large deltas that had formed at the mouths of major canyons along the Wasatch Range. The eroded material was then deposited in shallow lakes and marshes in the basin and in a series of recessional deltas and alluvial fans. Sediments toward the center of the valley are predominately deep-water deposits of clay, silt and fine sand whereas sediments closer to the mountain fronts are shallow-water deposits of coarse sand and gravel. However, these deep-water deposits are in places covered by a thin post-Bonneville alluvial cover.

4.2 SITE GEOLOGY

The surficial deposits within the study area are shown on Plate 4 Site Vicinity Geologic Map and Plate 5 Site Vicinity 30x60 Geologic Map. The geology within the study area as shown on Plate 4 is mapped as a series of alluvial fan deposits as well as a small landslide deposit located in the center of the study area (Sorensen and Crittenden, 1979). These deposits are mapped as overlying lacustrine sand and silt related to the Provo phase of the Bonneville lake cycle. The geology within the study area as shown on Plate 5 is mapped as a series of alluvial fan deposits overlying Lake Bonneville sand deposits (Coogan and King, 2016).

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5.0 GENERALIZED SITE CONDITIONS

5.1 SURFACE CONDITIONS

The study area is a steeply sloping hillside located north of Bair Canyon in Fruit Heights, Utah. Surficial deposits on the study area were observed to consist of bedded fine grained lake Bonneville sand deposits. The slope remains in a relatively natural state and is densely vegetated with large brush and trees. The top of the slope is currently an orchard field. The area below the slope is occupied by developed single family residences; most homes are located on the western portion of the lot away from the slope. Few homes are located less than one hundred feet from the slope. A native hillside containing several small drainages is located east of the study area. The area west of the study area is occupied by an established residential neighborhood.

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6.0 GEOLOGIC HAZARDS

Geologic hazards can be defined as naturally occurring geologic conditions or processes that could present a danger to human life and property. These hazards must be considered before development of the site. There are several hazards that if present at the site should be considered in the design of habitable structures and other critical infrastructure. The hazards considered for this site are presented on Table 2 and discussed in the following sections of this report.

Hazard	Hazard Rating					Further Study Recommended
	Not Applicable	Not Assessed	Low	Moderate	High	
Ground Shaking					X	E
Surface Fault Rupture					X	G
Tectonic Deformation					X	E
Liquefaction			X			
Rock Fall and Topple			X			
Landslide					X	
Slump					X	
Creep					X	
Avalanche			X			
Debris Flow				X		
Hyperconcentrated Flow			X			
Stream Flow			X			
Shallow Groundwater		X				E
Stream Flooding			X			
Canal Flooding	X					
Dam Failure	X					
Problem Soils		X				E
Radon		X				
Karst and Sink Hole		X				

Table 2: Summary of Geologic Hazards.

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Table 2 shows the summary of the geologic hazards assessed and not assessed as a part of this study for the study area. The hazard rating as shown on Table 2 is intended to assess the probability that the hazard could have an impact on the site and not the severity of the hazard. A hazard rating of "Not Assessed" are hazards this report does not consider and no inference is made as to the presence or absence of the hazard at the site. A hazard rating of "Low" indicates that no evidence was found to indicate that the hazard is present and has a low probability of impacting the site, hazard not known or suspect to be present. A hazard rating of "Moderate" indicates that the hazard has a moderate probability of impacting the site, but the evidence is equivocal, based only on theoretical studies, or was not observed and further study is necessary as noted. A hazard rating of "High" indicates that that evidence is strong and suggests that there is a high probability of impacting the site and mitigation measures should be taken. If a hazard is assessed to potentially impact the site then further studies may be recommended. The following are the recommended studies and the letter designation associated with those studies: "E" – geotechnical/engineering, "H" – hydrologic, "A" – avalanche, "G" – additional detailed geologic hazard study out of the scope of this study.

6.1 EARTHQUAKE GROUND SHAKING HAZARD

During the event of an earthquake, seismic waves radiate outward from the initial point of rupture and dissipate with distance. The ground shakes as the seismic waves displace the ground both vertically and horizontally. Ground shaking can cause significant damage to and potentially collapse structures and can also trigger landslides, avalanches and liquefaction. The type of soil a seismic wave travels through can amplify or dampen the effects of ground shaking.

Seismic hazard maps depicting probabilistic ground motions and spectral response have been developed for the United States by the U.S. Geological Survey as part of NEHRP/NSHMP (Frankel et al, 1996). These maps have been incorporated into both *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA, 1997) and the *International Building Code* (IBC) (International Code Council, 2015). Spectral responses for the Maximum Considered Earthquake (MCE_R) are shown in the table below. These values generally correspond to a two percent probability of exceedance in 50 years (2PE50) for a "firm rock" site. To account for site effects, site coefficients which vary with the magnitude of spectral acceleration are used. Based on our field and office investigations, it is our opinion that this location is best described as a Site Class D which represents a "Stiff Soil" profile. The spectral accelerations are shown in the table below. The spectral accelerations are calculated based on the site's approximate latitude and longitude of 41.035443° and -111.901521° respectively and the

United States Geological Survey U.S. Seismic Design Maps web based application. Based on the IBC, the site coefficients are F_a=1.0 and F_v= 1.5. From this procedure the peak ground acceleration (PGA) is estimated to be 0.57g.

Site Location: **Latitude = 41.035443 N** **Longitude = -111.901521 W**	**Site Class D Site Coefficients:** **F_a = 1.0** **F_v = 1.5**
Spectral Period (sec)	**Response Spectrum Spectral Acceleration (g)**
0.2	**S_{MS}**=(F_a*S_s=1.0*1.428) = 1.43
1.0	**S_{M1}**=(F_v*S_1=1.5*0.593) = 0.89
[a] IBC 1613.3.4 recommends scaling the MCE_R values by 2/3 to obtain the design spectral response acceleration values; values reported in the table above have not been reduced.	

Table 3: MCE_R Seismic Response Spectrum Spectral Acceleration Values for IBC Site Class D[a].

Based on the above information, it is the opinion of GeoStrata that the earthquake ground shaking hazard within the study area is considered high. It is the opinion of GeoStrata that earthquake ground shaking hazard should not preclude future planned development within the study area, however, GeoStrata recommends that the earthquake ground shaking hazard within the study area should be further evaluated by a professional engineer for any proposed development.

6.2 SURFACE FAULT RUPTURE HAZARD

Movement along faults within the crustal rocks beneath the ground surface generates earthquakes. During large magnitude earthquakes (Richter magnitude 6.5 or greater) along the normal faults in the intermountain region, fault ruptures can propagate to the ground surface resulting in a surface fault rupture (Smith and Arabasz, 1991). The fault scarp formed during a surface fault rupture event along a normal fault is generally nearly vertical. A surface rupture fault may be comprised of a larger single surface rupture or several smaller surface ruptures across a fault zone. For all structures designed for human occupancy, a surface rupturing fault is considered active if it has experienced movement in approximately the past 10,000 years (Christenson and others, 2003).

Based on review of published geologic maps, our stereographic aerial photograph interpretation, our review of the hillshades derived from 2013-2014 0.5 meter LiDAR and our field observations, the Weber Segment of the Wasatch Fault Zone is located trending northwest through the middle of the study area (Plate 4 Site Vicinity Geologic Map; Plate 5 Site Vicinity 30x60 Geologic Map; Plate 6 UGS Quaternary Fault Map). Given our field and office investigations, GeoStrata recommends that a surface fault rupture special study is conducted for any development planned within the study area.

6.3 TECTONIC DEFORMATION

Subsidence is a hazard associated with warping, lowering and tilting of a valley floor accompanying surface ruptures on normal faults (Robinson, 1993). Inundation along the shores of lakes and reservoirs and the rise of groundwater levels are the main hazards associated with subsidence. Structures that require gentle gradients or horizontal floors such as waste water treatment plants and sewer lines may be adversely affected by tectonic subsidence. Because subsidence may occur over very large areas, it is not generally practical to avoid the use of potentially affected land except in narrow areas of hazard due to lakeshore inundation (Keaton, 1987; Robison, 1993). According to Gary Christenson (UGS, personal communication 2001), tectonic subsidence is not typically assessed for subdivision development unless the development is located within an area of potential lake flooding.

Based on review of published geologic maps, our stereographic aerial photograph interpretation, our review of the hillshades derived from 2013-2014 0.5 meter LiDAR and our field observations, the Weber Segment of the Wasatch Fault Zone is mapped within the study area. It is the opinion of GeoStrata that the tectonic deformation hazard within the study area is considered high. GeoStrata recommends that the tectonic deformation hazard within the study area should be further evaluated by a professional engineer.

6.4 LIQUEFACTION

Certain areas within the intermountain region possess a potential for liquefaction during seismic events. Liquefaction is a phenomenon whereby loose, saturated, granular soil deposits lose a significant portion of their shear strength due to excess pore water pressure buildup resulting from dynamic loading, such as that caused by an earthquake. Among other effects, liquefaction can result in densification of such deposits causing settlements of overlying layers after an earthquake as excess pore water pressures are dissipated. The primary factors affecting

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liquefaction potential of a soil deposit are: (1) level and duration of seismic ground motions; (2) soil type and consistency; and (3) depth to groundwater.

Based on our review of the *Liquefaction Special Study Areas, Wasatch Front and Nearby Areas, Utah* compiled by Christenson and others, 2008, the site is located in an area currently designated as having a "Very Low" liquefaction potential. "Very Low" liquefaction potential indicates that there is less than a 5 percent probability of having an earthquake within a 100-year period that will be strong enough to cause liquefaction. The surface soils we observed during our field investigation are not considered to be susceptible to liquefaction. A liquefaction analysis was beyond the scope of this geologic hazards assessment and is typically not accessed for single family residential structures; however, if any future home owner or developer wishes to have a greater understanding of the liquefaction potential of the soils at greater depths, a liquefaction analysis should be completed at the site. It is the opinion of GeoStrata that liquefaction hazard should not preclude development within the study area.

6.5 ROCKFALL AND TOPPLE

Rockfalls are the fastest moving mass movement that predominantly occurs in mountains where a rock source exists along steep slopes and cliffs greater than 35 degrees. Rockfalls are a result of a loss of support from beneath the rock mass that can be caused by freeze/thaw action, rainfall, weathering and erosion, and/or strong ground shaking resulting from seismic activity. Rockfalls result in the collection of rock fall material, referred to as talus, at the base of the slope. The presence of talus indicates that a rockfall hazard has occurred and may still be present at the site.

Based on review of published geologic maps, our stereographic aerial photograph interpretation and our field observations, no rockfall or talus deposits are located within or immediately adjacent to the study area. Our field investigation revealed no indications that the study area has been subjected to previous rockfall. Therefore, the rockfall hazard within the study area is considered low and it is considered unlikely that rockfall will impact future proposed development. It is the opinion of GeoStrata that rock fall hazard should not preclude development within the study area.

6.6 LANDSLIDE, SLUMP, CREEP

There are several types of landslides that should be considered when evaluating geologic hazards at a site with moderately to steeply sloping terrain. These include shallow debris slides, deep-

seated earth or rock slumps and earth flows. Landslides, slumps, creep and other mass movements can develop on moderate to steep slopes where the slope has been altered or disturbed. Movement can occur at the top of a slope that has been loaded by fill placement, at the base of a slope that has been undercut, or where local groundwater rises resulting in increased pore pressures within the slope. Slopes that exhibit prior failures and large landslide deposits are particularly susceptible to instability and reactivation.

Based on review of published geologic maps, hillshades derived from 2013-2014 0.5 meter LiDAR, a landslide deposit is mapped within the study area (Plate 4 Site Vicinity Geologic Map; Plate 6 Landslide Hazard Map). Based on our review of our stereographic aerial photograph interpretation, our review of the hillshades derived from 2011 1 meter LiDAR and our field observations, additional landslide deposits are located south of the mapped landslide deposit as indicated on Plate 6 Landslide Hazard Map. These areas were observed to consist of hummocky topography and scarps of varying heights and ages. The landslide deposit observed and mapped by GeoStrata approximately 200 feet south of the mapped landslide deposit was observed to be the youngest and most prominent landslide deposit with a scarp approximately 9 feet tall and 60 feet long. The soil exposed along the landslide scarp was observed to consist of bedded fine grained lacustrine sand deposits related to Lake Bonneville. Vegetation indicative of moderately saturated soils was observed toward the base of this landslide deposit. Based on our review of historical aerial imagery, it is our opinion that the landslide deposits mapped by GeoStrata are recent failures; likely within the last 20 years. During our initial site visit in March of 2018, a scarp approximately 2 to 3 feet tall and 250 feet long was observed trending parallel to the hillside as indicated on Plate 7 Landslide Hazard Map.

Based on our field observations, our review of hillshades derived from 2013-2014 0.5 meter LiDAR, our stereographic aerial photograph interpretation and our review of published geologic maps, it is the opinion of GeoStrata that the landslide hazard within the study area is considered high. GeoStrata recommends slopes within the study area be monitored for further landslide movement and failures. Given the observed recent slope failures, GeoStrata recommends that prior to any development below or above the hillside formed by the Wasatch fault scarp, a slope stability analysis be performed by a professional engineer as part of a comprehensive site specific geotechnical investigation to assess the potential for slope failures within the study area and to determine buildable areas and setback distances from the hillside. Slope stability modeling should take landslide deposits and potential landslide deposits into consideration. The slope stability hazards within the study area is considered high. GeoStrata recommends that slope stability analysis be performed by a professional engineer for all portions of the study area being

considered for development to provide recommendations for construction that will aid in reducing the risk to structures posed by potential mass movement within the study area. It should be noted that grading or development adjacent to the top or toe of the existing fault scarp area could potentially impact the stability of the hillside within the study area. The geotechnical engineer should also provide recommendations to reduce the risk of planned grading and development activities within the site having a detrimental effect on the local slope stability within the study area.

It is the opinion of GeoStrata that landslide hazards should not preclude development at the study area as long as a slope stability analysis is conducted as a part of a comprehensive site specific geotechnical investigation for the site and that the findings of the geotechnical investigation indicate that the planned development will not be affected by potential slope failure within the site, that the planned grading and development activities within the site do not result in a detrimental effect on the local slope stability within the study area, and that all recommendations to reduce the risks of slope stability hazards contained in the site specific geotechnical report are followed. Saturated soil conditions and residual soil strengths should be considered in the slope stability analysis conducted as a part of a comprehensive site specific geotechnical investigation for the site.

Slope stability of the study area was not assessed as part of this geological hazard assessment. The study area was observed to be gently and steeply sloping to the west toward Salt Lake Valley (Plate 2 Topographic Map). The possibility that future development within the study area could negatively affect slope stability along the Wasatch fault scarp is increased if development is planned for areas with slopes steeper than approximately 3 horizontal: 1 vertical. If improvements are planned as a part of any proposed development that include cuts and fills, it is our opinion that the post development conditions and topography should be considered in the site specific geotechnical investigation which should include a detailed slope stability assessment Recommendations presented in the geotechnical report should be followed as a part of developing grading and drainage design for any future development.

6.7 AVALANCHE

An avalanche is a rapid flow of snow down a hill or mountainside. A snow avalanche can be a hazard in high alpine settings with slopes generally between 35 degrees and 45 degrees that accumulate appreciable amounts of snow. There are three types of avalanches: slough, dry slab and wet slab. Sloughs typically occur right after a heavy snowfall event. This type of slide occurs

from a single point and accumulates snow as it moves downslope. Dry slabs are the most common type of avalanche and are the result of a fracture that occurs along a weak layer within the snowpack. Dry slabs can travel upwards of 80 mph removing trees and structures in its path. Wet slabs are triggered when percolating water dissolves bonds and decreases the strength of the weak snow layer. This type of slab can travel up to 20 mph. Several factors that influence a snow avalanche include weather, temperature, slope steepness, slope orientation, wind direction and wind loading, terrain, vegetation, and snowpack conditions. Snow avalanche hazard could affect access and snow removal on roads as well as the safety of habitable structures and critical facilities.

Based on review of our stereographic aerial photograph interpretation and our field observations, no evidence of prior snow avalanche was observed within the study area. It is the opinion of GeoStrata that snow avalanche hazards should not preclude development within the study area.

6.8 ALLUVIAL FAN FLOODING

Alluvial fan flooding is a potential hazard that may exist in areas containing Holocene alluvial fan deposits. This type of flooding typically occurs as a stream flows, hyperconcentrated flows and debris flows consisting of a mixture of water, soil, organic material, and rock debris with variations in sediment-water concentrations transported by fast-moving water flows. Stream flows contains approximately less than 20% sediment by volume and involves sediment transport by entrained and suspended sediment load (Bowman and Lund, 2016). Unconfined stream flows are referred to as sheetfloods which are spread over and occur in the distal areas of the alluvial fan. Hyperconcentrated flows are alluvial fan flows with 20 to 60% sediment by volume whereas debris flows contain greater than 60% sediment by volume.

Alluvial fan flooding can be a hazard on or below alluvial fans or in stream channels above alluvial fans. Precipitation (rainfall and snowmelt) is generally viewed as an alluvial fan flood “trigger”, but this represents only one of the many factors that contribute to alluvial fan flooding hazard. Vegetation, root depth, soil gradation, antecedent moisture conditions and long term climatic cycles all contribute to the generation of debris and initiation of alluvial fan flooding. Events of relatively short duration, such as a fire, can significantly alter a basin’s absorption of storm water and snowmelt runoff and natural resistance to sediment mobilization for an extended period of time. These factors are difficult to quantify or predict and vary not only between different watersheds, but also within each sub-area of a drainage basin. In general, there are two methods by which alluvial fan flooding can be mobilized: 1) when shallow landslides from

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channel side-slopes are conveyed in existing channels when mixed with water and 2) channel scour where debris is initially mobilized by moving water in a channel and then the mobilized debris continues to assemble and transport downstream sediments.

Based on review of published geologic maps, review of stereographic aerial photographs and hillshades derived from 2013-2014 0.5 meter LiDAR, a series of Holocene age alluvial fan deposits are mapped as underlying the entire study area (Plate 4 Site Vicinity Geologic Map; Plate 5 Site Vicinity 30' X 60' Geologic Map). Several small drainages are located on the slope east of the study area and could be a potential for debris flow. During our review of 2017 aerial imagery and our field observations, a minor debris flow deposit was identified as indicated on Plate 9 Debris Flow. GeoStrata recommends that a debris flow study be conducted prior to any development within the study area.

6.9 SHALLOW GROUNDWATER

Shallow groundwater flooding is a hazard that can cause the flooding of excavated areas where the depth of excavation exceeds the depth of the local water table. Shallow groundwater flooding should be considered when designing habitable structures that require excavation that may exceed the depth to the shallow groundwater.

Shallow groundwater assessment is out of the scope of this study. Seasonal fluctuations in precipitation, surface runoff from adjacent properties, or other on or offsite sources may increase moisture conditions; groundwater conditions can be expected to rise several feet seasonally depending on the time of year. Since shallow groundwater was not assessed as part of this study, a separate geotechnical study including subsurface exploration would be needed to assess this hazard for any proposed development.

6.10 STREAM FLOODING HAZARD

Stream flooding can be caused by precipitation, snowmelt or a combination of both. Throughout most of Utah floods are most common in spring during the snowmelt. High flows in drainages can last for a few hours to several weeks. Factors that affect the potential for flooding at a site include surface water drainage patterns and hydrology, site grading and drainage design, and seasonal runoff.

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Based on review of published topographic maps, our stereographic aerial photograph interpretation, our review of the hillshades derived from 2006 2 meter LiDAR and our field observations, no drainages are located within or adjacent to the study area (Plate 8 Drainage Map). The active channels were observed to be approximately 3 to 5 feet wide with gently to moderately dipping banks. Flowing water was not observed within any of the drainages at the time of our site visit. Streambeds consisted of large well-rounded cobbles and boulders embedded in fine grained soils.

Given our field and office investigations, the stream flooding hazard within the study area is considered low and it is considered unlikely that stream flooding will impact future proposed development. It is the opinion of GeoStrata that stream flooding hazard should not preclude development within the study area. Proper site grading and drainage plans should be developed for any proposed development as a part of the civil engineering design proposed development to mitigate the potential for stream flooding to impact and damage planned structures or other planned associated infrastructure.

6.11 CANAL FLOODING

High runoff in a short period of time can lead to canal water breaching their banks and flooding the surrounding area. Failure of the canal embankments or a blockage in the canal could also lead to flooding surrounding the canal.

Based on review of published topographic maps, our stereographic aerial photograph interpretation, our review of the hillshades derived from 2011 1 meter LiDAR and our field observations, no canals were observed or are mapped within or adjacent to the study area. Given our field and office investigations, the canal flooding hazard within the study area is considered low and it is considered unlikely that canal flooding will impact future proposed development. It is the opinion of GeoStrata that canal flooding hazard should not preclude development within the study area.

6.12 DAM FAILURE

Dams are structures that store water and diverge and impound water upstream. Most dams have a spillway where water flow from the reservoir is controlled and hydroelectric power is produced. Failure in dams can occur from a collapse or a breach in the structure most commonly due to extended periods of high runoff.

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Based on our review of the Kaysville topographic quadrangle and our field investigation, no reservoir is located within or adjacent to the study area (Plate 1 Site Vicinity Map; Plate 2 Topographic Map). Given our field and office investigations, the dam failure hazard within the study area is considered low and it is considered unlikely that dam failure will impact the proposed development. It is the opinion of GeoStrata that dam failure hazard should not preclude development within the study area.

6.13 PROBLEM SOILS

Problem soils include collapsible soils and expansive soils. Collapsible soils are low density and typically dry soils that decrease in volume when exposed to water. This type of problem soil typically occurs in alluvial fan flooding deposits, dry loess or eolian deposits or unconsolidated colluvium deposits (Owens and Rollins, 1990). Expansive soils are soils that undergo an increase in volume upon wetting and typically include fine grained soils such as clay.

The problem soils hazard is out of the scope of this study. Based on our review of published geologic maps and our field observations, the study area is underlain by fine-grained lacustrine sand. No laboratory testing was performed on these soils as part of this study and therefore this hazard was not assessed as part of this study. A geotechnical study should be completed for any proposed development in order to understand soil properties for use in the design of footing, foundation elements and grading.

6.14 RADON

Radon is a naturally occurring odorless, tasteless and colorless gas that is released during the breakdown of uranium in well drained permeable soils and uranium rich rocks which include granite, metamorphic rocks, black shales, and some volcanic rocks (Sprinkel and Solomon, 1990). Radon gas moves freely in the air and can also dissolve in water which can potentially migrate through cracks and open spaces in rock, soils, and foundations as well as utility pipes.

The radon gas hazard is out of the scope of this study. Based on our review of the *Radon Hazard Potential Map of the Ogden Valley, Weber County, Utah* compiled by Solomon, 1996, the radon gas hazard within the study area is considered high. A high radon gas potential indicates that the indoor radon gas levels are likely to be greater than 4 picocuries per liter (pCi/L). Indoor testing

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following construction is recommended for determining radon gas levels and mitigation methods needed.

6.15 KARST AND SINK HOLES

A karst is a type of underground drainage terrain that is the result of dissolution of soluble bedrock such as limestone, carbonate rock, salt beds or other types of rocks that are easily dissolved by groundwater circulating through them. The most common type of hazard that forms within a karst terrain is subsidence or collapse of soils, these are referred to as sink holes. Sink holes can be a few feet to hundreds of acres wide and 1 to 100 feet deep and can form slowly or collapse suddenly.

Based on our review of published geologic maps, the karst and sink holes hazards within the study area are considered low and it is unlikely that karst and sink holes hazards will impact the proposed development. It is the opinion of GeoStrata that karst and sink hole hazards should not preclude development within the study area.

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7.0 GEOLOGIC HAZARDS SUMMARY AND CONCLUSIONS

It is the opinion of GeoStrata that the geologic hazards that we assessed in this study that could impact the study area or that have not been assessed as a part of this study, but which could impact the study area include: earthquake ground shaking, surface fault rupture, tectonic deformation, landslide, slump and creep, shallow groundwater, problem soils and radon gas. Below is a summary of each geologic hazard and GeoStrata's recommendation for mitigation:

- It is the opinion of GeoStrata that the earthquake ground shaking hazard within the study area is considered high. It is also the opinion of GeoStrata that earthquake ground shaking hazard should not preclude future planned development within the study area, however, GeoStrata recommends that the earthquake ground shaking hazard within the study area should be further evaluated by a professional engineer for any proposed development.

- The Weber Segment of the Wasatch Fault Zone is located trending northwest through the middle of the study area. Given our field and office investigations, GeoStrata recommends that a surface fault rupture special study is conducted for any development planned within the study area.

- It is the opinion of GeoStrata that the landslide hazard within the study area is considered high. Given the observed recent slope failures, GeoStrata recommends that prior to any development below or above the hillside formed by the Wasatch fault scarp, a slope stability analysis be performed by a professional engineer as part of a comprehensive site specific geotechnical investigation to assess the potential for slope failures within the study area and to determine buildable areas and setback distances from the hillside. Slope stability modeling should take landslide deposits and potential landslide deposits into consideration. The slope stability hazards within the study area is considered high. GeoStrata recommends that slope stability analysis be performed by a professional engineer for all portions of the study area being considered for development to provide recommendations for construction that will aid in reducing the risk to structures posed by potential mass movement within the study area. It should be noted that grading or development adjacent to the top or toe of the existing fault scarp area could potentially impact the stability of the hillside within the study area. The geotechnical engineer should also provide recommendations to reduce the risk of planned grading and development activities within the site having a detrimental effect on the local slope stability within the study area.

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Slope stability of the study area was not assessed as part of this geological hazard assessment. The study area was observed to be gently and steeply sloping to the west toward Salt Lake Valley (Plate 2 Topographic Map). The possibility that future development within the study area could negatively affect slope stability along the Wasatch fault scarp is increased if development is planned for areas with slopes steeper than approximately 3 horizontal: 1 vertical. If improvements are planned as a part of any proposed development that include cuts and fills, it is our opinion that the post development conditions and topography should be considered in the site specific geotechnical investigation which should include a detailed slope stability assessment Recommendations presented in the geotechnical report should be followed as a part of developing grading and drainage design for any future development.

- Shallow groundwater hazard within the study area was not assessed as part of this study. Seasonal fluctuations in precipitation, surface runoff from adjacent properties, or other on or offsite sources may increase moisture conditions; groundwater conditions can be expected to rise several feet seasonally depending on the time of year. Since shallow groundwater was not assessed as part of this study, a separate geotechnical study including subsurface exploration would be needed to assess this hazard.

- Problem soils hazard within the study area was not assessed as part of this study. The study area is underlain by fine-grained clayey soils with sand and gravel. No laboratory testing was performed on these soils as part of this study and therefore this hazard was not assessed as part of this study. Since problem soils were not assessed as part of this study, a separate geotechnical study including subsurface exploration would be needed to assess this hazard.

- Radon gas hazard within the study area is considered high. Indoor testing following construction is recommended for determining radon gas levels and mitigation methods if needed.

It is the opinion of GeoStrata that these hazards should not preclude the development of the study area, assuming that these recommendations given above will be followed.

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7.0 CLOSURE

7.1 LIMITATIONS

The conclusions and recommendations contained in this report, which include professional opinions and judgments, are based on the information available to us at the time of our evaluation, the results of our field observations and our understanding of the proposed site development. If any conditions are encountered at this site that are different from those described in this report, our firm should be immediately notified so that we may make any necessary revisions to recommendations contained in this report. In addition, if the scope of the proposed development changes from that described in this report, our firm should also be notified.

All services were completed in accordance with the current standard of care and generally accepted standard of practice at the time and in the place our services were completed. No other warranty, expressed or implied, is made. Development of property in the immediate vicinity of geologic hazards involves a certain level of inherent risk. It is impossible to predict where geologic hazards will occur. New geologic hazards may develop and existing geologic hazards may expand beyond their current limits.

All services were performed for the exclusive use and benefit of the above addressee. No other person is entitled to rely on GeoStrata’s services or use the information contained in this letter without the express written consent of GeoStrata. We are not responsible for the technical interpretations by others of the information described or documented in this report. The use of information contained in this report for bidding purposes should be done at the Contractor's option and risk.

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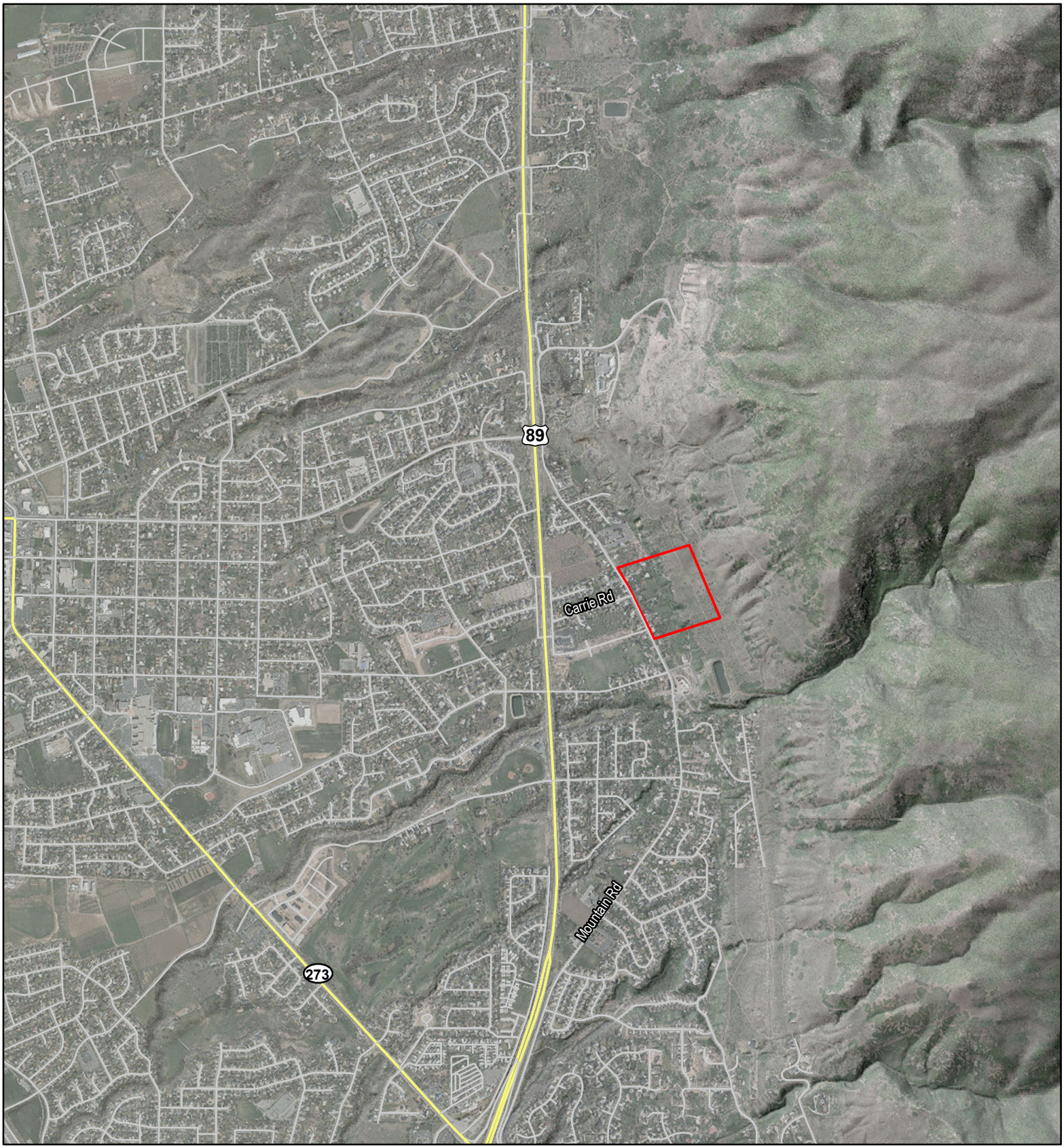
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Appendix



Legend

Approximate Site Boundary

0 625 1,250 2,500 3,750 5,000 Feet

1 inch = 2,000 feet

Basemap:
2012 12.5cm HRO aerial imagery and hillshades derived from 2013-2014 0.5m LiDAR provided by the State of Utah AGRC.

N

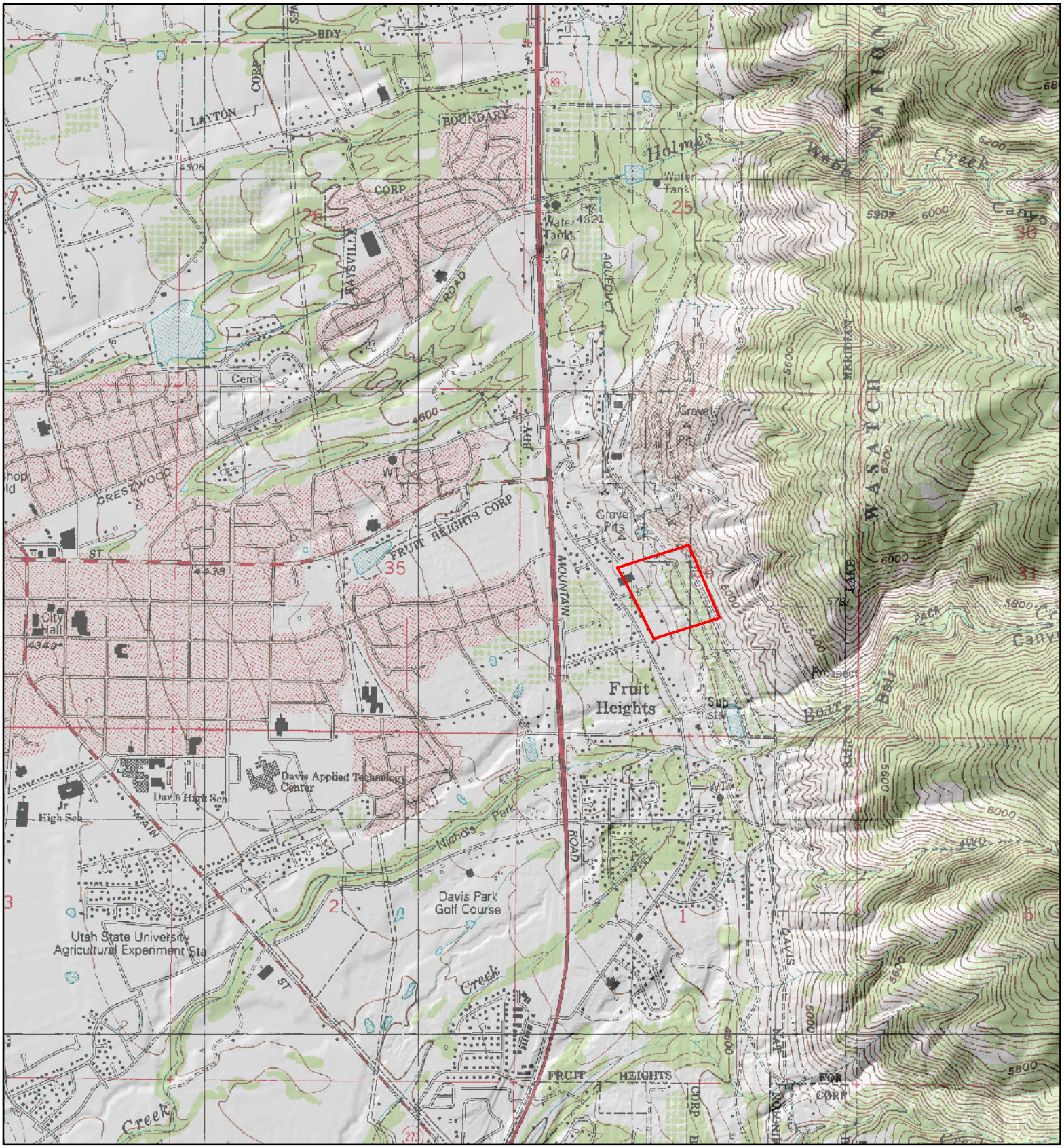
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City of Fruit Heights
S. Mountain View Rd and Carrie Dr
Fruit Heights, Utah
Project Number: 1058-004

Site Vicinity Map

Plate 1



Legend

Approximate Site Boundary

0 625 1,250 2,500 3,750 5,000 Feet

1 inch = 2,000 feet

Basemap:
Kaysville Quadrangle, Utah, 7.5-Minute Series (Topographic), USGS 1998 and hillshades derived from 2013-2014 0.5m LiDAR provided by the State of Utah AGRC.

N

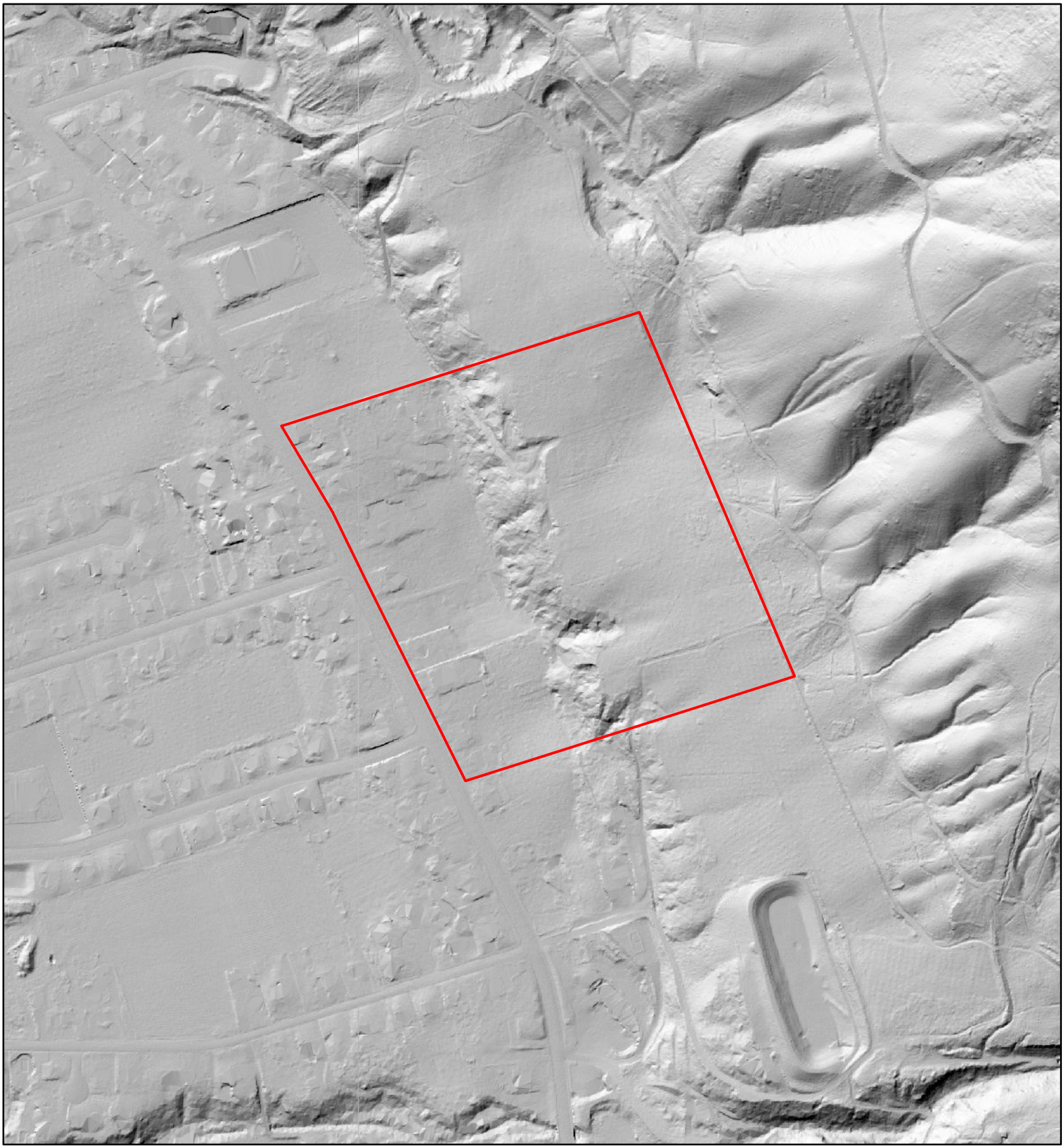
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Fruit Heights, Utah
Project Number: 1058-004

Topographic Map

Plate 2



Legend

Approximate Site Boundary

0 125 250 500 750 1,000 Feet

1 inch = 400 feet

Basemap:
Hillshades derived from 2013-2014 0.5m
LiDAR provided by the State of Utah AGRC.

N

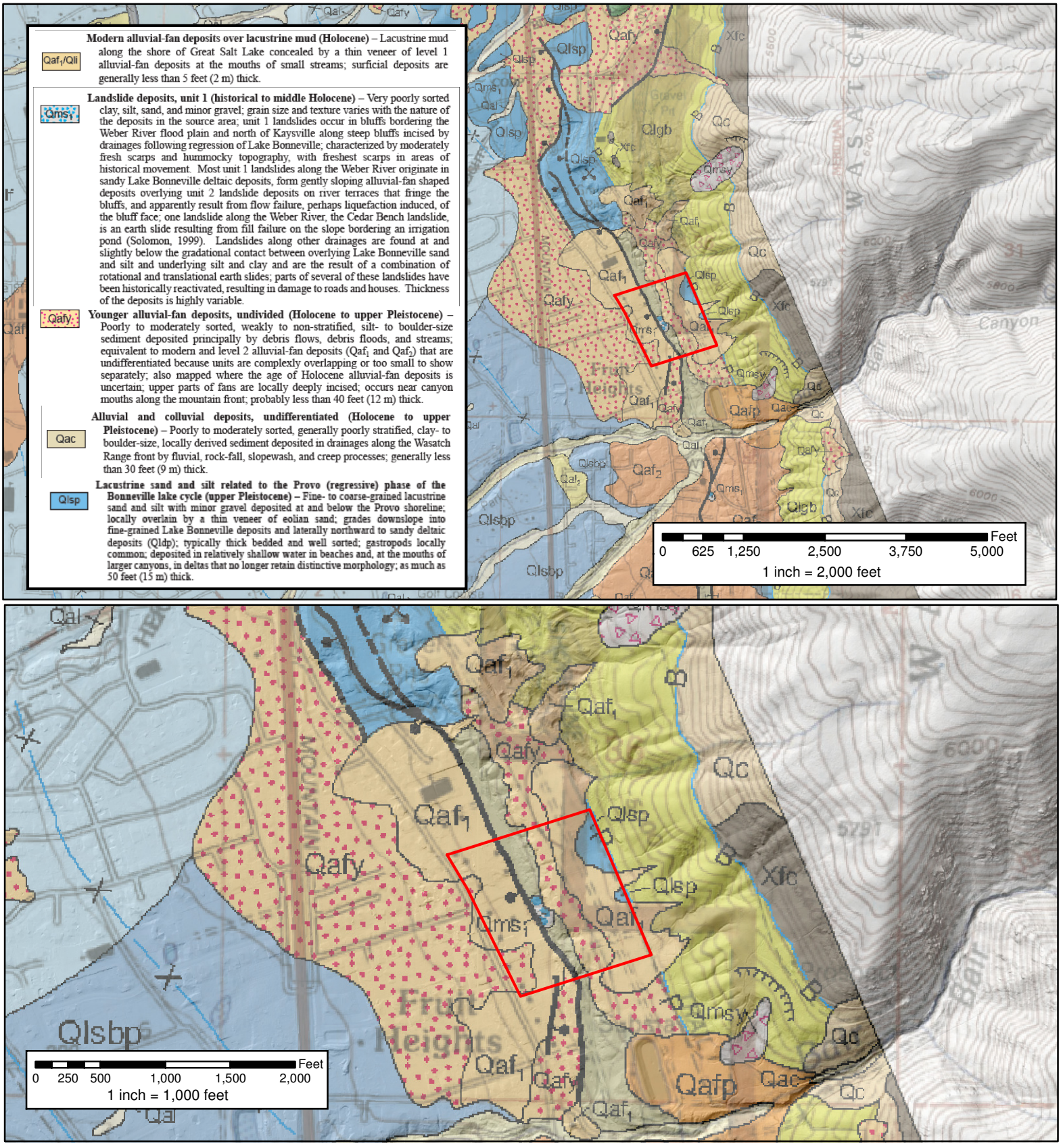
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Fruit Heights, Utah
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Hillshade Map

Plate 3



Legend

Approximate Site Boundary

Basemap:
Surficial Geologic Map of Part of the Kaysville Quadrangle, Davis County, Utah, Solomon, 2007. Hillshades derived from 2013-2014 0.5m LiDAR provided by the State of Utah AGRC.

N

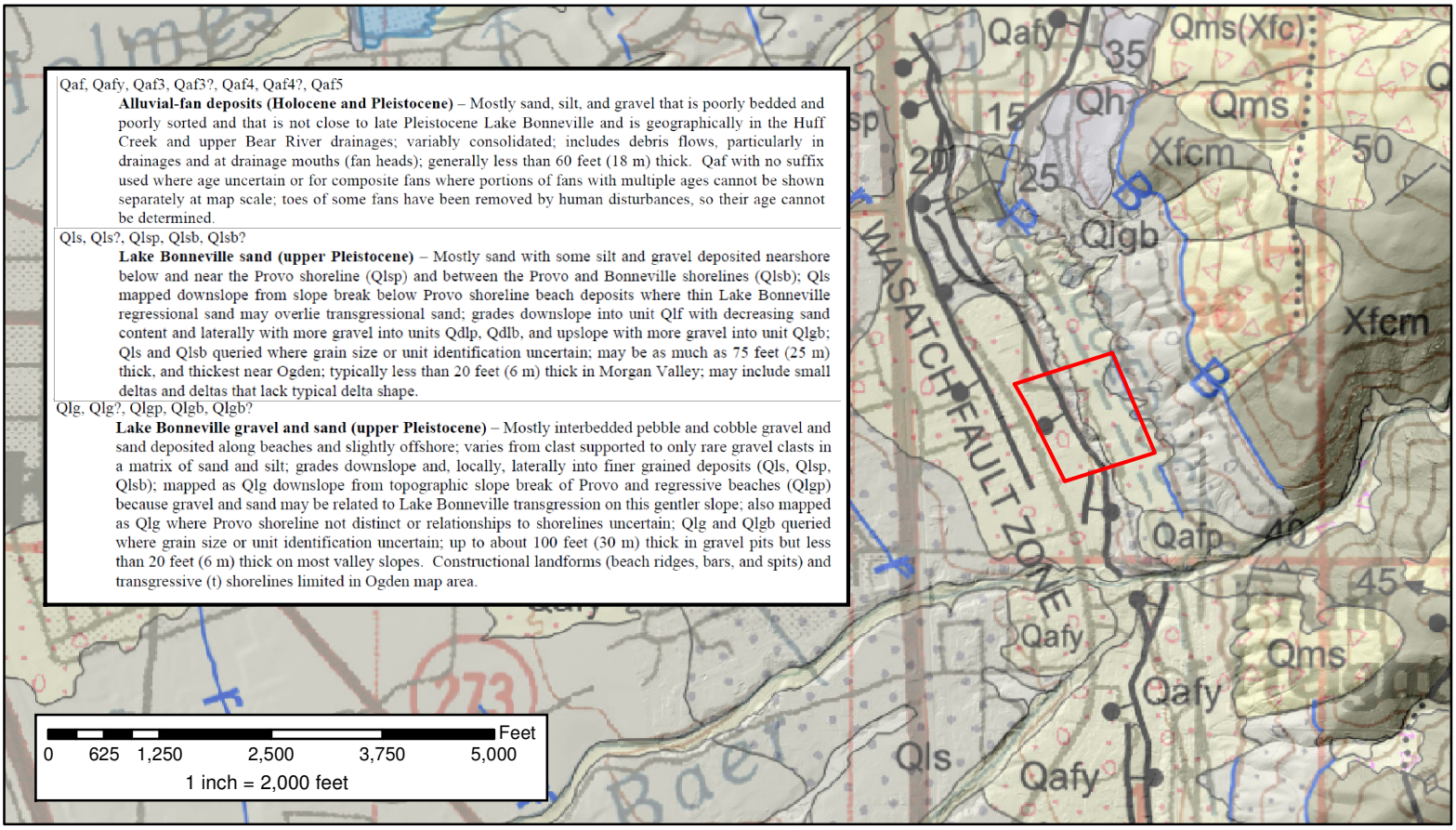
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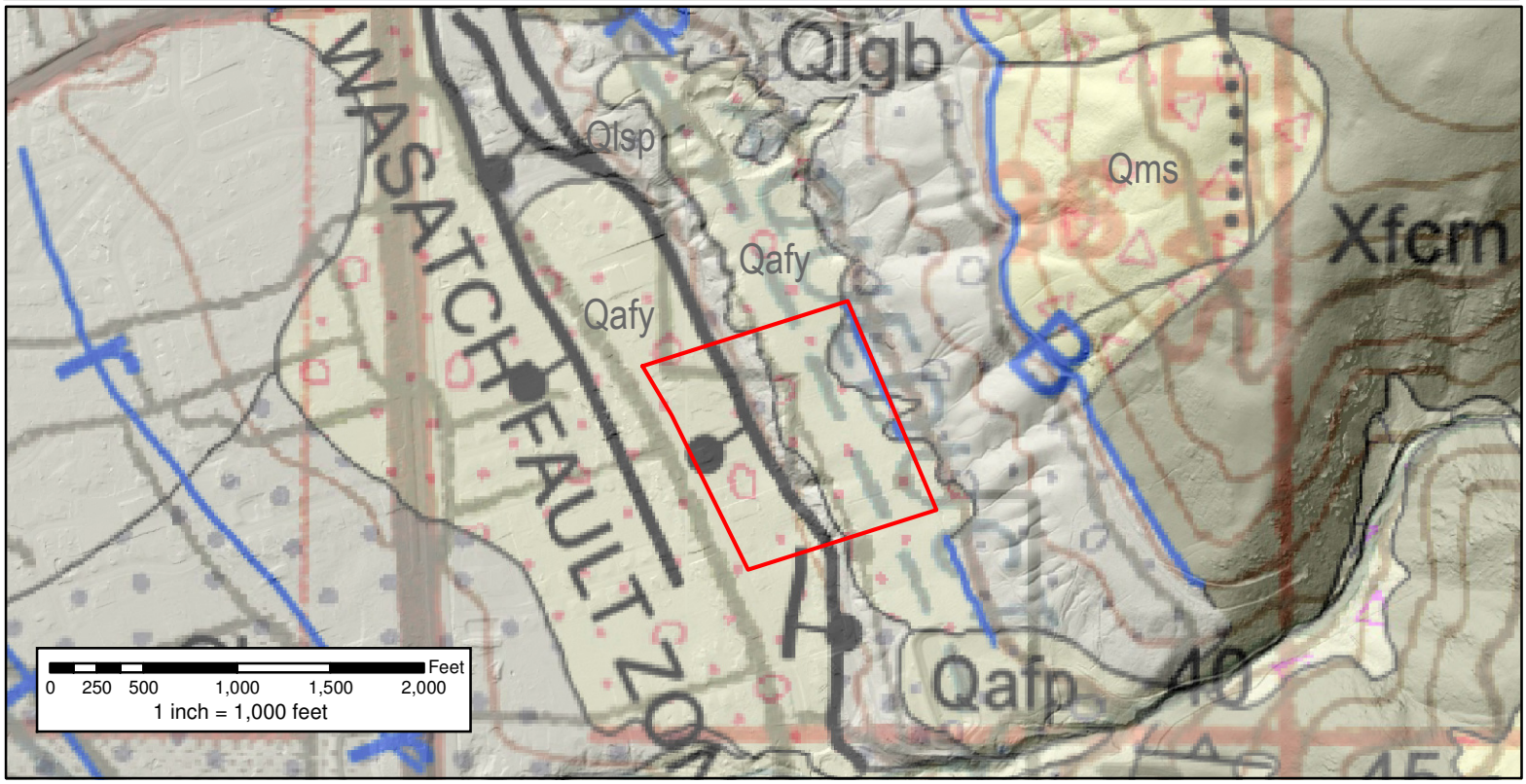
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Site Vicinity Geologic Map

Plate 4





Legend

Approximate Site Boundary

Basemap:
Interim Geologic Map of the Ogden 30' x 60' Quadrangle, Box Elder, Cache, Davis, Morgan, Rich and Summit Counties, Utah, and Uinta County, Wyoming, Coogan and King, 2016.
Hillshades derived from 2013-2014 0.5m LiDAR provided by the State of Utah AGRC.

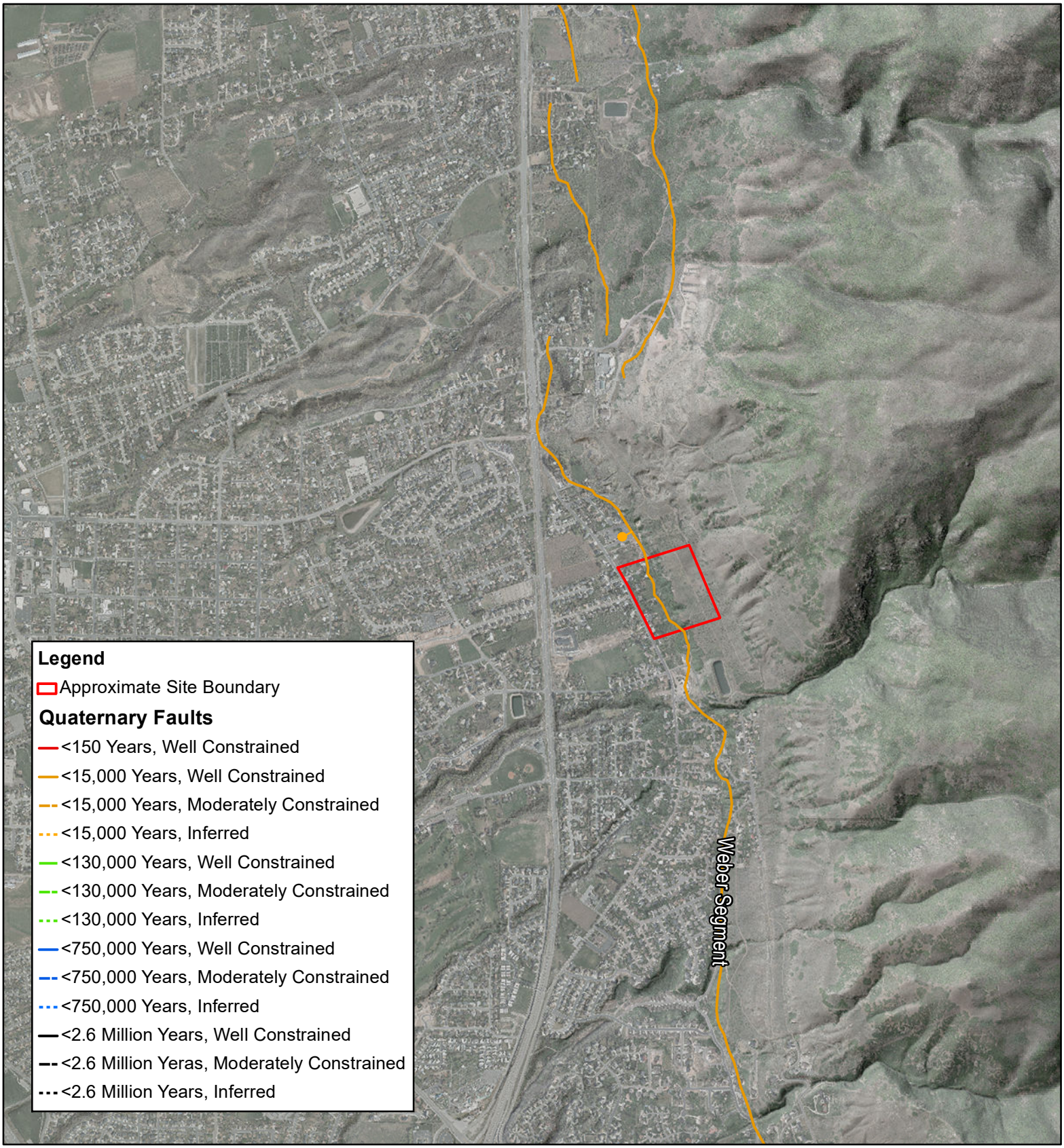
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City of Fruit Heights
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Fruit Heights, Utah
Project Number: 1058-004
Site Vicinity 30x60 Geologic Map

Plate 5

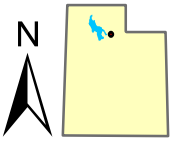


0 625 1,250 2,500 3,750 5,000 Feet

1 inch = 2,000 feet

Basemap:
UGS Quaternaty Fold and Fault Database, 2012
12.5 cm HRO aerial imagery and hillshades derived from 2013-2014 0.5m LiDAR provided by the State of Utah AGRC.

N



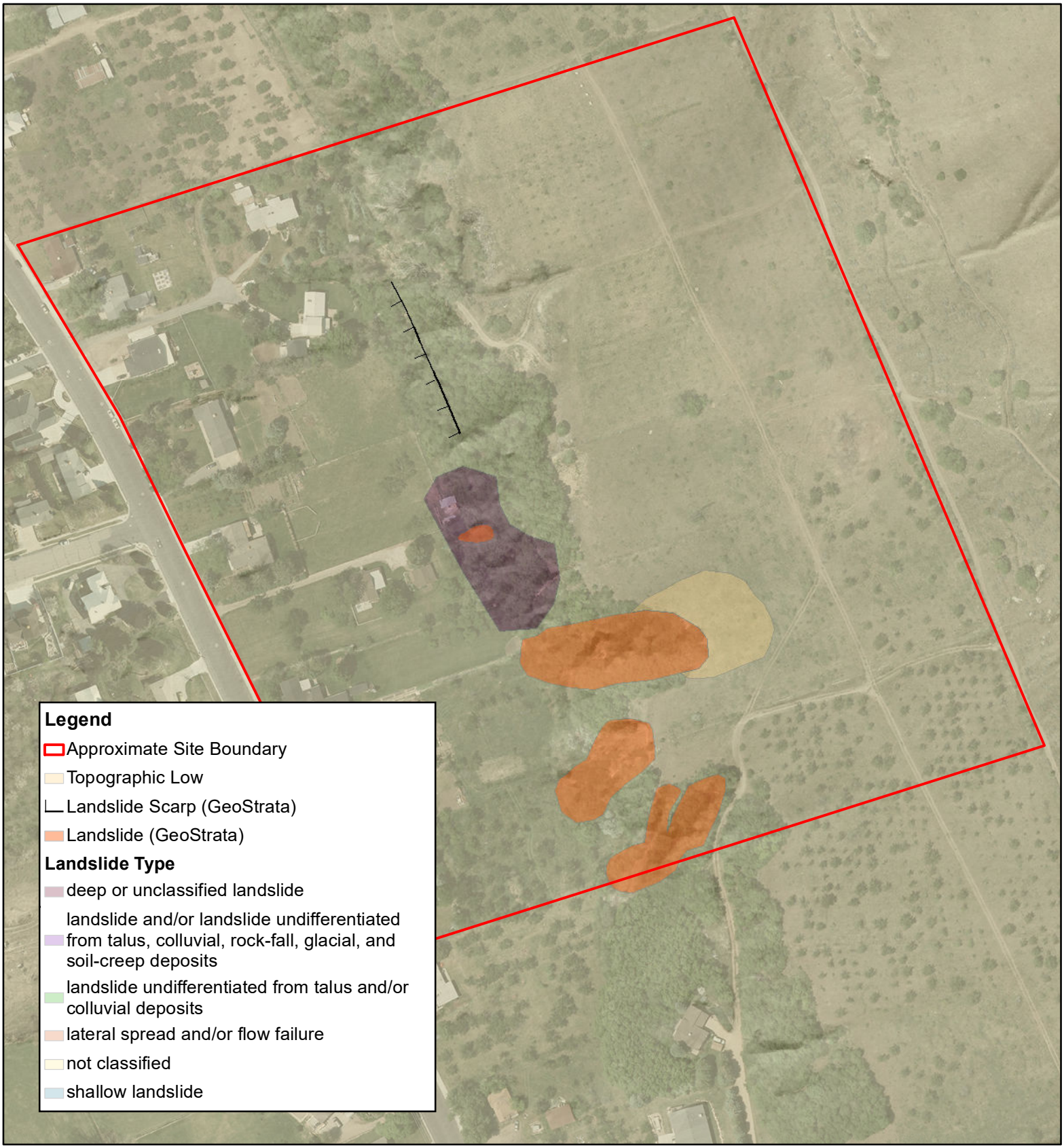
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Quaternary Fault Map

Plate 6



Feet
0 62.5 125 250 375 500

1 inch = 200 feet

Basemap:
Landslide Maps of Utah, Ogden 30' x60' Quadrangle, Elliot and Harty, 2016. 2012 12.5 cm HRO aerial imagery and hillshades derived from 2013-2014 0.5m LiDAR provided by the State of Utah AGRC.

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Landslide Hazard Map

Plate 7



Legend

- Approximate Site Boundary
- StreamsNHDHighRes
- LakesNHDHighRes

0 125 250 500 750 1,000 Feet

1 inch = 400 feet

Basemap:
National Hydrology Dataset, 2012 12.5 cm HRO aerial imagery and hillshades derived from 2013-2014 0.5m LiDAR provided by the State of Utah AGRC.

N

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City of Fruit Heights
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Drainage Map

Plate 8



Legend

- Approximate Site Boundary
- Minor Debris Flow

0 62.5 125 250 375 500 Feet

1 inch = 200 feet

Basemap:
2017 aerial imagery provided by Google Earth. Hillshades derived from 2013-2014 0.5m LiDAR provided by the State of Utah AGRC.

N

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City of Fruit Heights
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Fruit Heights, Utah
Project Number: 1058-004

Plate 9

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Debris Flow Hazard Assessment
Francis Fire Burn
Fruit Heights, Utah

GeoStrata Job No. 1058-008

March 19, 2020

Prepared for:

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Debris Flow Hazard Assessment
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March 19, 2020

TABLE OF CONTENTS

1.0 EXECUTIVE SUMMARY 1
2.0 INTRODUCTION 2
2.1 PURPOSE AND SCOPE OF WORK 2
2.2 PROJECT DESCRIPTION 2
3.0 METHODS OF STUDY 4
3.1 OFFICE INVESTIGATION 4
3.2 FIELD INVESTIGATION 4
4.0 GEOLOGIC CONDITIONS 5
4.1 GEOLOGIC SETTING 5
4.3 TECTONIC SETTING 5
5.0 GENERALIZED SITE CONDITIONS 7
5.1 SURFACE CONDITIONS 7
6.0 FIELD INVESTIGATION OF DRAINAGES 8
6.1 FIELD RECONNAISANCE 8
6.2 DESCRIPTION OF DRAINAGE BASINS 9
6.2.1 Basin A 9
6.2.2 Basin B 9
6.2.3 Basin C 10
6.2.4 Basin D 10
6.2.5 Basin E 11
6.2.6 Basin F 11
6.2.7 Basin G 11
6.2.8 Basin H 12
6.2.9 Basin I 12
7.0 GEOLOGIC HAZARDS 14
7.1 ALLUVIAL FAN FLOODING 14
7.1.1 Estimates of Debris Volume 15
8.0 GEOLOGIC HAZARDS SUMMARY AND CONCLUSIONS 23
9.0 CLOSURE 24
10.0 REFERENCES CITED 25

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APPENDICES

Appendix A Plate A-1 – Site Vicinity Map
Plate A-2 – Topographic Map
Plate A-3 – Hillshade Map
Plate A-4 – Site Vicinity Geologic Map
Plate A-5 – Site Vicinity 30x60 Geologic Map
Plate A-6 – Debris Flow Assessment Map
Plate A-7 – Extent of Alluvial Fan
Plate A-8 – Basin Characteristics
Plate A-9 – Debris Catchment Areas

Appendix B Plate B-1 to B-8 – Site Photos

Appendix C Plate C-1 to C-20 – Cross Sections

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1.0 EXECUTIVE SUMMARY

The purpose of this investigation and supplemental report is to investigate the alluvial fan flooding and debris flow hazard for several unnamed drainages that trend generally east-west through the Francis Fire burned area located along the west facing foothills of the Wasatch Mountains on the eastern margin of Fruit Heights, Utah.

The area of study comprises the Francis Fire burn scar area located on the western foothills of the Wasatch Mountains along the eastern margin of Fruit Heights, Utah at an elevation ranging from approximately 4,850 to 6,700 feet above sea level. The Francis Fire burned approximately 374 acres. Approximately 364 acres of the burn scar area is owned by the United Stated Forestry Service and approximately 10 acres are privately owned. A large orchard area is located at the base of the native hillside that comprises the burn scar. Established residential neighborhoods are currently located approximately 700 feet downslope and southwest of the burn area.

Based on the debris flow volumes calculated using the unit-volume analysis and post fire debris flow methods, we assess the alluvial fan flooding hazard at the mouths of the subject drainages to be high. Two low spots created by manmade excavation of surficial soils currently exist in the northwest margin of the study area. These two excavated areas could potentially accommodate the collection of debris from Basin I, Basin H and Basin G during an alluvial fan flooding event from those drainage basins. It should be noted that these are not engineered detention basins and that currently unassessed runup velocities and basin volumes could allow the debris flow to overrun the potential collection areas rather than deposit sediment in these areas. It is therefore our opinion, that while these areas may provide some benefit, they should not be relied upon for mitigation of the alluvial fan flooding hazard from these three drainage basins. It is also important to note, that wherever these excavated areas are located on U.S. Forest Service land, the forest service will give no permission to improve or maintain these areas. Our understanding of the U.S. Forest Service's position on this issue is based on recent communication between the Forest Service and the City of Fruit Heights. As such, it is our opinion that these areas should not be relied upon for debris flow and alluvial fan flooding mitigation. GeoStrata recommends that debris flow fencing, berms and/or other structures be considered for installation along the base of the slope as a temporary mitigation method to collect debris and reduce the velocity of the debris flow and alluvial fan flooding.

NOTICE: The scope of services provided within this report are limited to the assessment of the subsurface conditions for the proposed development. This executive summary is not intended to replace the report of which it is part and should not be used separately from the report. The executive summary is provided solely for purposes of overview. The executive summary omits a number of details, any one of which could be crucial to the proper application of this report.

2.0 INTRODUCTION

2.1 PURPOSE AND SCOPE OF WORK

The purpose of this investigation and supplemental report is to investigate the alluvial fan flooding and debris flow hazard for several unnamed drainages that trend generally east-west through the Francis Fire burned area located along the west facing foothills of the Wasatch Mountains on the eastern margin of Fruit Heights, Utah. The location of the site and the unnamed drainages that are assessed in this report are shown on Plate A-1 Site Vicinity Map. The work performed for this report was performed in accordance with our proposal, dated October 3, 2019 and signed October 9, 2019. Our scope of services included the following:

- Review of available references and maps of the area.
- Stereographic aerial photograph interpretation of aerial photographs covering the site area.
- Review of the sub-meter Wasatch Front lidar elevation data (2013 to 2014) obtained from the State of Utah AGRC.
- Geologic reconnaissance of the site by an engineering geologist to observe and document pertinent surface features indicative of possible debris flow hazards and to collect cross-sections of drainage; and
- Evaluation of our observations combined with existing information to assess the potential debris flow hazard and prepare this written report with conclusions and recommendations for assessed debris flow volumes of the unnamed drainage.

The recommendations contained in this report are subject to the limitations presented in the Limitations section of this report.

2.2 PROJECT DESCRIPTION

The area of study comprises the Francis Fire burn scar area located on the western foothills of the Wasatch Mountains along the eastern margin of Fruit Heights, Utah at an elevation ranging from approximately 4,850 to 6,700 feet above sea level. The Francis Fire was a lightning caused fire that was first detected on September 16, 2019. The Francis Fire burned approximately 374 acres. Approximately 364 acres of the burn scar area is owned by the United Stated Forestry Service and approximately 10 acres are privately owned. The hillside within the burn scar slopes to the west approximately 30 to 70% with the majority of the burned area containing slopes of 55 to 70%. A large orchard area is located at the base of the native hillside that comprises the burn

scar. Established residential neighborhoods are currently located approximately 700 feet downslope and southwest of the burn area. The location and approximate boundaries of the Francis Fire Burned area are shown on the Site Vicinity Map and the Topographic Map included in the Appendix of this report (Plate A-1; Plate A-2).

3.0 METHODS OF STUDY

3.1 OFFICE INVESTIGATION

To prepare for the investigation, GeoStrata reviewed pertinent literature and maps listed in the references section of this report, which provided background information on the geologic history of the hillside burned by the Francis Fire and the locations of suspected or known alluvial fan flooding hazard areas adjacent to the mountain front west of the burn scar area (Elliot and Harty, 2010; Black and others, 2003; UGS, 2016, Coogan and King, 2016; Solomon, 2007). A stereographic aerial photograph interpretation was performed for the study area using two sets of stereo aerial photographs obtained from the UGS as shown in Table 1.

Source	Photo Number	Date	Scale
AMS	AMS_121-16-2963_B	August 10, 1953	64,000
AMS	AMS_121-9-1555	June 26, 1953	64,000

Table 1: Aerial Stereosets.

GeoStrata also conducted a review of hillshade imagery derived from 2013-2014 0.5-meter lidar obtained from the State of Utah AGRC to assess the study area for visible alluvial fan deposits. The lidar elevation data was used to create hillshade imagery that could be reviewed for assessment of geomorphic features related to alluvial fan flooding hazards (Plate A-3 Hillshade Map).

3.2 FIELD INVESTIGATION

An engineering geologist investigated the geologic conditions within the general site area. A field geologic reconnaissance was conducted to observe existing geologic conditions and to assess existing surficial evidence of alluvial fan flooding and debris flow. GeoStrata also collected cross sectional data of the unnamed drainages to further assess the alluvial fan flooding and debris flow hazard at the site.

4.0 GEOLOGIC CONDITIONS

4.1 GEOLOGIC SETTING

The site is located within the Salt Lake Valley along the foothills of the Wasatch Mountain Range in Fruit Heights, Utah at approximately 4,715 feet above sea level. The Salt Lake basin is a deep, sediment-filled structural basin of Cenozoic age flanked by the Wasatch Range and Wellsville Mountains to the east and the Promontory Mountains, the Spring Hills, and the West Hills to the west (Hintze, 1980). The southern portion of the Salt Lake Basin is bordered on the west by the east shore of the Great Salt Lake. The Wasatch Range is the easternmost expression of pronounced Basin and Range extension in north-central Utah (Stokes, 1986).

The near-surface geology of the Salt Lake Valley is dominated by lake sediments which were deposited within the last 30,000 years during the high stand of the Lake Bonneville Cycle (Scott and others, 1983; Hintze, 1993; Leggette and Taylor, 1937; King and others, 2008). As Lake Bonneville receded, streams began to incise large deltas that had formed at the mouths of major canyons along the Wasatch Range. The eroded material was then deposited in shallow lakes and marshes in the basin and in a series of recessional deltas and alluvial fans. Sediments toward the center of the valley are predominately deep-water deposits of clay, silt and fine sand whereas sediments closer to the mountain fronts are shallow-water deposits of coarse sand and gravel. However, these deep-water deposits are in places covered by a thin post-Bonneville alluvial cover.

4.2 SITE GEOLOGY

The surficial deposits within the study area are shown on Plate A-4 Site Vicinity Geologic Map and Plate A-5 Site Vicinity 30x60 Geologic Map. The geology within the study area as shown on Plate 4 is mapped as a series of alluvial fan deposits as well as a small landslide deposit located in the center of the study area (Sorensen and Crittenden, 1979). These deposits are mapped as overlying lacustrine sand and silt related to the Provo phase of the Bonneville lake cycle. The geology within the study area as shown on Plate A-5 is mapped as a series of alluvial fan deposits overlying Lake Bonneville sand deposits (Coogan and King, 2016).

4.3 TECTONIC SETTING

The Weber segment of the Wasatch Fault Zone (WFZ) is located approximately 700 feet

southwest of the site (WFZ) (UGS, 2016, Coogan and King, 2016; Solomon, 2007; Hecker, 1993). The Weber segment is the second largest segment of the WFZ and extends north to south 56 km from the Salt Lake Salient, a ridge of Paleozoic and Tertiary bedrock that extends west of the Wasatch Front, to the Pleasant View Salient which marks the boundary between the Weber segment and the Brigham City segment (DuRoss and others, 2009). The geometry of linkage between the main rupture zones in the Weber segment and faults in the interior of the Salt Lake salient is not clear. Surface scarps at the southern margin of the salient are discontinuous but apparently extend into the large normal fault along the eastern boundary of the segment. There is no reported evidence for Quaternary movement on this fault in the interior of the salient, so presumably the Quaternary ruptures have not reactivated most of this fault. Prior paleoseismic studies report that the Weber segment of the WFZ is thought to have experienced four surface faulting seismic events since the middle Holocene. Nelson and others (2006) report four surface faulting seismic events since the middle Holocene with the most recent event being a partial segment rupture which occurred approximately 500 years ago resulting in a 1.6 feet surface rupture displacement. DuRoss and others (2009) report evidence from the 2007 Rice Creek trench site of as many as six surface faulting seismic events during the Holocene with four surface faulting events in approximately the past 5,400 years. This data from DuRoss and others (2009) supports the partial segment surface rupture timing reported by Nelson and others (2006). A location near Kaysville, Utah indicated that the Weber Segment has a measurable offset of 1.4 to 3.4 meters per event (McCalpin and others, 1994). The Weber Segment may be capable of producing earthquakes as large as magnitude 7.5 (Ms). The consensus preferred recurrence interval for the Weber segment, determined by the Utah Quaternary Fault Working Group, is approximately 1,400 years for the past four surface fault rupture earthquakes (Lund, 2005).

Analysis of the ground shaking hazard along the Wasatch Front suggests that the Wasatch Fault Zone is the single greatest contributor to the seismic hazard in the Salt Lake City region. The Weber segment of the WFZ shows evidence of Holocene-aged movement, and is therefore considered active.

5.0 GENERALIZED SITE CONDITIONS

5.1 SURFACE CONDITIONS

As previously stated, the project site is located on a recently burned hillside in Fruit Heights, Utah (Plate A-1 Site Vicinity Map; Plate A-2 Topographic Quadrangle). At the time of our site visit, GeoStrata observed that much of the hillside was severely burned with only few patches of new growth. Scrub oak trees appeared to still be rooted in the ground, however, the trunks and branches were charred. Some trees were entirely void of leaves while others still had leaves attached that were completely dried out. Most of the surficial deposits within the drainages observed to contain gravel and fine-grained soils with occasional cobbles and boulders with the exception of one drainage which was observed to contain intact bedrock outcroppings. Sub-angular boulders ranging from approximately 2 feet to 4 feet in diameter were observed in the southern portion of the site.

6.0 FIELD INVESTIGATION OF DRAINAGES

6.1 FIELD RECONNAISANCE

Field investigations and observations used to assess the debris flow potential, probability and magnitude can be categorized into two areas of study (Giraud, 2005):

1. Channel Investigation – Studies of debris flows indicate that the majority of material/debris transported onto the alluvial fan comes from existing deposits within the defined drainage channel. The unit volume technique is commonly used to assign applicable debris yield rates (unit volume along distinct reaches of the channel) in order to approximate the potential debris volume.

2. Alluvial Fan Investigation – the thickness of debris deposits measured on the alluvial fan contribute to an understanding of past debris flow magnitude and potential run-out distance.

GeoStrata completed a site reconnaissance of the drainages on October 24, 2019. The site reconnaissance included observations of the surficial deposits in the drainage and collection of cross-sections from each drainage. Along with GeoStrata's field observations, geologic mapping of the site (Plate A-4 Site Vicinity Geologic Map; Plate A-5 Site Vicinity 30'x60' Geologic Map) was reviewed by GeoStrata as part of this investigation. The drainage basins for each drainage and profile cross section locations that were delineated from 2013-2014 0.5-meter lidar are shown on Plate A-6 Debris Flow Assessment Map. A cross section from each drainage was collected in the field at the time of our site visit to compare to those delineated in the office.

The cross-sectional geometry of the channels within each drainage is variable and ranged from a narrow channel bottom to a shallow and broad channel bottom. It was our objective to produce cross-sections that would be representative of the various geometries that exist in the main channel of each drainage. Evidence observed within the drainages suggests that surface water is present in the channels during periods of high runoff, however, no water was observed in any of the drainages at the time of our site reconnaissance. The drainages are predominantly vegetated with grass and scrub oak that is moderately to severely burned. The following sections present results of our field investigations of each drainage basin.

6.2 DESCRIPTION OF DRAINAGE BASINS

There are several drainages within the site ranging from small to moderate in size. A total of nine drainage basins were assessed as part of this study (Plate A-6 Debris Flow Assessment Map), which includes four major drainage basins (Basin C, Basin F, Basin H and Basin I) and five minor drainage basins (Basin A, Basin B, Basin D, Basin E, and Basin G). Water was not observed in any of the drainages observed in these basins and therefore these drainages are considered perennial drainages. Descriptions of each drainage and their estimated stored debris yield rates are discussed in the following paragraphs.

6.2.1 Basin A

Two small drainage channels were observed in Basin A. The geometry within the drainage channels were observed to range from V-shaped to broad. Dense scrub oak was observed in the lower and central portions of Basin A. The remainder was observed to be vegetated by sagebrush and grasses. Basin A is reported as having less than 1 acre burned at moderate to high severity.

In order to estimate potential debris discharge volumes from the two drainages, GeoStrata produced cross-sections in three different locations to more accurately estimate the amount of debris currently available for transport (Plate C-1 to C-3). The amounts of stored debris yield rates within Basin A varied and were calculated to be approximately 0.13 ft^3/ft to 3.77 ft^3/ft. The approximate locations of channel cross-sections are shown on the Debris Flow Assessment Map (Plate A-6).

6.2.2 Basin B

The geometry of the active channel bottom of the drainage in Basin B was observed to range from V-shaped to broad (Plate B-1). The channel is predominantly vegetated by grasses with only few scrub oaks. The burn severity in Basin B is considered predominantly low to moderate.

In order to estimate potential debris discharge volumes from these drainages, GeoStrata produced cross-sections in two different locations to more accurately estimate the amount of debris currently available for transport (Plate C-4 to C-5). The amounts of stored debris yield rates within Basin B varied slightly and were calculated to be approximately 0.70 ft^3/ft to 1.34 ft^3/ft. The approximate locations of channel cross-sections are shown on the Debris Flow Assessment Map (Plate A-6).

6.2.3 Basin C

Basin C was observed to consist of a main drainage channel and two subsidiary drainage channels. The channel bottom within the main channel was observed to comprise of bedrock at two locations (Plate B-2 and Plate B-3) while the remainder of the channel was observed to consist of predominantly fine-grained soils with cobbles and boulders (Plate B-4). The geometry of the active channel bottom was observed to range from V-shaped to broad. The entire length of the main channel is approximately 2,186 feet while the two subsidiary drainages have a combined length of 241 feet and 388 feet. The channel was observed to be vegetated by grasses, sagebrush and scrub oak, however, the burn severity in Basin C is considered moderate to high in upper elevations of the basin and low in lower elevations.

In order to estimate potential debris discharge volumes from this drainage, GeoStrata produced cross-sections in six different locations within the channel to more accurately estimate the amount of debris currently available for transport (Plate C-3 to Plate C-8). The estimated debris available for transport in the areas where bedrock was observed (CS6, CS7 and CS10) is null and was not used in our total estimations for debris available for transport in Basin C. The amounts of stored debris yield rates within Basin C varied and were calculated to be between approximately 3.34 ft^3/ft and 13.34 ft^3/ft. The approximate locations of channel cross-sections are shown on the Debris Flow Assessment Map (Plate A-6).

6.2.4 Basin D

The geometry of the active channel bottoms in Basin D was observed to range from V-shaped to broad (Plate B-5). The channels are predominantly vegetated by grasses with only few scrub oaks. The burn severity in Basin D is considered predominantly low to moderate. The upper portions of Basin D have a moderate to high burn severity.

In order to estimate potential debris discharge volumes from the drainage, GeoStrata produced cross-sections in two different locations within the drainage channel to more accurately estimate the amount of debris currently available for transport (Plate C-14 and C-15). The amounts of stored debris yield rates within Basin D varied slightly and were calculated to be approximately 1.66 ft^3/ft to 1.80 ft^3/ft. The approximate locations of channel cross-sections are shown on the Debris Flow Assessment Map (Plate A-6).

6.2.5 Basin E

Basin E is a relatively small basin comprised of three small drainages. The active channel bottoms within each drainage were observed to be broad and to contain fine-grained soils. The vegetation within the drainages were observed to be predominately grasses with minor scrub oaks in the upper portions. The burn severity in Basin E is reported as predominantly low with less than 1 acre of moderate to high burn severity in the lower portions.

In order to estimate potential debris discharge volumes from this drainage, GeoStrata produced one cross-section in each drainage and one at the base of Basin E where the drainages converge to one drainage channel to more accurately estimate the amount of debris currently available for transport (Plate C-16 to Plate C-19). The amounts of stored debris yield rates within Basin E varied and were calculated to be between approximately 0.62 ft^3/ft and 4.25 ft^3/ft. The approximate locations of channel cross-sections are shown on the Debris Flow Assessment Map (Plate A-6).

6.2.6 Basin F

Drainage F is comprised of three drainages. The channel bottoms within Basin F were observed to consist of fine-grained soils with cobbles and boulders (Plate B-6). The geometry of the active channel bottoms were observed to range from V-shaped to broad. The total length of all the channels is approximately 2,505 feet. The channels were observed to be vegetated with grasses, sagebrush and scrub oak. The burn severity in Basin F is considered moderate to high in the mid to upper elevations of the basin and low in lower elevations.

In order to estimate potential debris discharge volumes from this drainage, GeoStrata produced cross-sections in seven different locations within the channel to more accurately estimate the amount of debris currently available for transport (Plate C-11 to Plate C-17). Two tributary drainages were also included in our debris flow estimations for Basin F. The amounts of stored debris yield rates within Basin F varied and were calculated to be between approximately 3.07 ft^3/ft and 15.39 ft^3/ft. The approximate locations of channel cross-sections are shown on the Debris Flow Assessment Map (Plate A-6).

6.2.7 Basin G

Basin G is a relatively small basin comprised of one active drainage channel. The active channel bottom was observed to be broad and to contain fine-grained soils with cobbles and boulders (C-

7 and C-8). The vegetation within the drainages were observed to be grasses and scrub oaks. The burn severity in Basin G is reported as predominantly low and approximately 1 acre of moderate to high burn severity.

In order to estimate potential debris discharge volumes from this drainage, GeoStrata produced two cross-section in each drainage and one at the base of Basin G where the drainages converge to one drainage channel to more accurately estimate the amount of debris currently available for transport (Plate C-27 to Plate C-28). The amounts of stored debris yield rates within Basin G are similar and were calculated to be between approximately 9.10 ft^3/ft and 9.31 ft^3/ft. The approximate locations of channel cross-sections are shown on the Debris Flow Assessment Map (Plate A-6).

6.2.8 Basin H

The channel bottom within Basin H was observed to comprise of fine-grained soils with cobbles and boulders. The geometry of the active channel bottom was observed to be broad near the bottom and V-shaped throughout the remainder of the channel (Plate B-7 and Plate B-8). Two tributary channels were included in our debris flow estimations for Basin H. The entire length of the channel, including the two tributary channels, is approximately 2,580 feet. The channel was observed to be vegetated by grasses and scrub oak. The burn severity in Basin H is considered moderate to high in upper elevations of the basin and predominantly low in lower elevations.

In order to estimate potential debris discharge volumes from this drainage, GeoStrata produced cross-sections in three different locations within the channel to more accurately estimate the amount of debris currently available for transport (Plate C-18 to Plate C-20). The amounts of stored debris yield rates within Basin H varied and were calculated to be between approximately 1.76 ft^3/ft and 2.26 ft^3/ft. The approximate locations of profile cross-sections are shown on the Debris Flow Assessment Map (Plate A-6).

6.2.9 Basin I

Basin I is comprised of two drainage channels. The geometry of the channel bottoms within these drainages varied and were observed to be broad and V-shaped. The vegetation within the drainage channels consisted predominantly of grass and scrub oak in the upper portions of the basin. Much of Basin I is considered to have low burn severity. Approximately 3 acres is identified as moderate to high burn severity and is located in the upper portions of Basin I.

In order to estimate potential debris discharge volumes from this drainage, GeoStrata produced cross-sections in eight different locations within the channels to more accurately estimate the amount of debris currently available for transport (Plate C-33 to Plate C-40). The amounts of stored debris yield rates within Basin I varied and were calculated to be between approximately 1.97 ft^3/ft and 8.83 ft^3/ft. The approximate locations of profile cross-sections are shown on the Debris Flow Assessment Map (Plate A-6).

7.0 GEOLOGIC HAZARDS

Geologic hazards can be defined as naturally occurring geologic conditions or processes that could present a danger to human life and property. These hazards must be considered before development of the site. There are several hazards that if present at the site should be considered in the design of habitable structures and other critical infrastructure. A memorandum titled "*Francis Fire Burn Scar Preliminary Assessment"* and dated November 26, 2019 was compiled by GeoStrata for this site. As previously stated, this is a supplemental report that will assess the debris flow hazard of the drainages trending generally east-west within the burn scar subject site as shown on Plate A-1, Plate A-2, Plates A-4 through A-9.

7.1 ALLUVIAL FAN FLOODING

Alluvial fan flooding is a potential hazard that exists in areas containing Holocene alluvial fan deposits. This type of flooding typically occurs as stream flows, hyperconcentrated flows or debris flows consisting of a mixture of water, soil, organic material, and rock debris with variations in sediment-water concentrations transported by fast-moving water flows. Stream flows contain approximately less than 20% sediment by volume and involve sediment transport by entrained and suspended sediment load (Bowman and Lund, 2016). Unconfined stream flows are referred to as sheetfloods which are spread over and occur in the distal areas of the alluvial fan or within unchanneled, broad, relatively flat-bottomed portions of drainages. Hyperconcentrated flows are alluvial fan flows with approximately between 20 to 60% sediment by volume whereas debris flows contain approximately 60% to 85% sediment by volume.

Alluvial fan flooding can be a hazard on or below alluvial fans or in stream channels above alluvial fans. Precipitation (rainfall and snowmelt) is generally viewed as an alluvial fan flood "trigger", but this represents only one of the many factors that contribute to alluvial fan flooding hazard. Vegetation, root depth, soil gradation, antecedent moisture conditions and long-term climatic cycles all contribute to the generation of debris and initiation of alluvial fan flooding. Events of relatively short duration, such as a fire, can significantly alter a basin's absorption of storm water and snowmelt runoff and natural resistance to sediment mobilization for an extended period of time. These factors are difficult to quantify or predict and vary not only between different watersheds, but also within each sub-area of a drainage basin. In general, there are two methods by which alluvial fan flooding can be mobilized: 1) when shallow landslides from channel side-slopes are conveyed in existing channels when mixed with water and 2) channel

scour where debris is initially mobilized by moving water in a channel and then the mobilized debris continues to assemble and transport downstream sediments.

Based on review of published geologic maps, our stereographic aerial photograph interpretation, our review of the hillshade imagery derived from the 0.5-meter lidar elevation data (2013-2014) and our field observations, modern (upper Holocene) alluvial fan deposits are mapped at the mouth of the drainages assessed as part of this study (Plate A-4 Site Vicinity Geologic Map; Plate A-5 Site Vicinity 30x60 Geologic Map). In addition, Holocene to upper Pleistocene alluvial fan deposits are mapped as underlying the modern alluvial fan deposits further down slope. As previously mentioned, the geometry of the channels within the drainages were observed to vary from V-shaped to broad. No water was observed to be flowing in any of the drainage at the time of our site visit in the beginning of October of 2019. No subsurface exploration to evaluate the types of debris flow deposits sourced by the unnamed drainage was conducted as part of this investigation. Based on our review of published geologic maps, our aerial photograph interpretation, our review of hillshade imagery derived from 0.5-meter lidar and our field observations, the alluvial fan flooding hazard is considered high and our more in-depth assessment of this hazard is addressed in subsequent paragraphs.

7.1.1 Estimates of Debris Volume

The prediction of total debris volumes is complex and dependent on several factors which include but are not limited to precipitation and vegetation as previously mentioned. While methods of initiation differ, our observations of the drainage basins and channels lead us to assume that under existing conditions the majority of debris currently available for transport in the drainages would be mobilized from existing deposits within their developed channels beds.

Since GeoStrata did not conduct a subsurface investigation to classify the type of alluvial fan flooding (stream flow, hyperconcentrated flow and debris flow), it is the opinion of GeoStrata that the alluvial fan flooding hazard sourced by the drainage could potentially impact the area delineated as Alluvial Fan Deposit on Plate A-7 Extent of Alluvial Fan.

There are several methods available for predicting peak discharge rates and total debris flow volumes associated with debris-flows. The methods used in our analysis for this investigation are discussed below. Results of each of the methods of analysis are presented below.

Unit-Volume Analysis

The unit-volume analysis method involves measuring and estimating the stored erodible sediment in the channel. Cross-sections are taken at various points along a channel and the geometry of the channel is used to estimate the sediment stored in the bottom of the channel (Giraud, 2005). Estimating channel sediment volume available for bulking is critical because study of historical debris flows indicates that 80% to 90% of the debris flow volume comes from bulking of sediment from the bottom of the channel (Bowman and Lund, 2016).

Each of the streambed cross sections used in our analysis were collected from 2013-2014 0.5-m lidar. Estimates for debris yield at each of these cross sections was calculated as volume per linear foot of channel and this yield was then extrapolated beyond the investigation locations along the length of the channel in order to approximate the potential total debris yield for the unnamed drainage as presented in Table 3 below. As previously mentioned, Basin C was observed to contain bedrock outcroppings in the areas of the following cross sections: CS6, CS7 and CS10. The total debris flow volumes presented below were calculated by assuming flows with sediment loads of 50% which mobilize all of the estimated available sediment within the stream channels. The accuracy of these volumes could be improved by using design level flood flow rates and total volumes for each of the drainages from a hydrologic study report in conjunction with the estimated total volumes of erodible sediment, as well as by excavating test pits within the channels to observe the depth stored sediment at various points. Below is a table summarizing our results:

Basin	Cross Section	Erodible Cross Sectional Area (ft^2)	Reach of Cross Section (ft)	Erodible Sediment (ft^3)	Erodible Sediment (ac-ft)	Total Volume of Erodible Sediment		Total Debris Flow Volume
						V ft^3	V ac-ft	V ac-ft
A	1	3.77	185	697.35	0.02	**1000.57**	**0.02**	**0.05**
	2	0.13	126	16.75	0.00			
	3	1.75	164	286.47	0.01			
B	4	0.67	260	174.00	0.00	**1285.46**	**0.03**	**0.06**
	5	4.94	225	1111.46	0.03			
C	6	NA	NA	NA	NA	**13738.39**	**0.32**	**0.63**
	7	NA	NA	NA	NA			
	8	3.82	790.00	3019.05	0.07			
	9	4.56	242.00	1103.33	0.03			
	10	2.25	129.00	290.03	0.01			
	11	3.34	531.00	1773.40	0.04			
	12	11.07	241.00	2667.38	0.06			
	13	13.34	388.00	5175.23	0.12			

Basin	Cross Section	Erodible Cross Sectional Area (ft^2)	Reach of Cross Section (ft)	Erodible Sediment (ft^3)	Erodible Sediment (ac-ft)	Total Volume of Erodible Sediment		Total Debris Flow Volume
						V ft^3	V ac-ft	V ac-ft
D	14	1.66	262	434.03	0.01	**814.33**	**0.02**	**0.04**
	15	1.80	211	380.31	0.01			
E	16	4.25	97	412.28	0.01	**2335.78**	**0.05**	**0.11**
	17	1.85	314	580.07	0.01			
	18	3.76	309	1161.47	0.03			
	19	0.62	292	181.97	0.00			
F	20	12.87	158	2034.21	0.05	**26765.30**	**0.61**	**1.23**
	21	15.39	375	5771.84	0.13			
	22	3.07	256	785.93	0.02			
	23	11.83	206	2436.49	0.06			
	24	8.96	290	2597.17	0.06			
	25	12.96	785	10172.83	0.23			
	26	6.82	435	2966.84	0.07			
G	27	9.10	226	2057.30	0.05	**5194.76**	**0.12**	**2.74**
	28	9.31	337	3137.46	0.07			
H	29	2.26	673	1522.64	0.03	**7788.19**	**0.18**	**0.36**
	30	1.90	988	1875.25	0.04			
	31	1.76	919	1621.33	0.04			
	32	4.42	627	2768.97	0.06			
I	33	7.70	347	2672.20	0.06	**10871.17**	**0.25**	**0.50**
	34	7.65	91	696.58	0.02			
	35	4.87	249	1212.52	0.03			
	36	2.00	108	215.59	0.00			
	37	4.02	551	2215.24	0.05			
	38	8.83	244	2154.82	0.05			
	39	6.20	208	1289.06	0.03			
	40	1.97	211	415.14	0.01			

Table 3: Volume of Sediment in Stream Channel

Post-fire Condition Assessment

The Western U.S. regression model was also used to estimate fire-related debris flow volumes (Gartner and others, 2008; Giraud and Castleton, 2009; Cannon and others 2010). The model estimates debris flow volumes as:

$$\ln V = 7.2 + 0.6(\ln A) + 0.7(B)^{1/2} + 0.2(T)^{1/2} + 0.3$$

where:

V = volume (cubic meters)

A = basin area with slopes greater than or equal to 30% (square kilometers)
B = basin area burned at moderate and high severity (square kilometers)
T = total storm rainfall (millimeters)

Based on elevation data derived from 2013-2014 0.5-m lidar, slopes greater than or equal to 30% ranged from 0% to 100% (Table 4: Basin Characteristics and Plate A-7 Basin Characteristics). GeoStrata utilized the burn severity map provided by the National Forest Service to estimate the total area burned at moderate and high severity within each drainage which ranged from 22% to 80%.

Basin	A	B	C	D	E	F	G	H	I
Slopes Equal to or Greater than 30%	67%	100%	90%	89%	75%	83%	80%	76%	76%
Basin Area Burned at Moderate and High Severity	0%	25%	47%	22%	0%	56%	20%	49%	14%

Table 4: Basin Characteristics

Cannon and others (2010) recommend evaluation of debris flow events in response to low recurrence (<2-10 years), low-duration (<1 hr) rainstorms. Total storm rainfall was taken from the NOAA Atlas 14, Volume 1, Version 5 Point Precipitation Frequency Estimates for rainstorm events with 60-minute durations with a recurrence interval of 2, 5, 10, and 100 years. Below is a summary of our results for each basin.

Basin A

B	0.000	sq km		
A	0.008	sq km		
R-2 year	14.4	mm		
R-5 year	19.6	mm		
R-10 year	24.4	mm		
R-100 year	48.5	mm		
V-2 year	214.5	m^3	0.2	ac-ft
V-5 year	243.8	m^3	0.2	ac-ft
V-10 year	269.6	m^3	0.2	ac-ft
V-100 year	404.7	m^3	0.3	ac-ft

Basin B

B	0.004	sq km		
A	0.016	sq km		
R-2 year	14.4	mm		
R-5 year	19.6	mm		
R-10 year	24.4	mm		
R-100 year	48.5	mm		
V-2 year	339.9	m^3	0.3	ac-ft
V-5 year	386.3	m^3	0.3	ac-ft
V-10 year	427.3	m^3	0.3	ac-ft
V-100 year	641.3	m^3	0.5	ac-ft

Basin C

B	0.057	sq km		
A	0.109	sq km		
R-2 year	14.4	mm		
R-5 year	19.6	mm		
R-10 year	24.4	mm		
R-100 year	48.5	mm		
V-2 year	1207.8	m^3	1.0	ac-ft
V-5 year	1372.6	m^3	1.1	ac-ft
V-10 year	1518.3	m^3	1.2	ac-ft
V-100 year	2278.5	m^3	1.8	ac-ft

Basin D

B	0.008	sq km		
A	0.032	sq km		
R-2 year	14.4	mm		
R-5 year	19.6	mm		
R-10 year	24.4	mm		
R-100 year	48.5	mm		
V-2 year	524.8	m^3	0.4	ac-ft
V-5 year	596.4	m^3	0.5	ac-ft
V-10 year	659.7	m^3	0.5	ac-ft
V-100 year	990.1	m^3	0.8	ac-ft

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Basin E

B	0.012	**sq km**		
A	0.012	**sq km**		
R-2 year	14.4	**mm**		
R-5 year	19.6	**mm**		
R-10 year	24.4	**mm**		
R-100 year	48.5	**mm**		
V-2 year	295.5	**m^3**	0.2	**ac-ft**
V-5 year	335.8	**m^3**	0.3	**ac-ft**
V-10 year	371.5	**m^3**	0.3	**ac-ft**
V-100 year	557.5	**m^3**	0.5	**ac-ft**

Basin F

B	0.093	**sq km**		
A	0.138	**sq km**		
R-2 year	14.4	**mm**		
R-5 year	19.6	**mm**		
R-10 year	24.4	**mm**		
R-100 year	48.5	**mm**		
V-2 year	1453.6	**m^3**	1.2	**ac-ft**
V-5 year	1651.9	**m^3**	1.3	**ac-ft**
V-10 year	1827.3	**m^3**	1.5	**ac-ft**
V-100 year	2742.2	**m^3**	2.2	**ac-ft**

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Basin G

B	0.004	sq km		
A	0.016	sq km		
R-2 year	14.4	mm		
R-5 year	19.6	mm		
R-10 year	24.4	mm		
R-100 year	48.5	mm		
V-2 year	339.9	m^3	0.3	ac-ft
V-5 year	386.3	m^3	0.3	ac-ft
V-10 year	427.3	m^3	0.3	ac-ft
V-100 year	641.3	m^3	0.5	ac-ft

Basin H

B	0.101	sq km		
A	0.158	sq km		
R-2 year	14.4	mm		
R-5 year	19.6	mm		
R-10 year	24.4	mm		
R-100 year	48.5	mm		
V-2 year	1592.8	m^3	1.3	ac-ft
V-5 year	1810.1	m^3	1.5	ac-ft
V-10 year	2002.2	m^3	1.6	ac-ft
V-100 year	3004.7	m^3	2.4	ac-ft

Basin I

B	0.012	sq km		
A	0.065	sq km		
R-2 year	14.4	mm		
R-5 year	19.6	mm		
R-10 year	24.4	mm		
R-100 year	48.5	mm		
V-2 year	806.8	m^3	0.7	ac-ft
V-5 year	916.9	m^3	0.7	ac-ft
V-10 year	1014.2	m^3	0.8	ac-ft
V-100 year	1522.1	m^3	1.2	ac-ft

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Utilizing this method, we estimate the total volume of a potential post fire debris flow to be an average of approximately 4 times the volume of total debris flow volume that we assessed using the unit volume analysis (Table 4. Debris Flow Volume Comparison).

Basin	A	B	C	D	E	F	G	H	I
Volume Analysis (ac-ft)	0.05	0.06	0.63	0.04	0.11	1.23	2.74	0.36	0.5
*Post Fire Volume (ac-ft)	0.3	0.5	1.8	0.8	0.5	2.2	0.5	2.4	1.2

Table 4: Debris Flow Volume Comparison; *volume from 100 year storm event

8.0 GEOLOGIC HAZARDS SUMMARY AND CONCLUSIONS

It is the opinion of GeoStrata that alluvial fan flooding hazards that we assessed in the Francis burn scar study area do pose a hazard within, and down gradient from, the Francis burn scar area. Below is a summary of the alluvial fan flooding hazard and GeoStrata's recommendation for mitigation.

Based on the debris flow volumes calculated using the unit-volume analysis and post fire debris flow methods, we assess the alluvial fan flooding hazard at the mouths of the subject drainages to be high. Assuming that approximately one half to one third of an alluvial fan may be flooded in a single debris flow event, the CMT 2019 study may underestimate alluvial fan size for Basin F and Basin H and the GBES 2008 study may underestimate alluvial fan size for Basin C and Basin D. Two low spots created by manmade excavation of surficial soils currently exist in the northwest margin of the study area. These two low spot areas are shown on Plate A-9 and labeled as Debris Catchment Areas. These two excavated areas could potentially accommodate the collection of debris from Basin I, Basin H and Basin G during an alluvial fan flooding event from those drainage basins. It should be noted that these are not engineered detention basins and that currently unassessed runup velocities and basin volumes could allow the debris flow to overrun the potential collection areas rather than deposit sediment in these areas. It is therefore our opinion, that while these areas may provide some benefit, they should not be relied upon for mitigation of the alluvial fan flooding hazard from these three drainage basins. It is also important to note, that wherever these excavated areas are located on U.S. Forest Service land, the forest service will give no permission to improve or maintain these areas. Our understanding of the U.S. Forest Service's position on this issue is based on recent communication between the Forest Service and the City of Fruit Heights. As such, it is our opinion that these areas should not be relied upon for debris flow and alluvial fan flooding mitigation. GeoStrata recommends that debris flow fencing, berms and/or other structures be considered for installation along the base of the slope as a temporary mitigation method to collect debris and reduce the velocity of the debris flow and alluvial fan flooding. Upon request, GeoStrata may provide more details on recommendations for mitigation of the debris flow hazard that exists at the mouths of the subject drainages and on the alluvial fans associated with each of the drainages.

9.0 CLOSURE

9.1 LIMITATIONS

The conclusions and recommendations contained in this report, which include professional opinions and judgments, are based on the information available to us at the time of our evaluation, the results of our field observations and our understanding of the proposed site development. If any conditions are encountered at this site that are different from those described in this report, our firm should be immediately notified so that we may make any necessary revisions to recommendations contained in this report. In addition, if the scope of the proposed development changes from that described in this report, our firm should also be notified.

All services were completed in accordance with the current standard of care and generally accepted standard of practice at the time and in the place our services were completed. No other warranty, expressed or implied, is made. Development of property in the immediate vicinity of geologic hazards involves a certain level of inherent risk. It is impossible to predict where geologic hazards will occur. New geologic hazards may develop, and existing geologic hazards may expand beyond their current limits.

All services were performed for the exclusive use and benefit of the above addressee. No other person is entitled to rely on GeoStrata's services or use the information contained in this letter without the express written consent of GeoStrata. We are not responsible for the technical interpretations by others of the information described or documented in this report. The use of information contained in this report for bidding purposes should be done at the Contractor's option and risk.

10.0 REFERENCES CITED

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Appendix A



Legend

Approximate Burn Area

0 625 1,250 2,500 3,750 5,000 Feet

1 inch = 2,000 feet

Basemap:
2018 1 meter NAIP aerial imagery and hillshades derived from 2013-2014 0.5m LiDAR provided by the State of Utah AGRC.

N

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City of Fruit Heights
Fruit Heights, Utah
Project Number: 1058-008

Site Vicinity Map

Plate A-1



Legend

Approximate Burn Area

0 625 1,250 2,500 3,750 5,000 Feet

1 inch = 2,000 feet

Basemap:
Kaysville Quadrangle, Utah, 7.5-Minute Series (Topographic), USGS 1998 and hillshades derived from 2013-2014 0.5m LiDAR provided by the State of Utah AGRC.

N

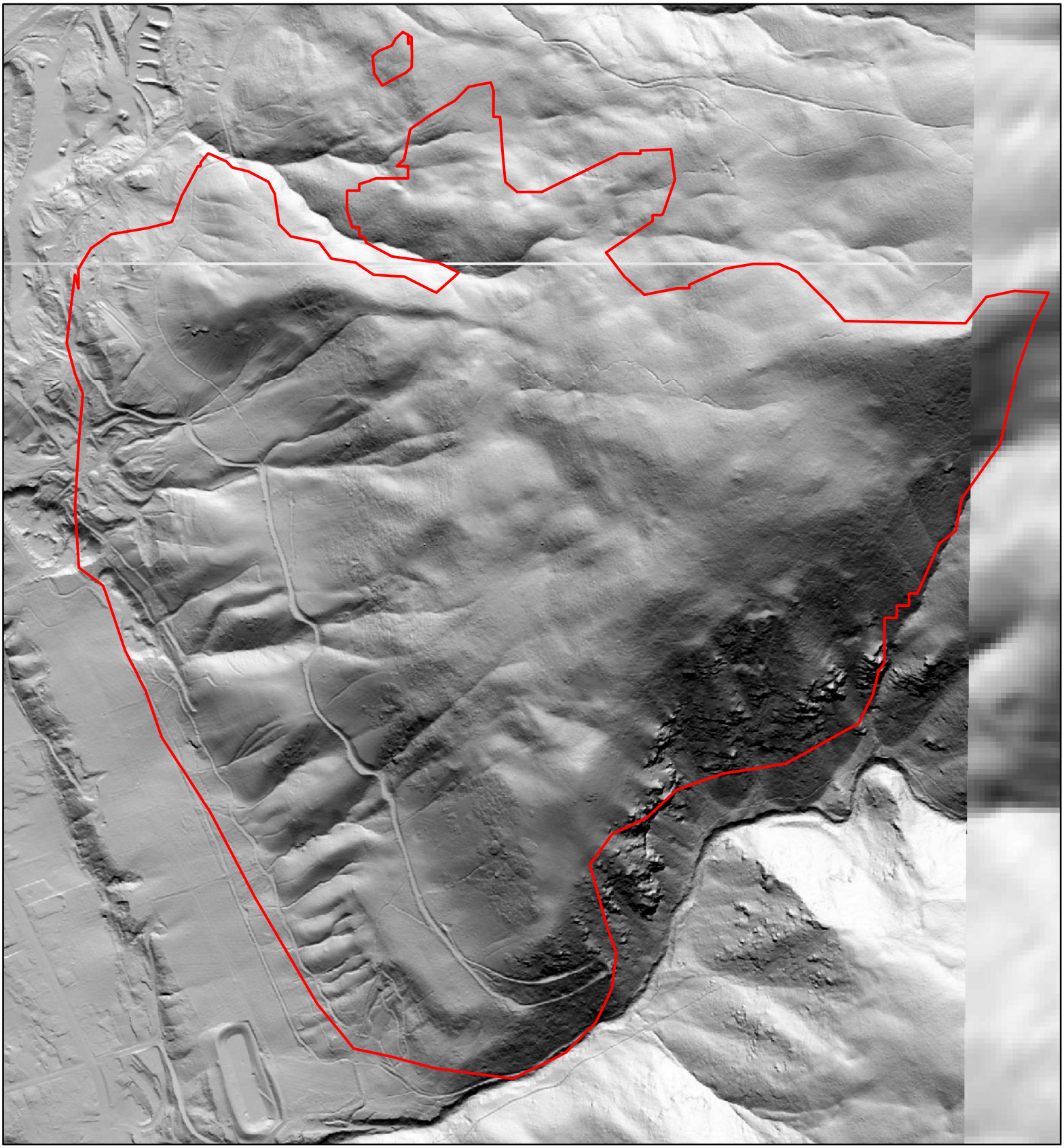
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Topographic Map

Plate A-2



Legend

Approximate Burn Area

0 220 440 880 1,320 1,760 Feet

1 inch = 700 feet

Basemap:
Hillshades derived from 2013-2014 0.5m LiDAR provided by the State of Utah AGRC.

N

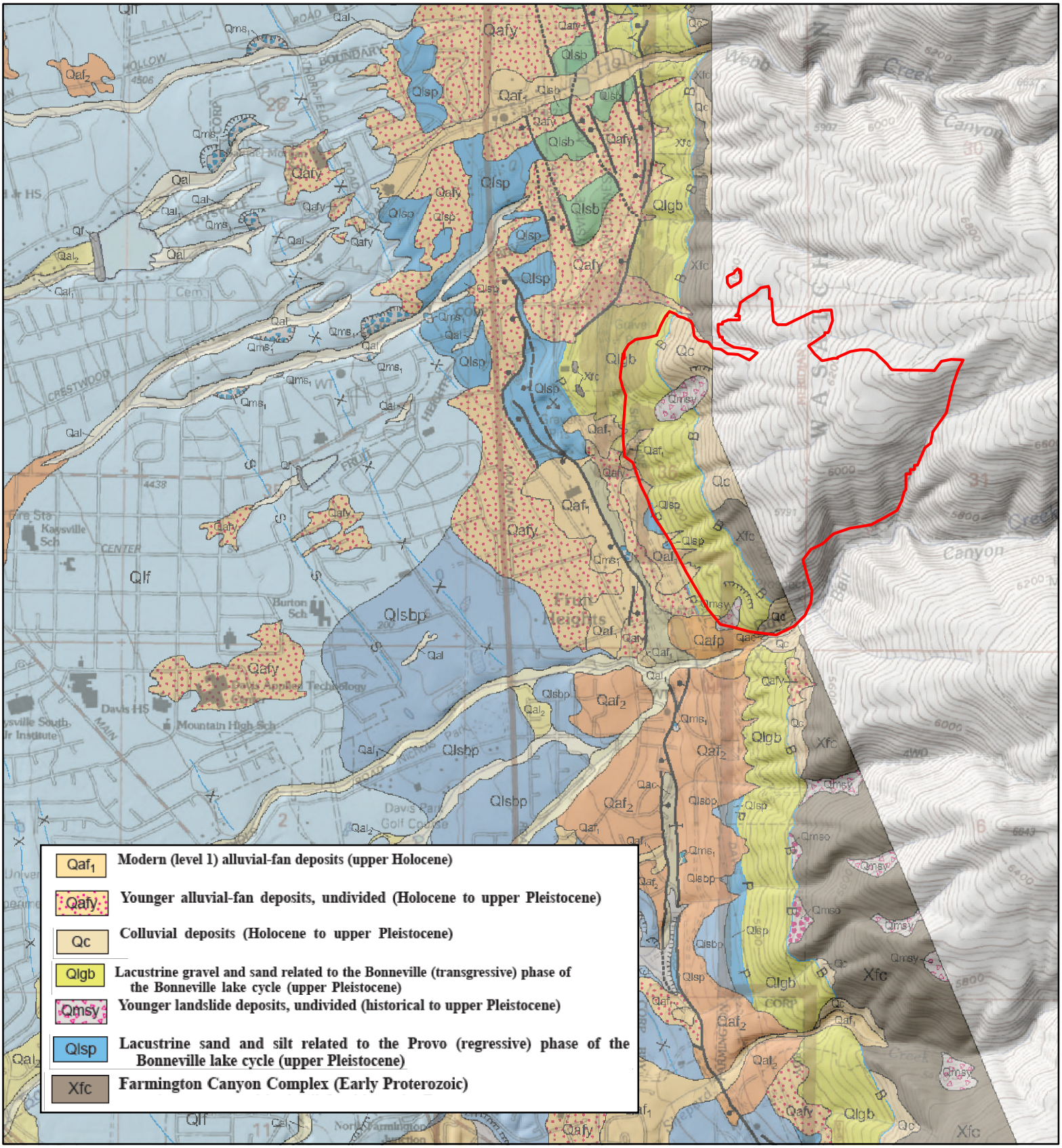
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Hillshade Map

Plate A-3



Legend

Approximate Burn Area

0 625 1,250 2,500 3,750 5,000 Feet

1 inch = 2,000 feet

N

Basemap:
Surficial Geologic Map of Part of the Kaysville Quadrangle, Solomon (2007). Hillshades derived from 2013-2014 0.5m LiDAR provided by the State of Utah AGRC.

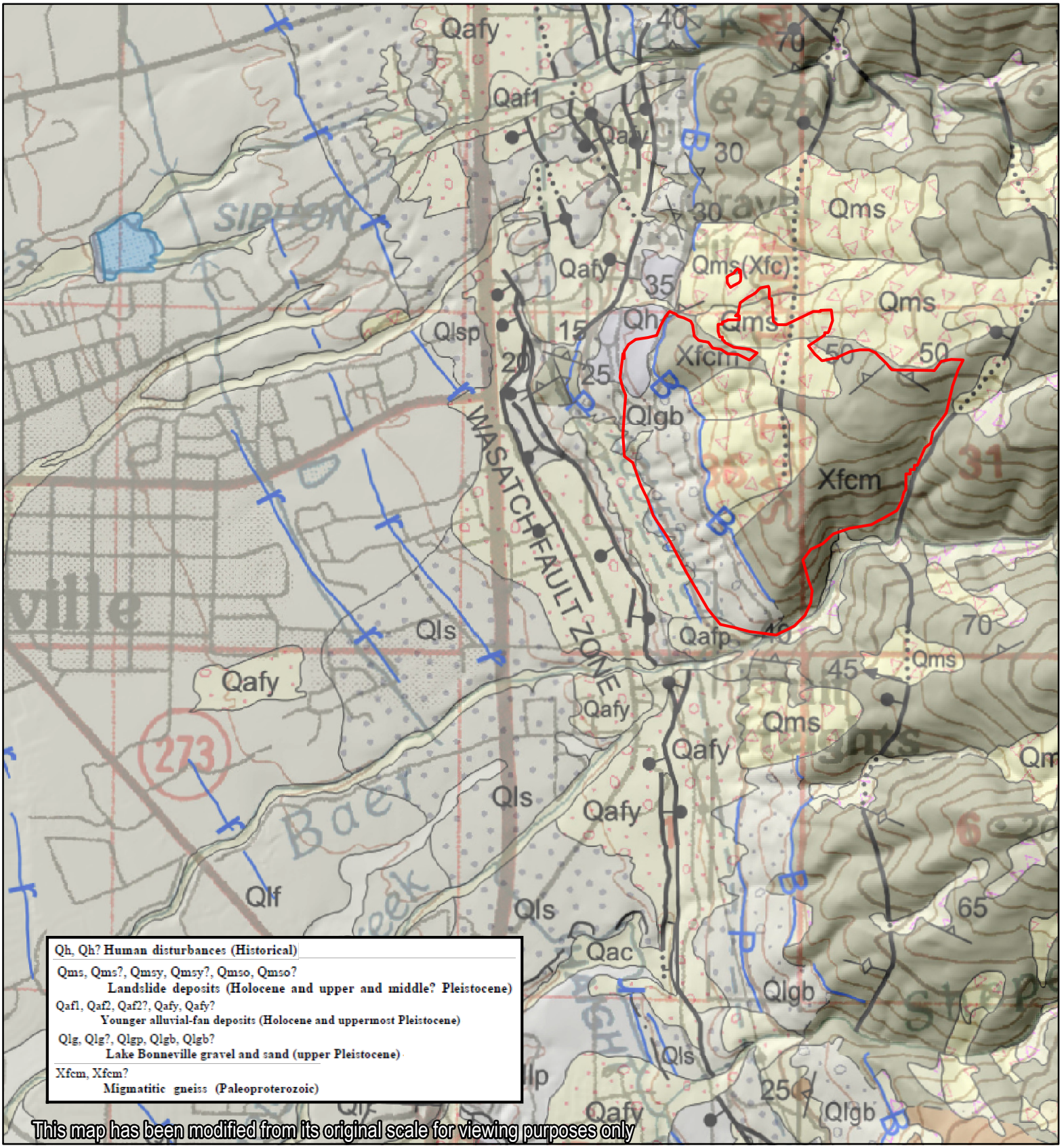
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Site Geologic Map

Plate A-4



Legend

Approximate Burn Area

Feet
0 625 1,250 2,500 3,750 5,000
1 inch = 2,000 feet

N

Basemap:
Surficial Geologic Map of Part of the Kaysville Quadrangle, Solomon (2007). Hillshades derived from 2013-2014 0.5m LiDAR provided by the State of Utah AGRC.

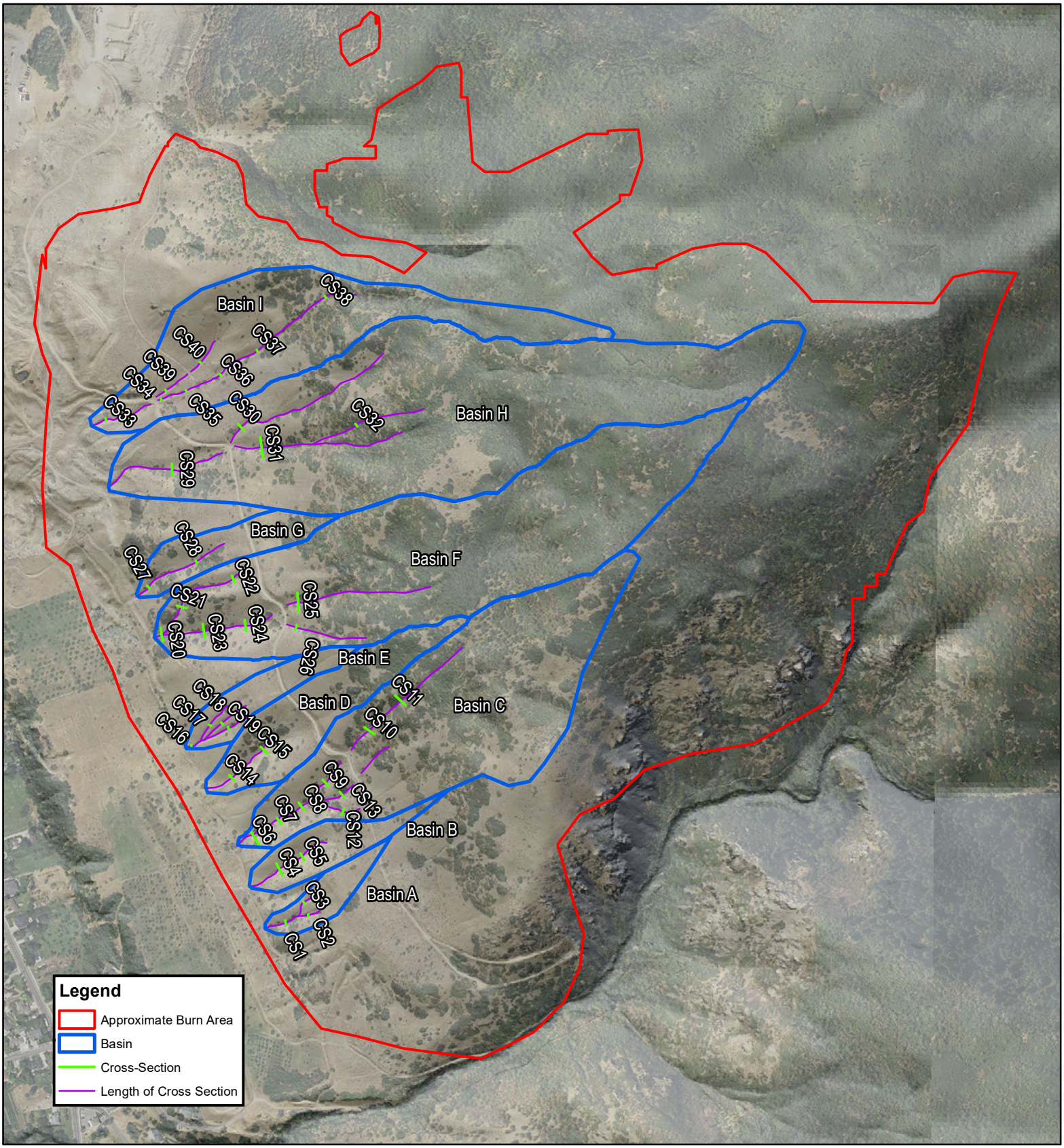
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Project Number: 1058-008

Site 30x60 Geologic Map

Plate A-5



0 235 470 940 1,410 1,880 Feet

1 inch = 700 feet

Basemap:
2018 1 meter NAIP aerial imagery and hillshades derived from 2013-2014 0.5 meter lidar and DEMs provided by the State of Utah AGRC.

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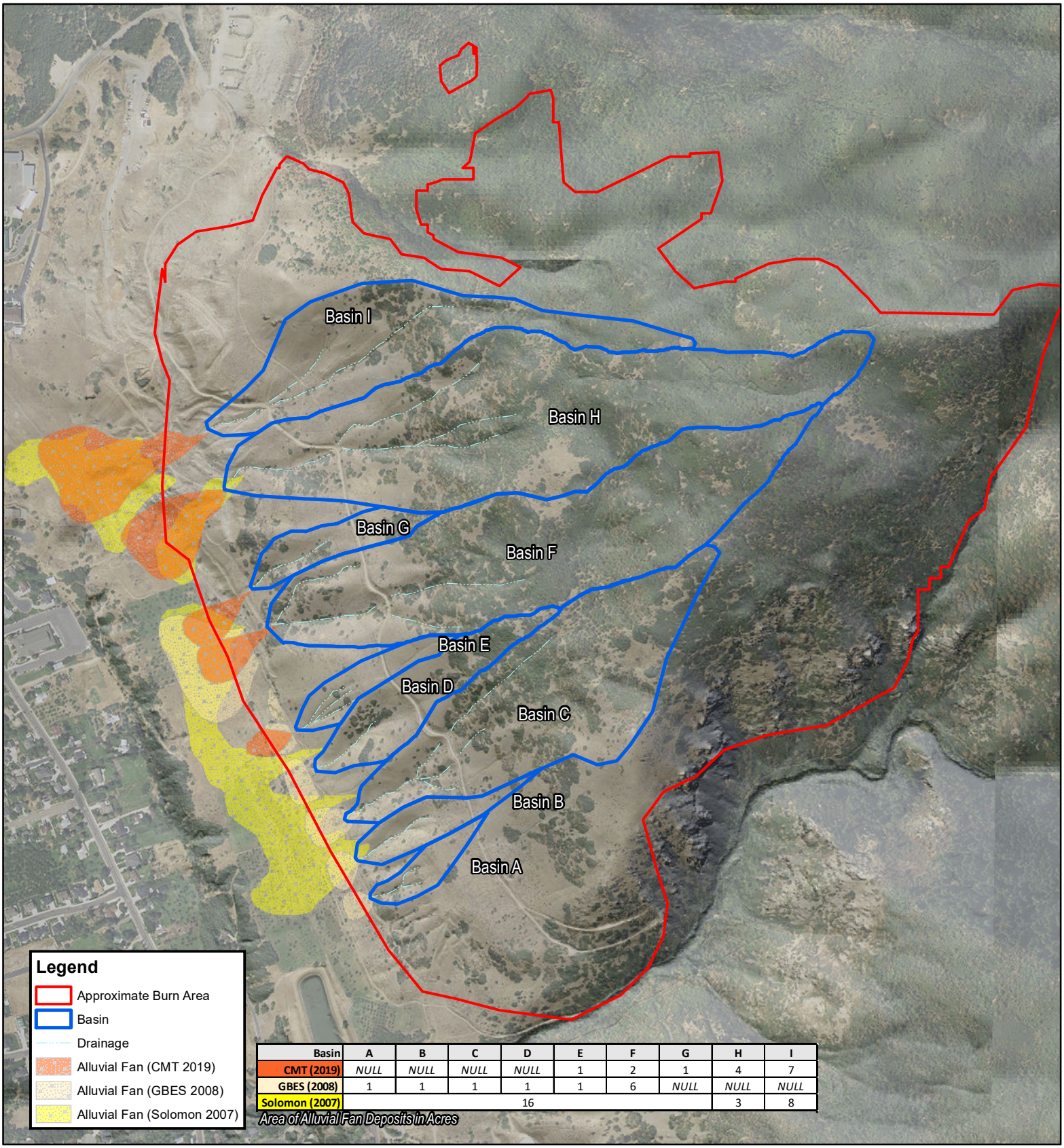
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Debris Flow Assessment Map

Plate A-6



Basin	A	B	C	D	E	F	G	H	I
CMT (2019)	*NULL*	*NULL*	*NULL*	*NULL*	1	2	1	4	7
GBES (2008)	1	1	1	1	1	6	*NULL*	*NULL*	*NULL*
Solomon (2007)	16							3	8

Area of Alluvial Fan Deposits in Acres

0 250 500 1,000 1,500 2,000 Feet

1 inch = 750 feet

Basemap:
2018 1 meter NAIP aerial imagery and hillshades derived from 2013-2014 0.5 meter lidar and DEMs provided by the State of Utah AGRC.

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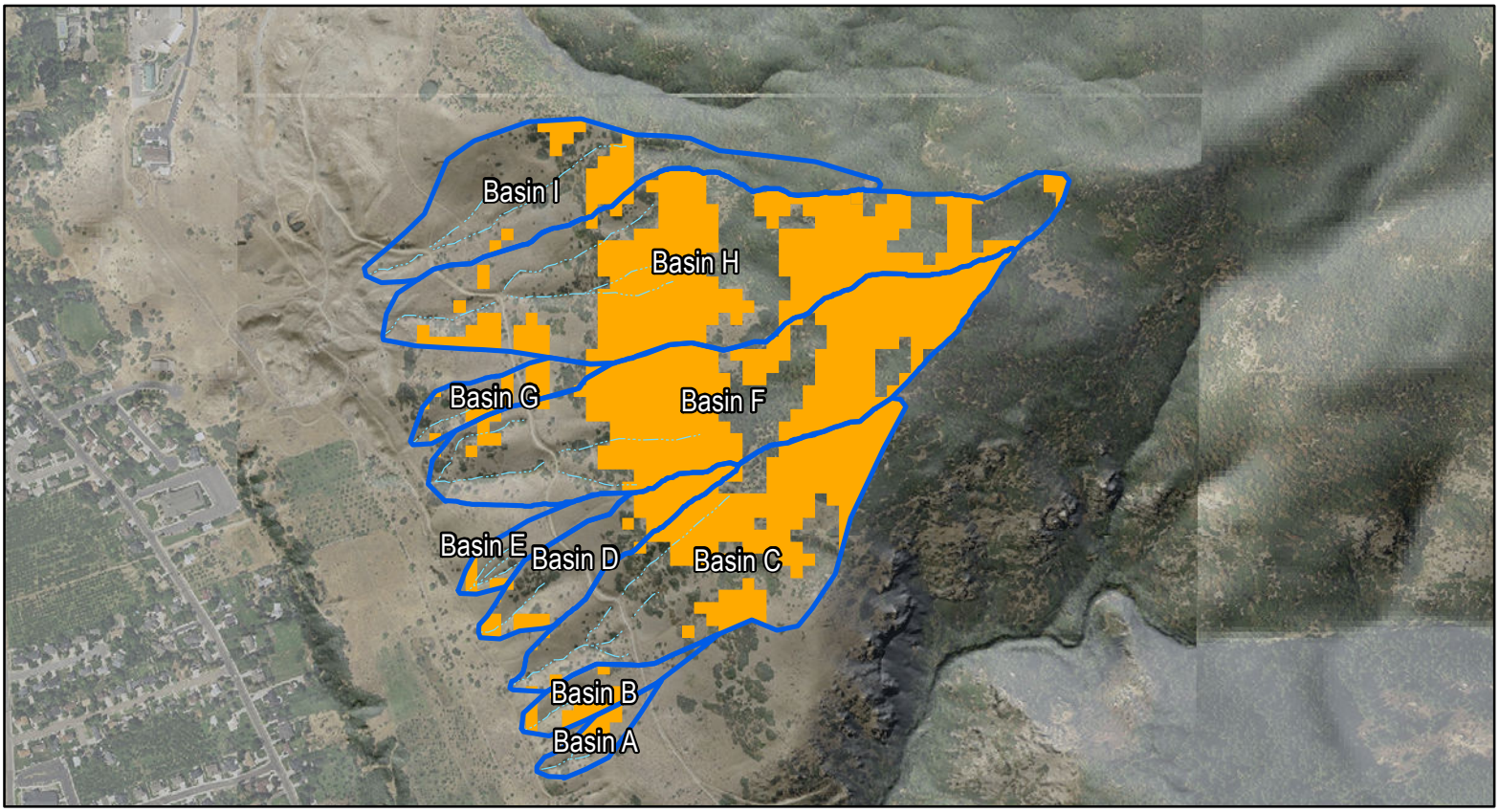
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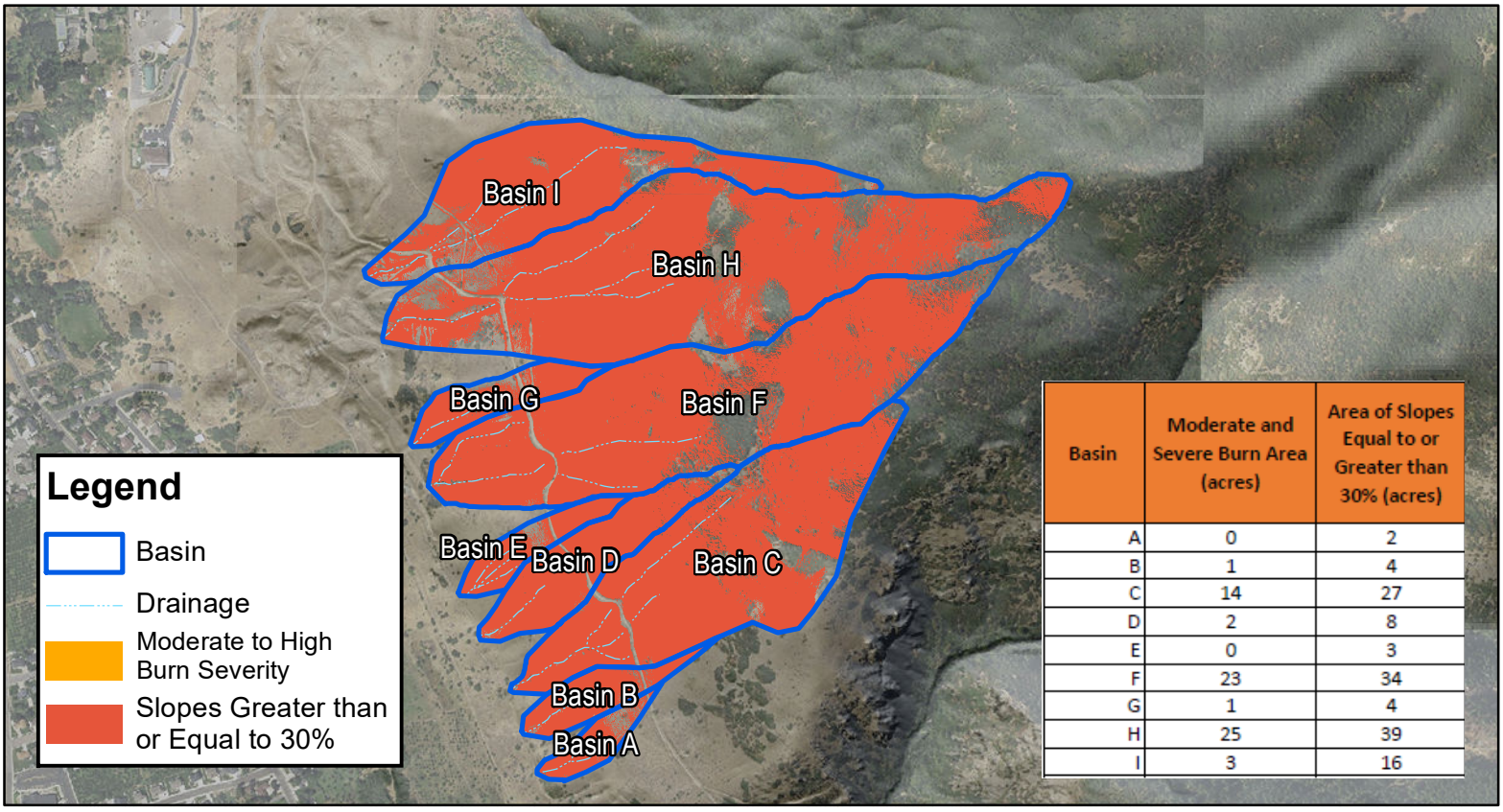
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Extent of Alluvial Fan

Plate A-7





Basin	Moderate and Severe Burn Area (acres)	Area of Slopes Equal to or Greater than 30% (acres)
A	0	2
B	1	4
C	14	27
D	2	8
E	0	3
F	23	34
G	1	4
H	25	39
I	3	16

0 312.5 625 1,250 1,875 2,500 Feet

1 inch = 1,000 feet

Basemap:
2018 1 meter NAIP aerial imagery and hillshades derived from 2013-2014 0.5 meter lidar and DEMs provided by the State of Utah AGRC.

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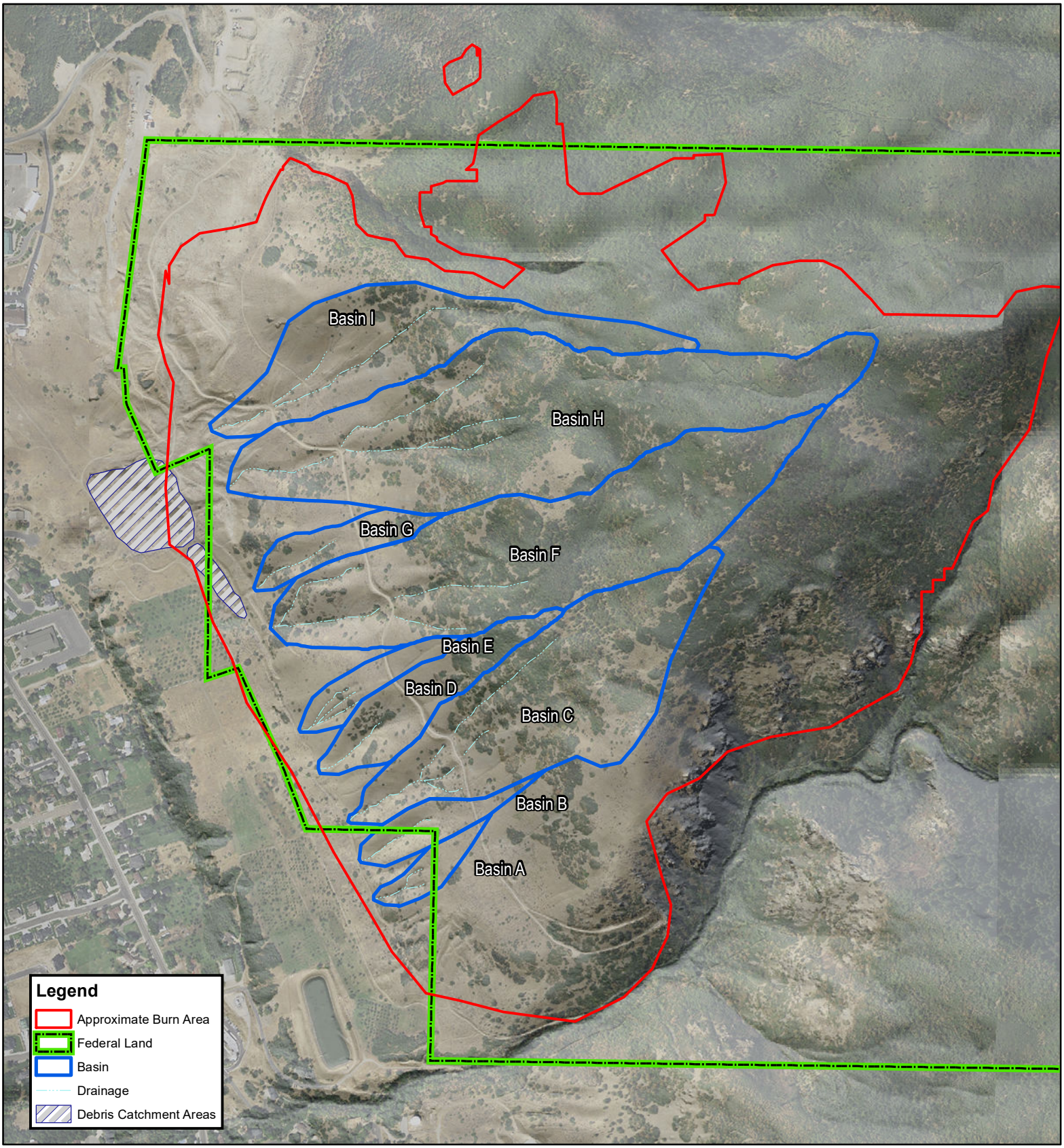
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Basin Characteristics

Plate A-8



0 250 500 1,000 1,500 2,000 Feet

1 inch = 750 feet

Basemap:
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Debris Catchment Areas

Plate A-9



Plate B-1

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Drainage B

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Plate B-2

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Drainage C

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Plate B-3

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Drainage C

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Plate B-4

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Drainage C

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Plate B-5

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Drainage D

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Plate B-6

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Drainage F

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Plate B-7

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Drainage G

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Plate B-8

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Drainage G

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Plate B-9

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Drainage H

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Plate B-10

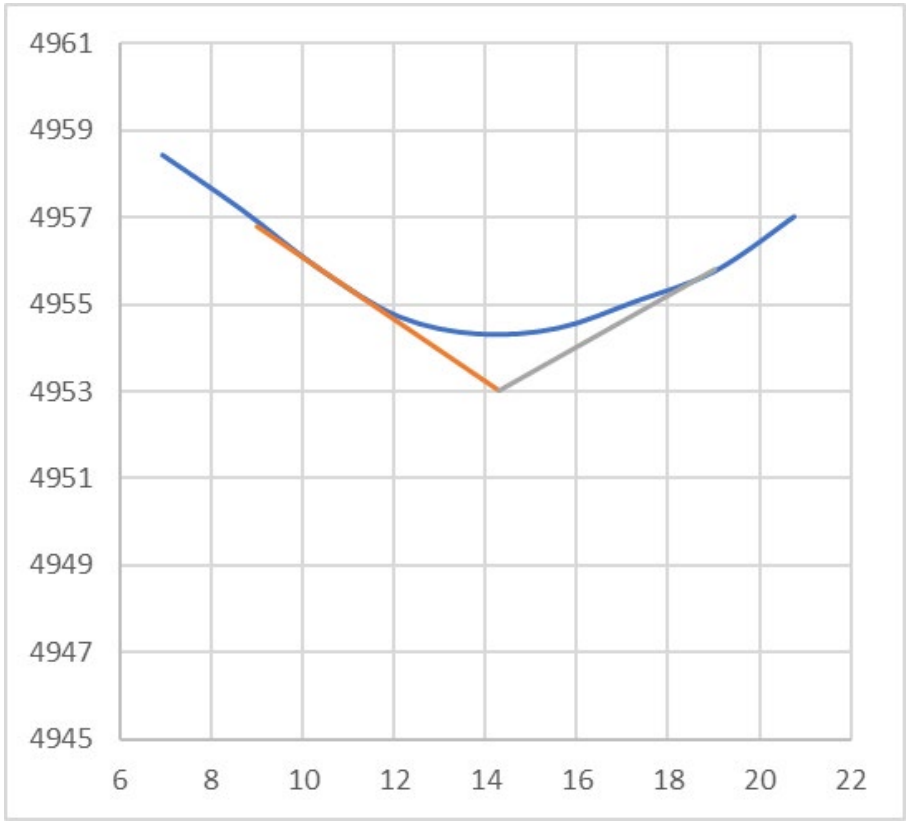
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City of Fruit Heights
Fruit Heights, Utah
Project Number: 1058-008

Drainage H

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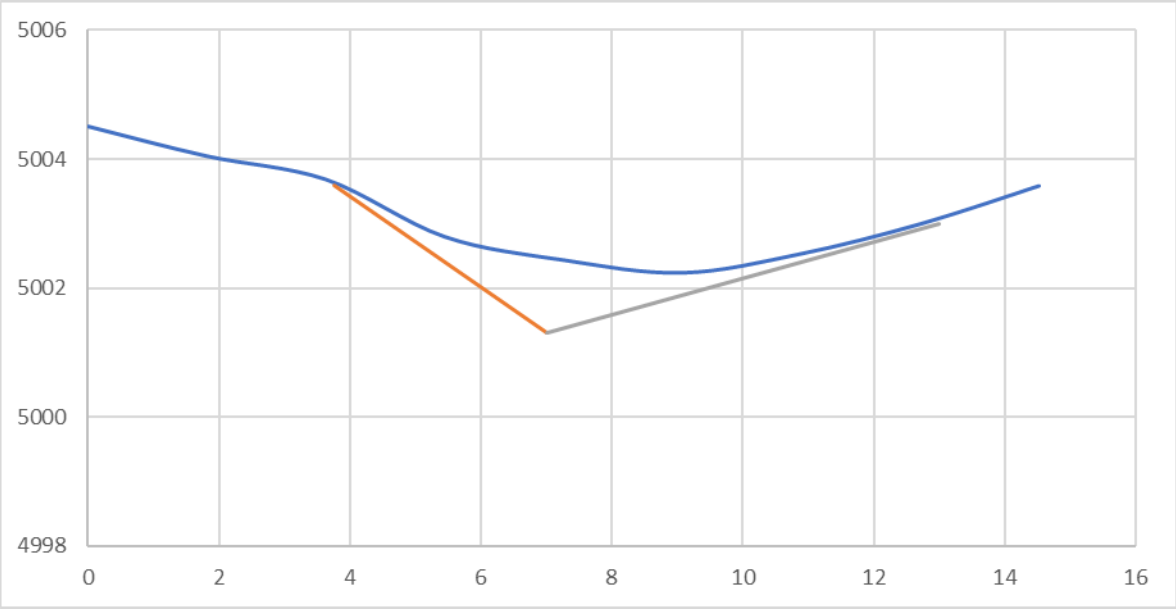
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Appendix C



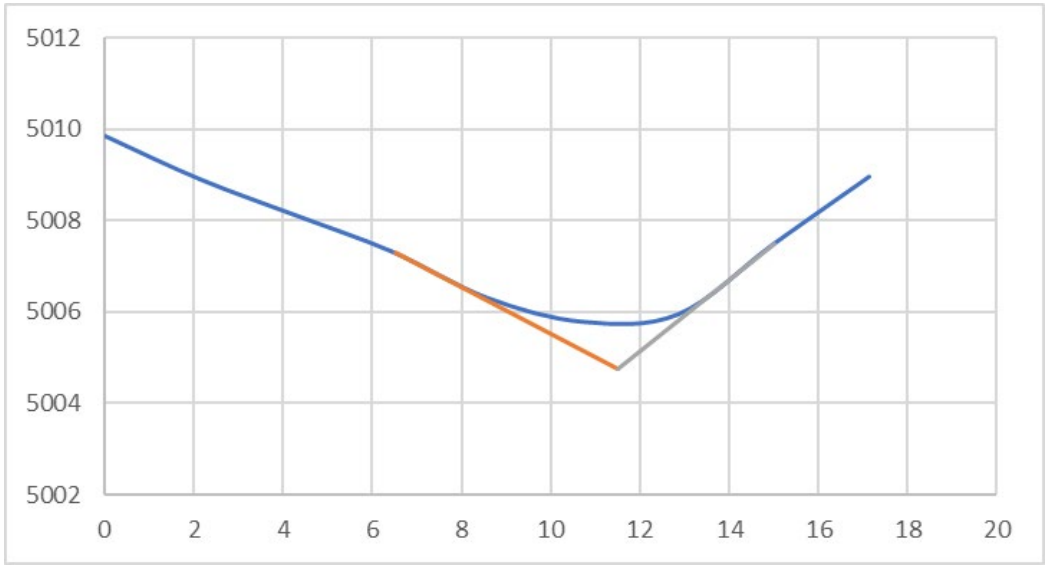
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45.23	3.77

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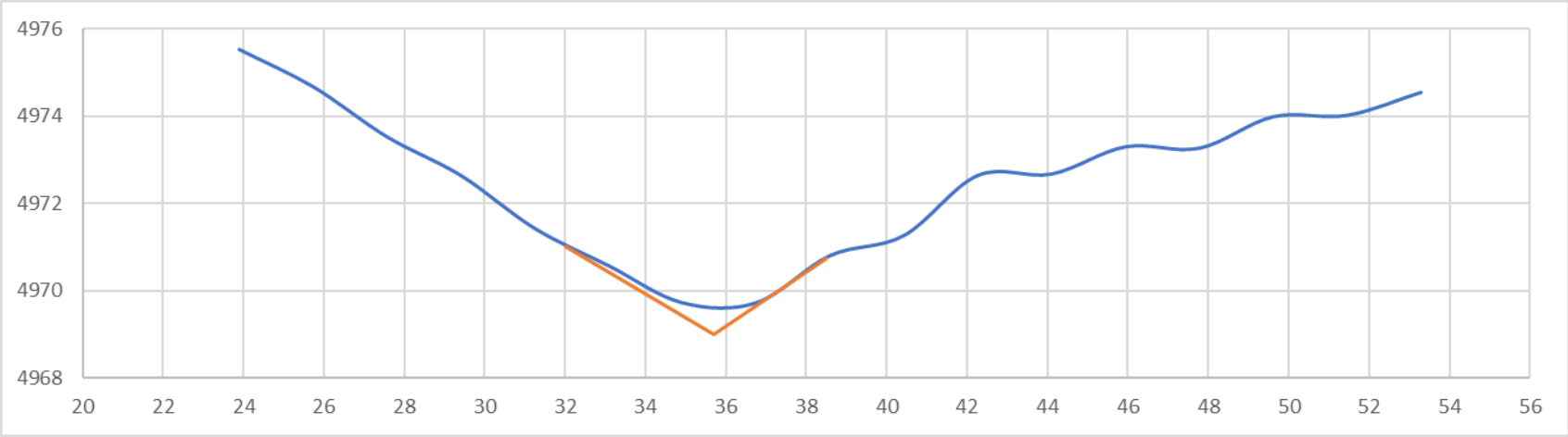
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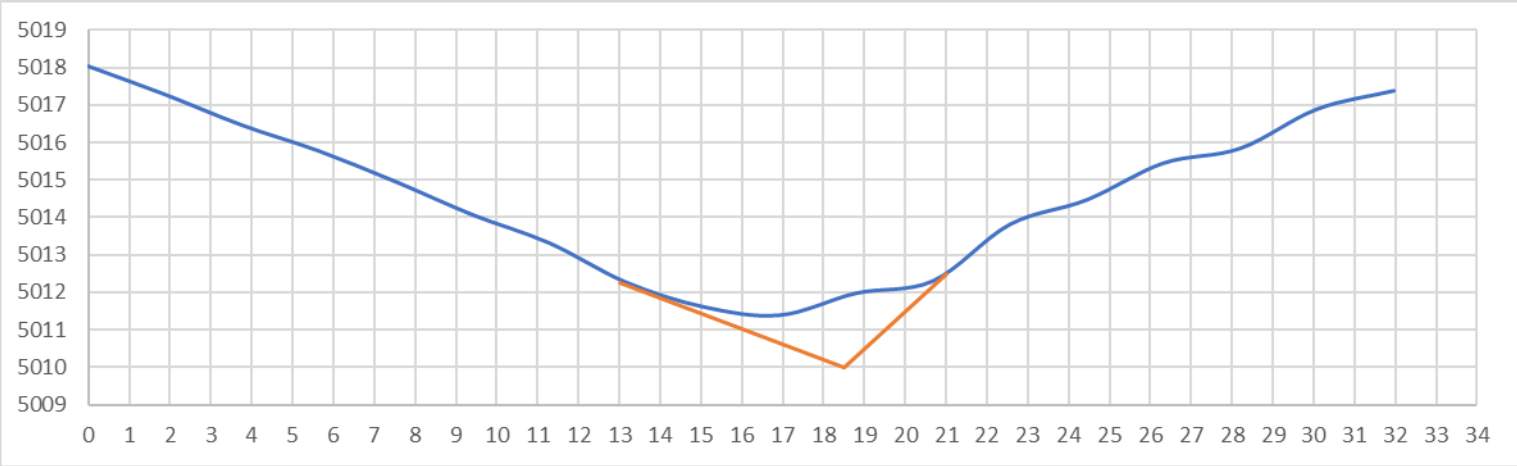
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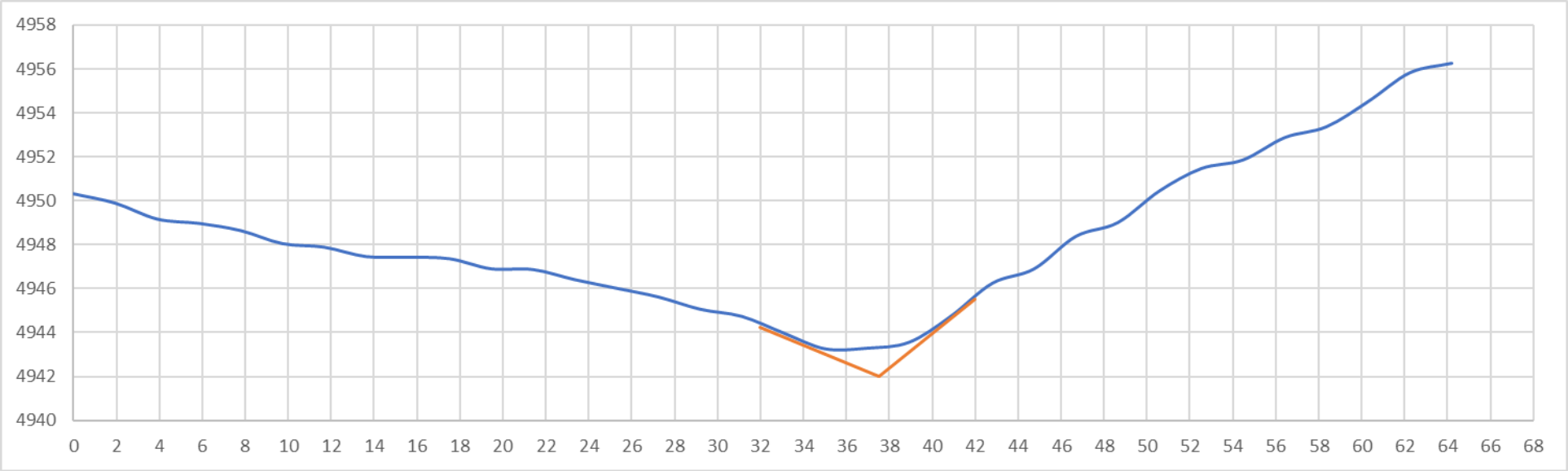
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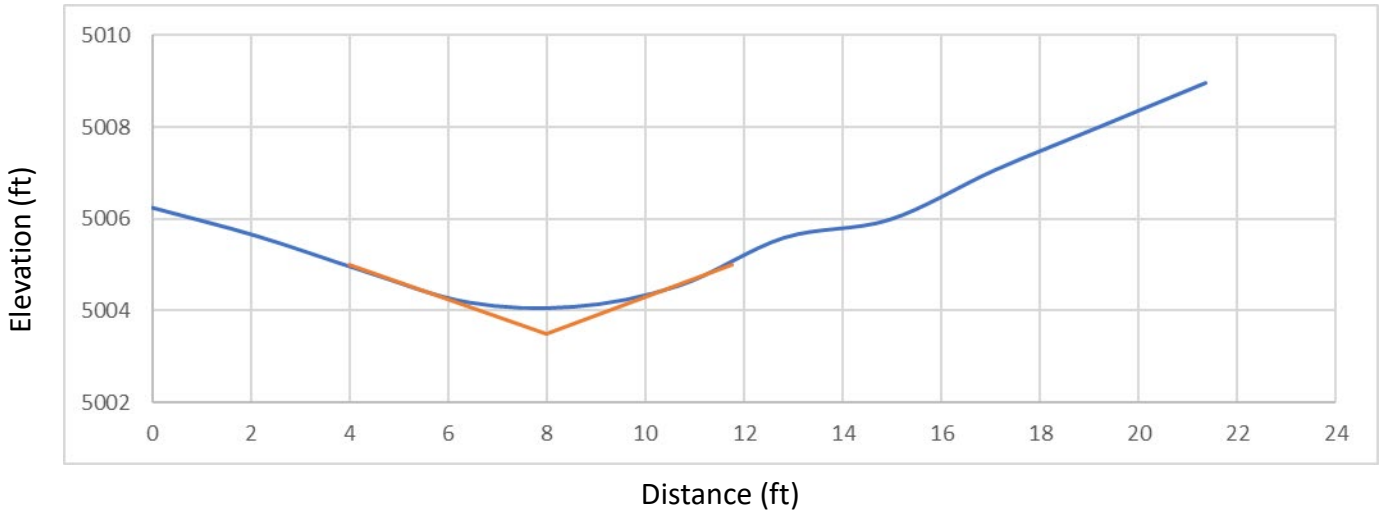
Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
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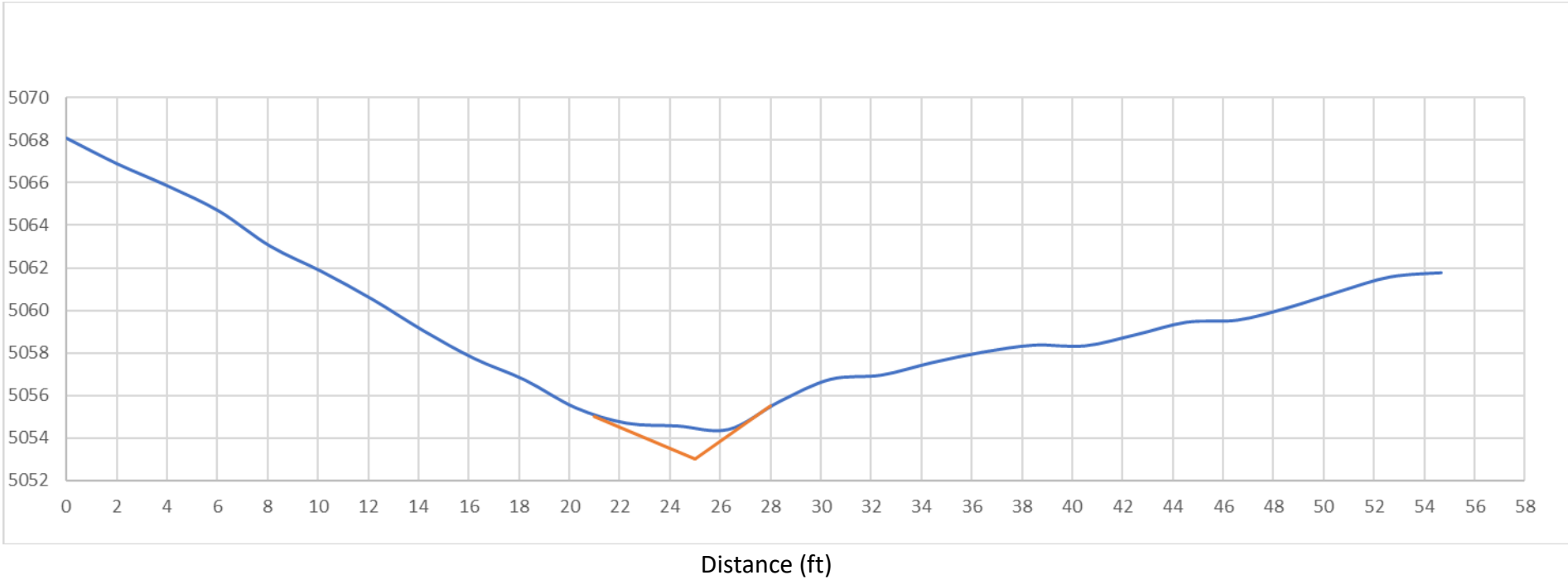
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43.53	3.63

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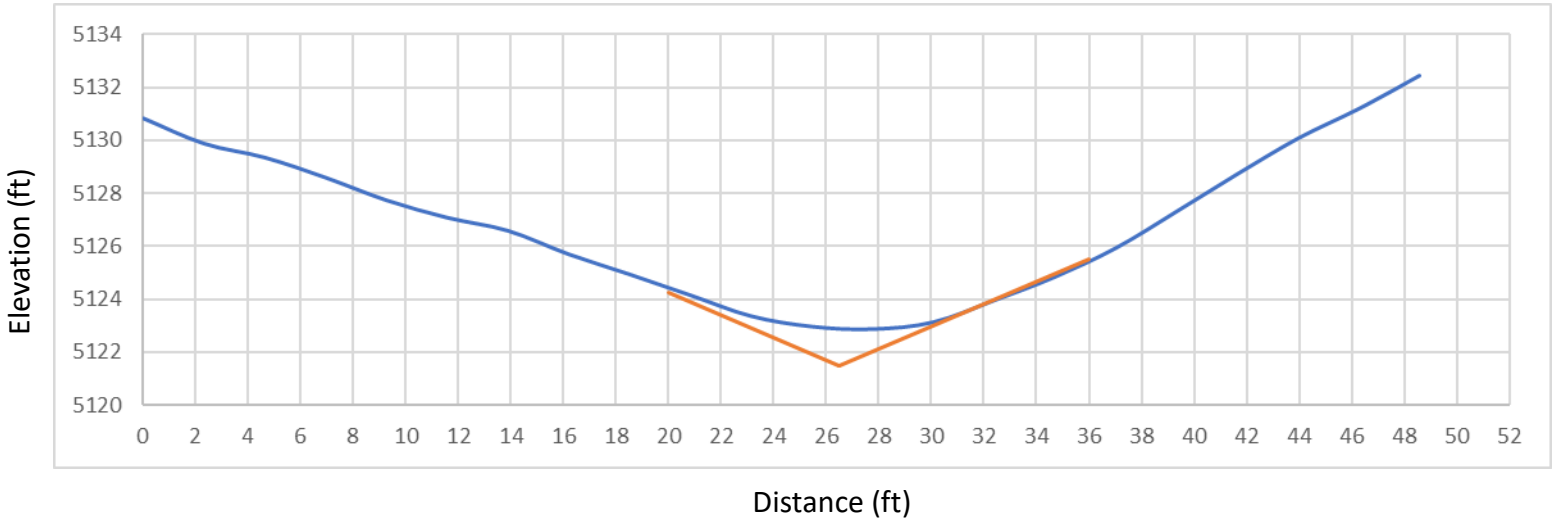
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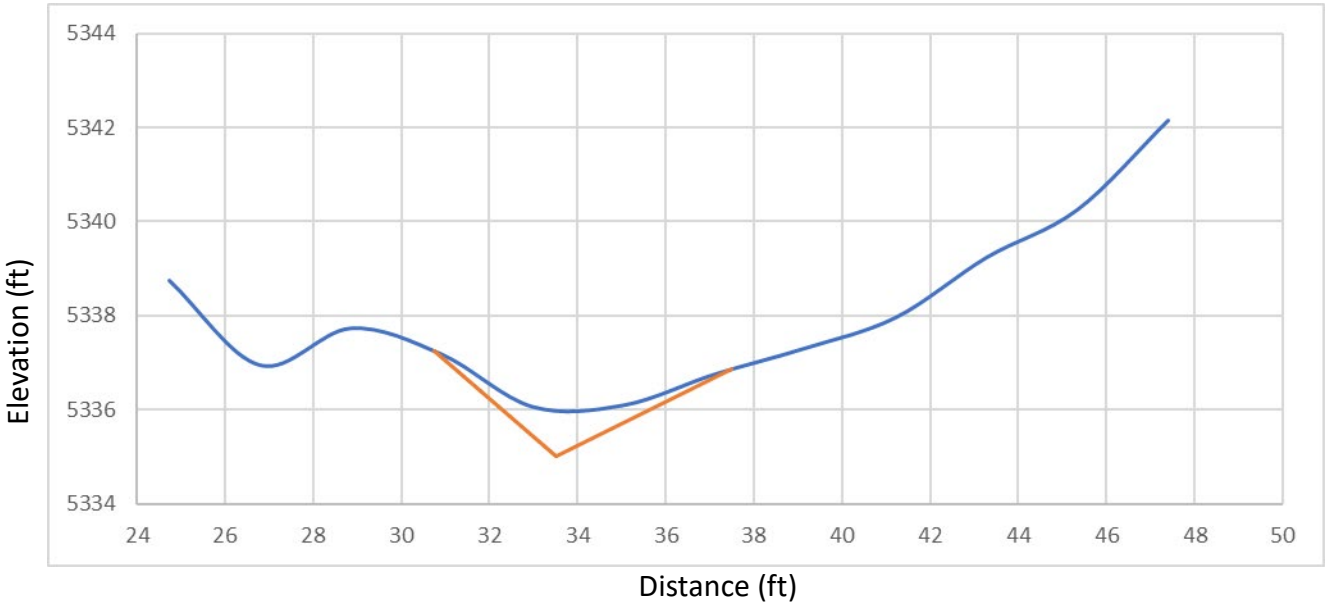
Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
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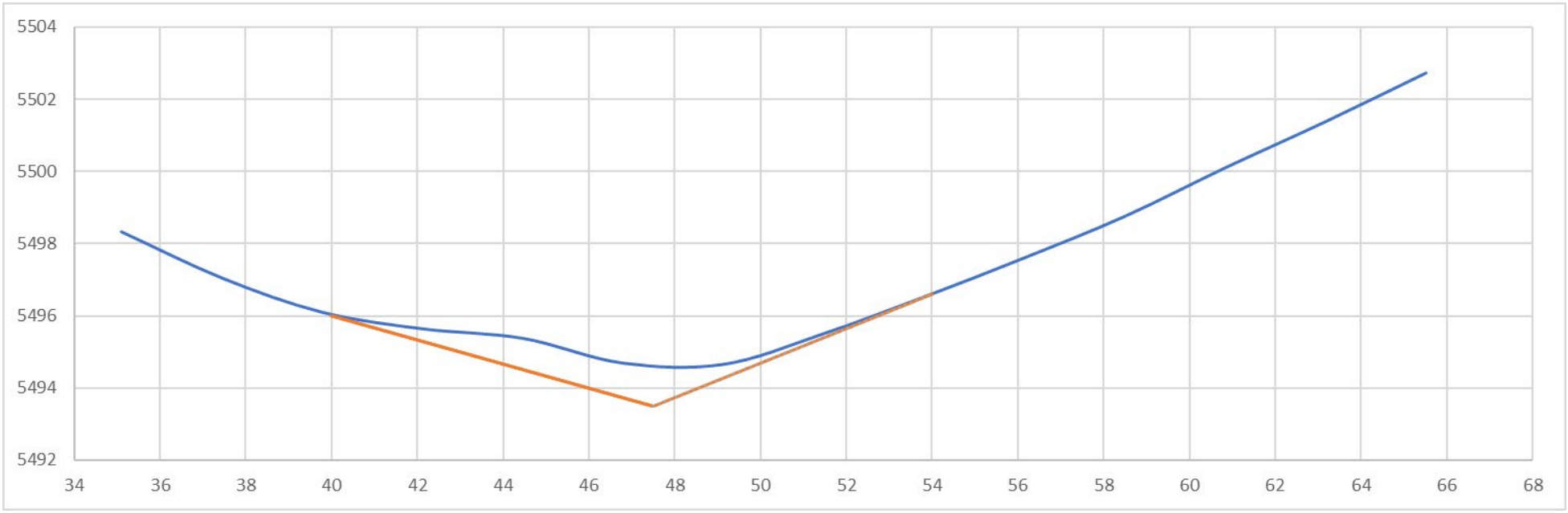
Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
54.71	4.56

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Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
26.98	2.25

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Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
40.08	3.34

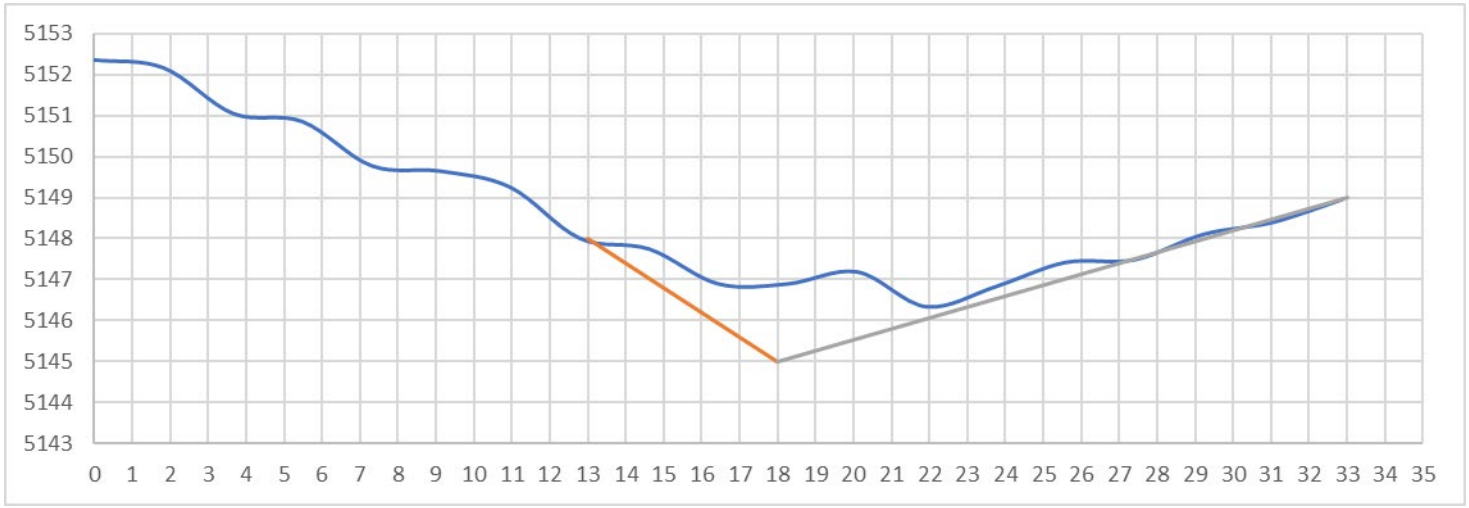
Plate C-11

Francis Fire Burn Scar Debris Flow Assessment
City of Fruit Heights
Fruit Heights, Utah
Project Number –1058-008

Cross-Section 11

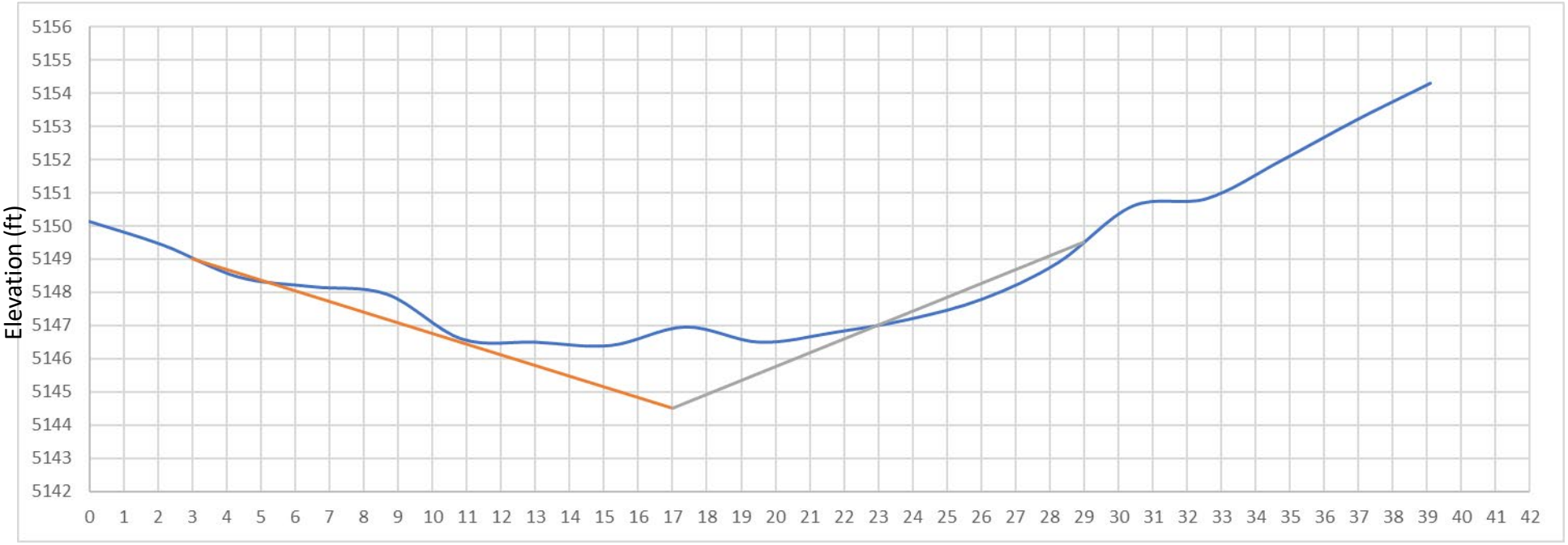
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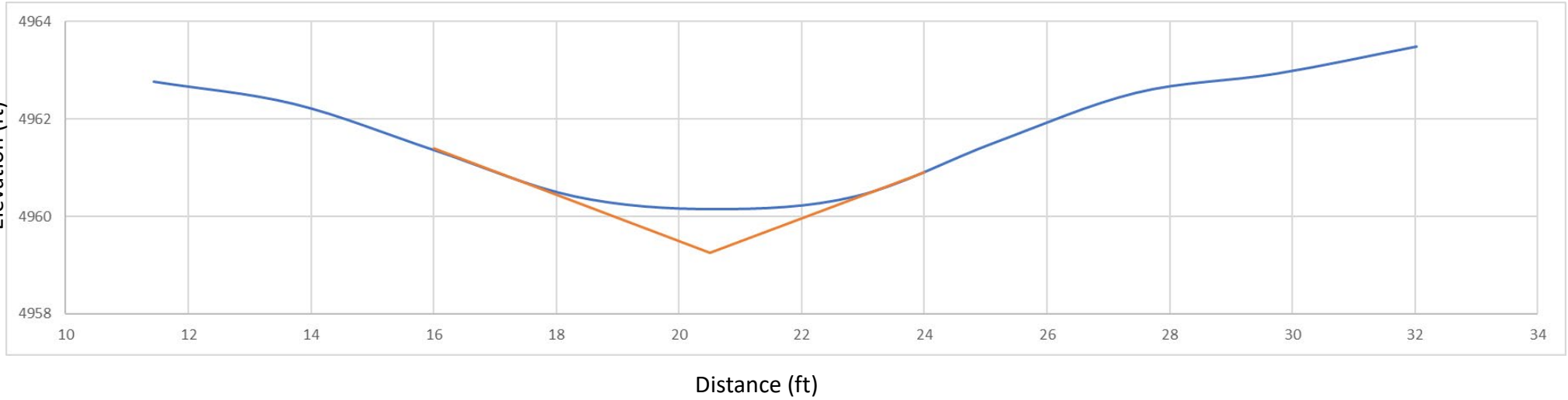
Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
132.82	11.07

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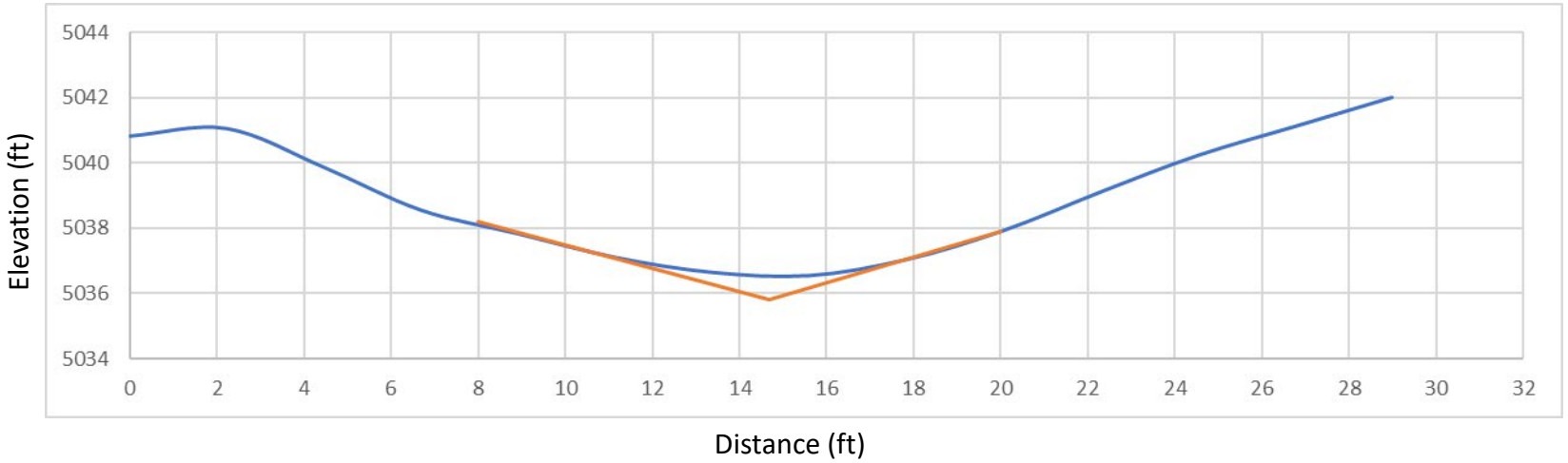
Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
160.06	13.34

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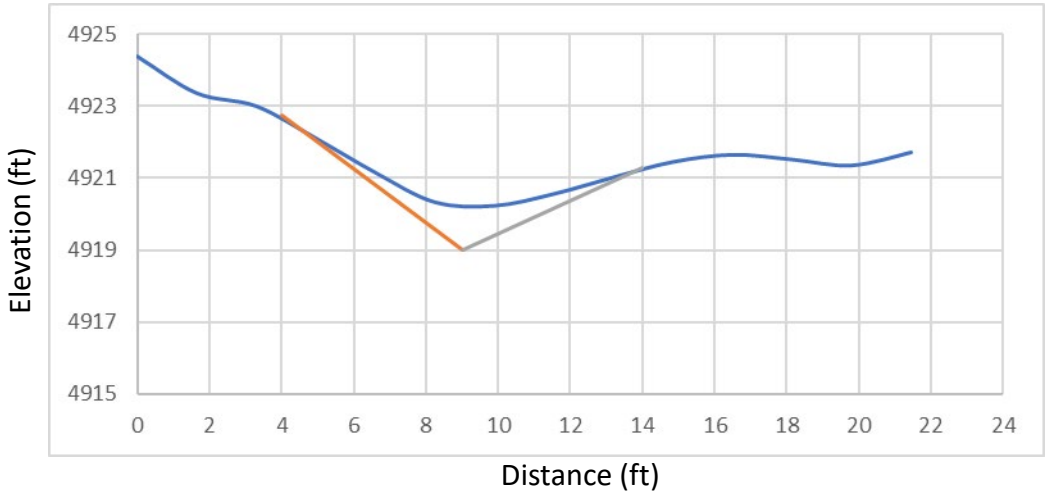
Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
19.88	1.66

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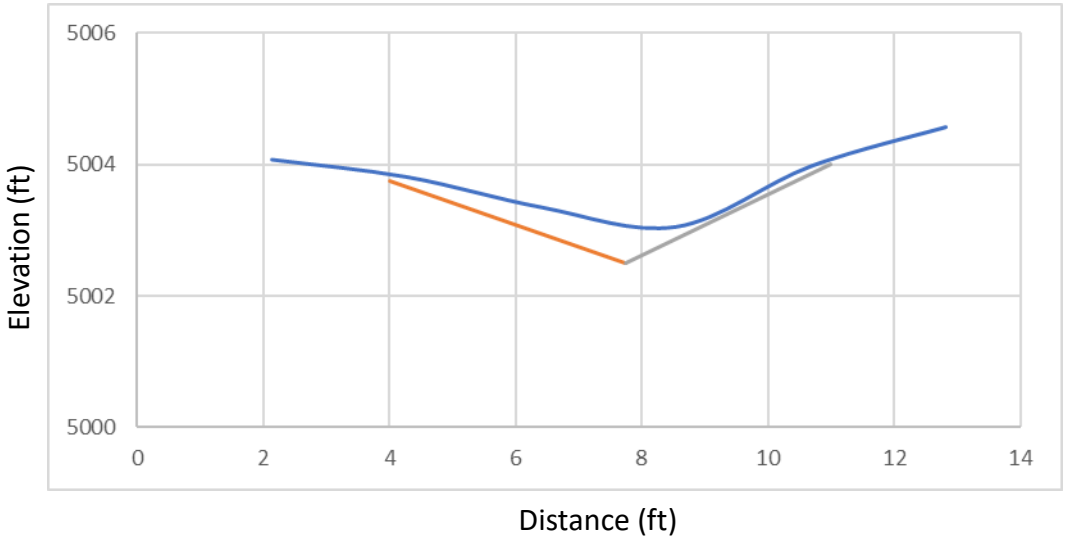
Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
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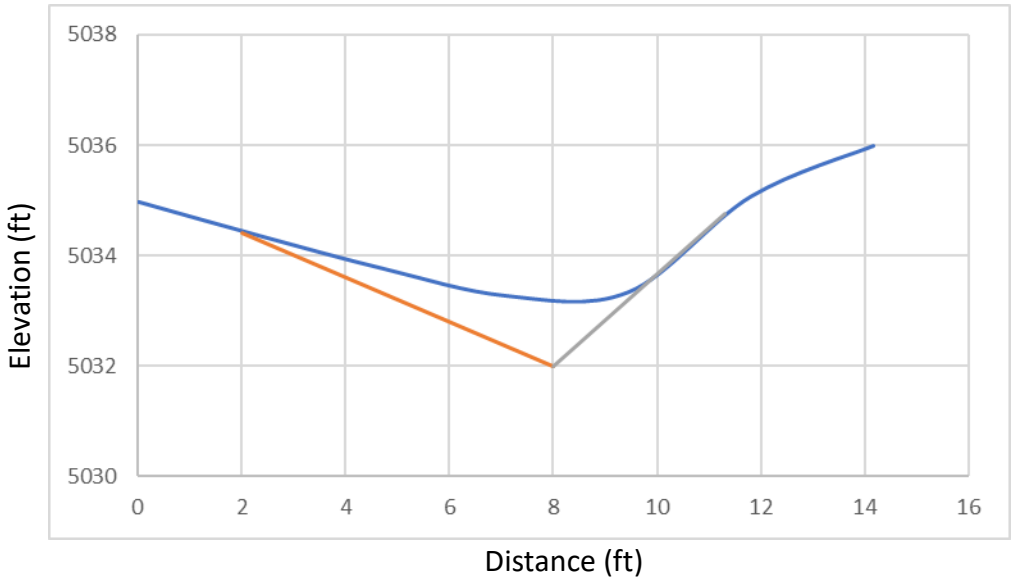
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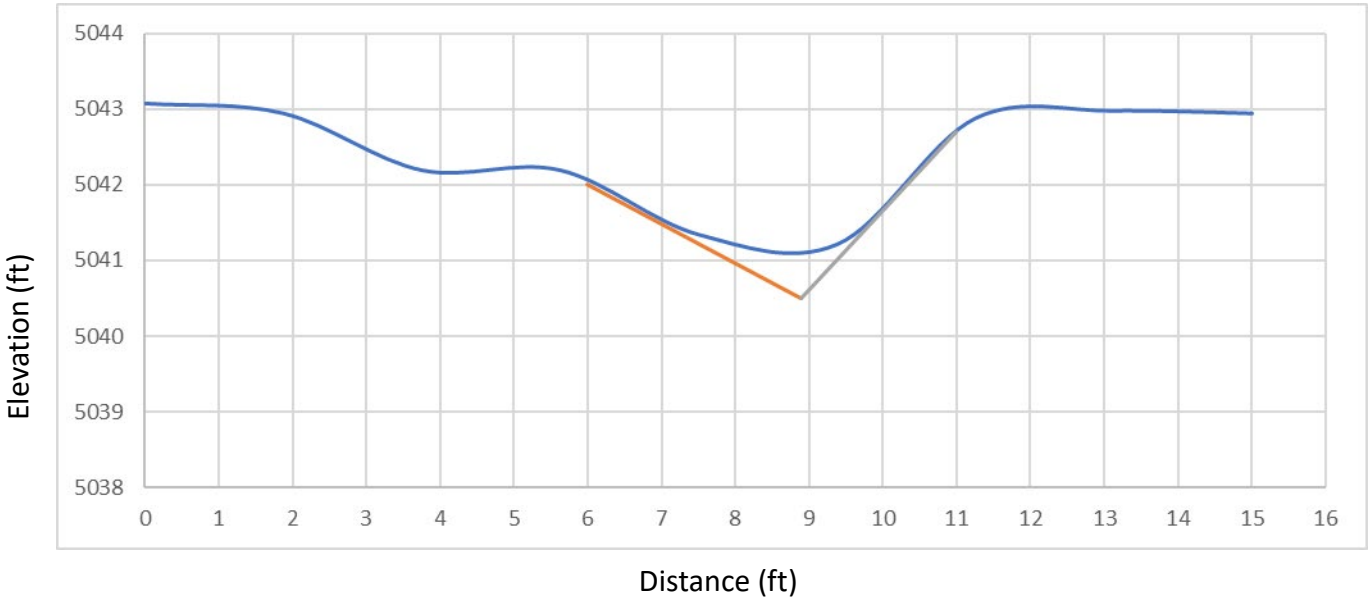
Area of Erodible Sediment (in2)	Area of Erodible Sediment (ft2)
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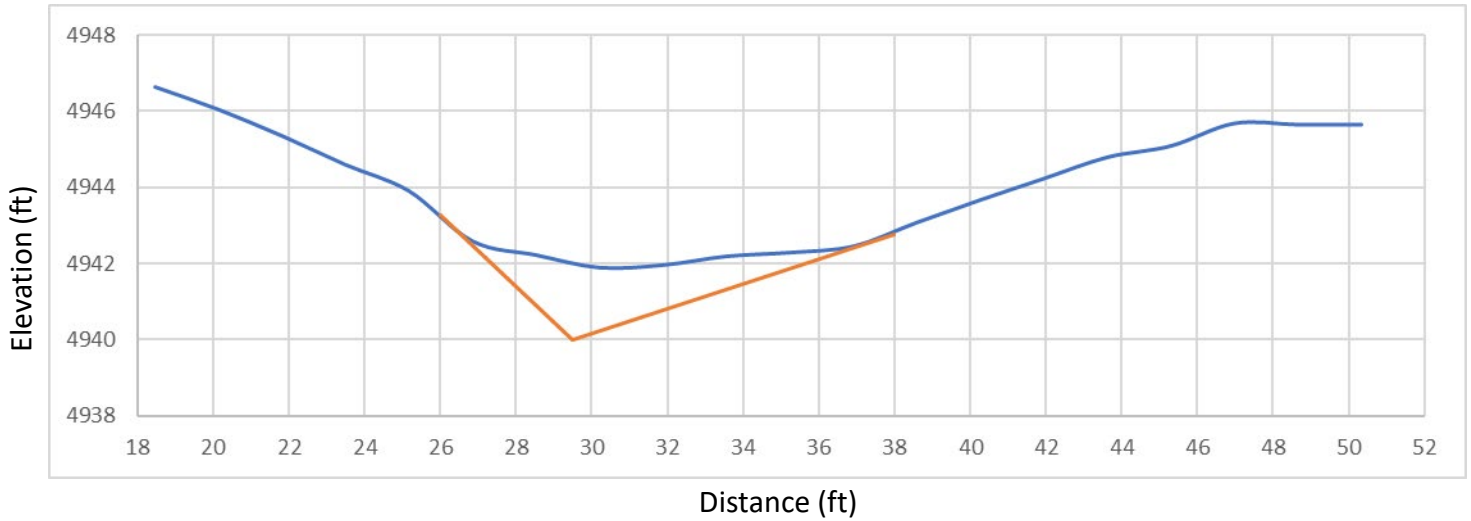
Area of Erodible Sediment (in2)	Area of Erodible Sediment (ft2)
45.11	3.76

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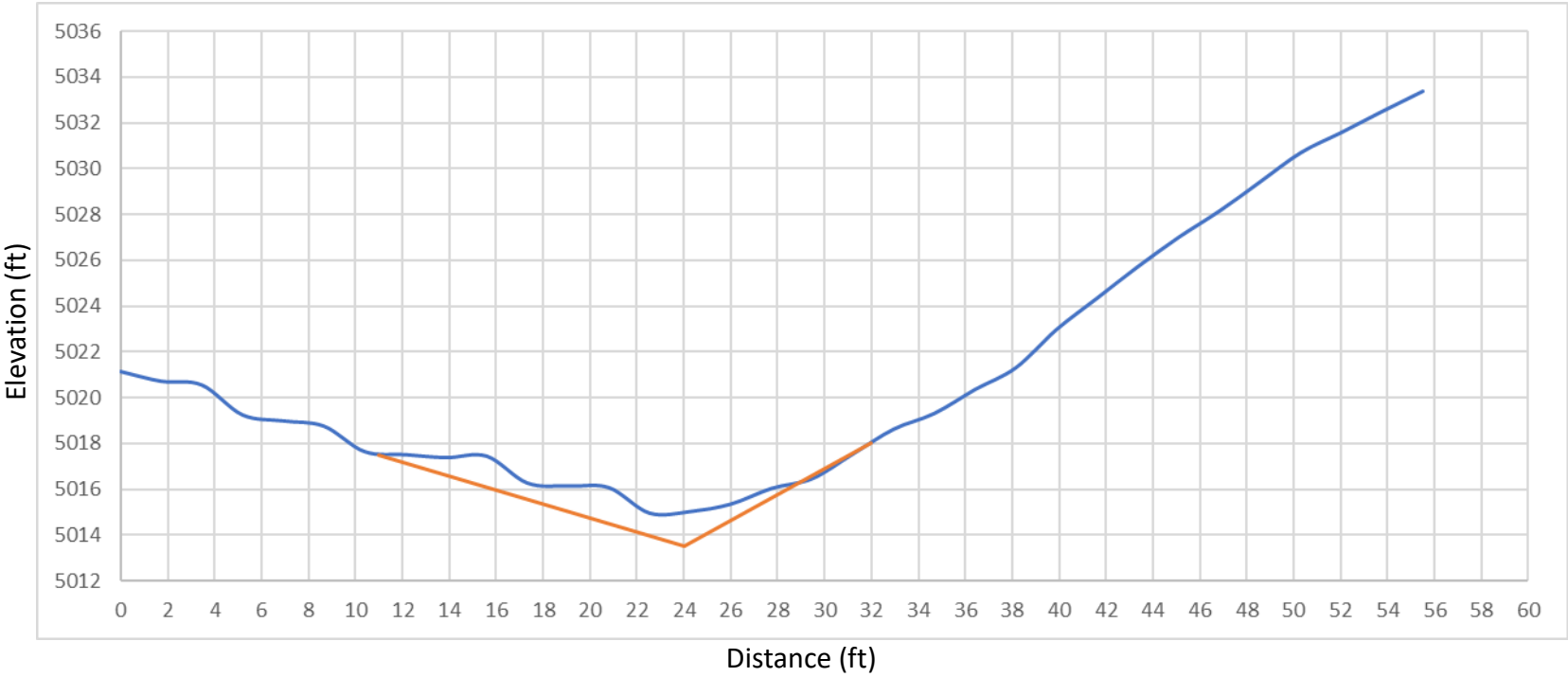
Area of Erodible Sediment (in2)	Area of Erodible Sediment (ft2)
7.48	0.62

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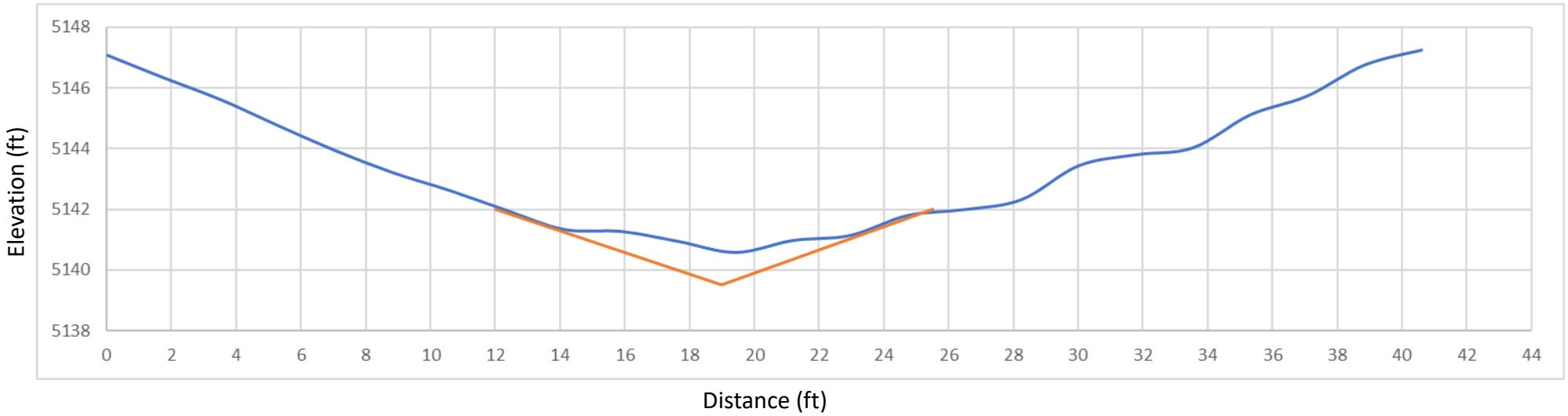
Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
154.50	12.87

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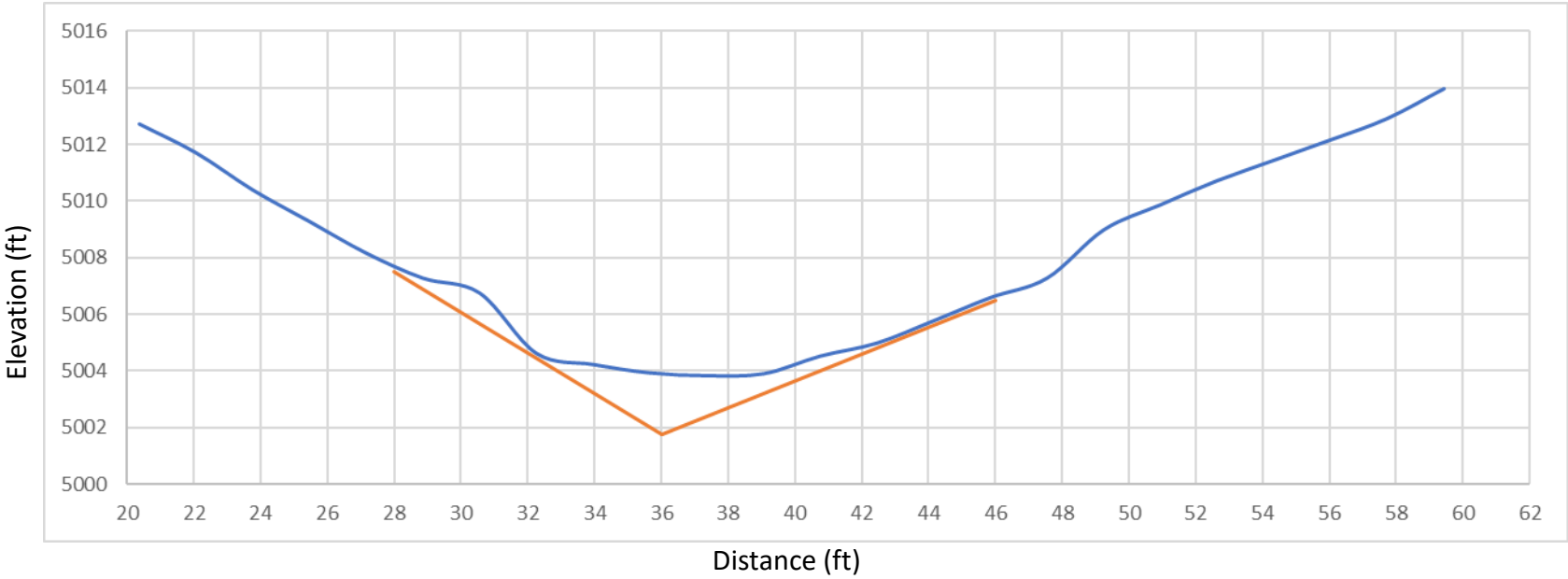
Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
184.70	15.39

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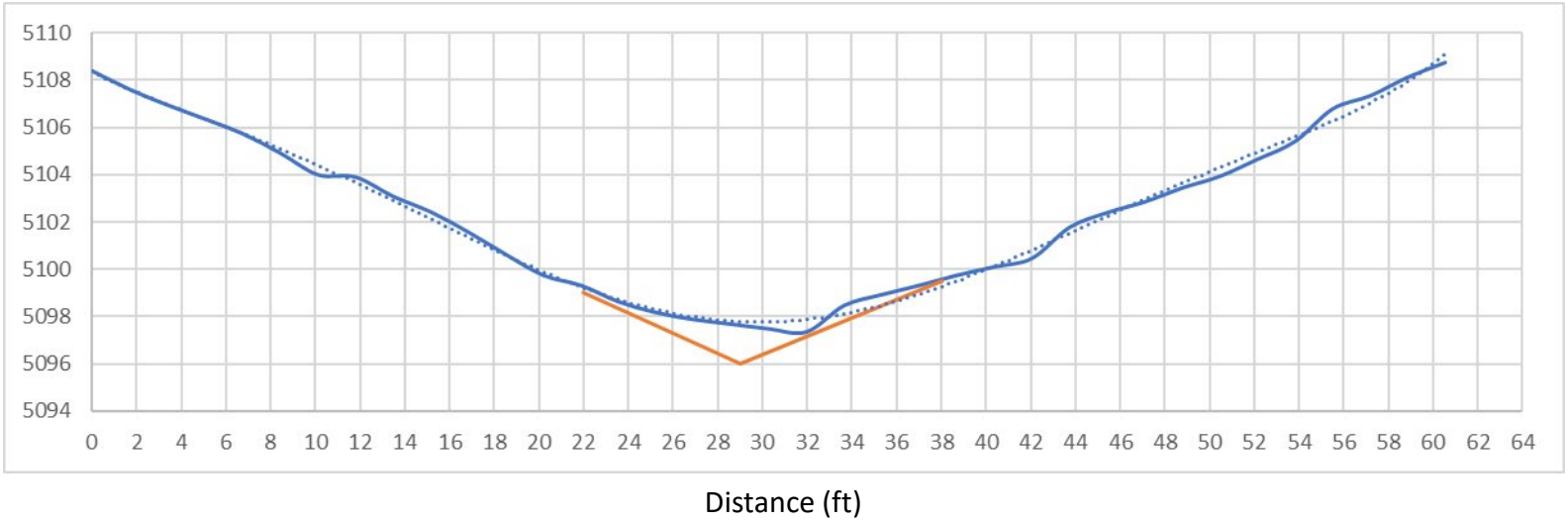
Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
36.84	3.07

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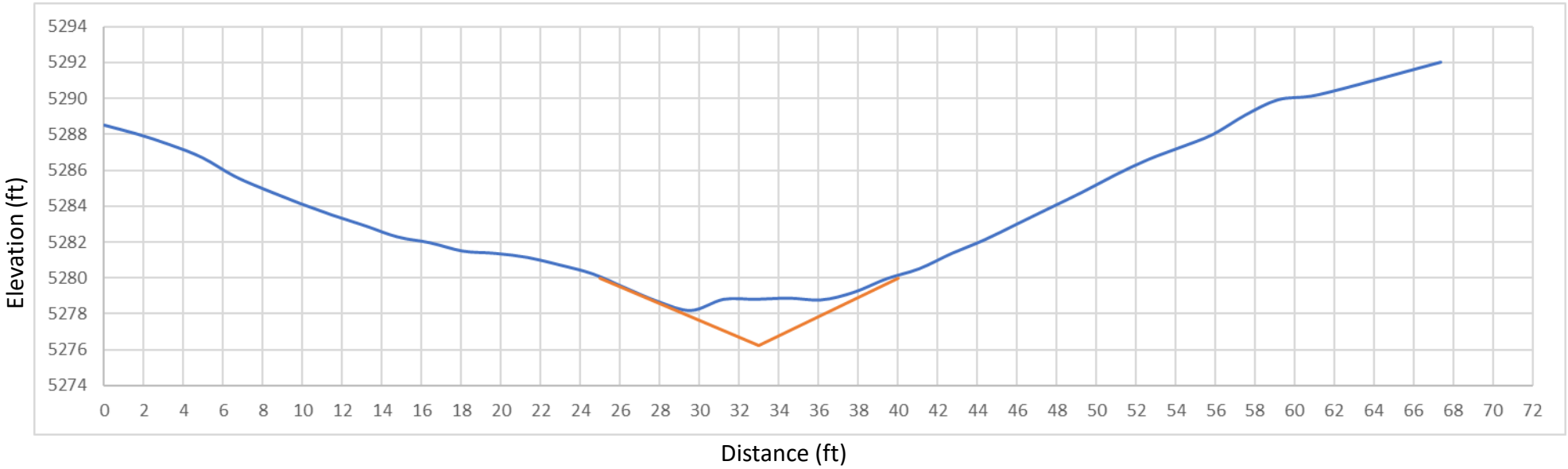
Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
141.93	11.83

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Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
107.47	8.96

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Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
155.51	12.96

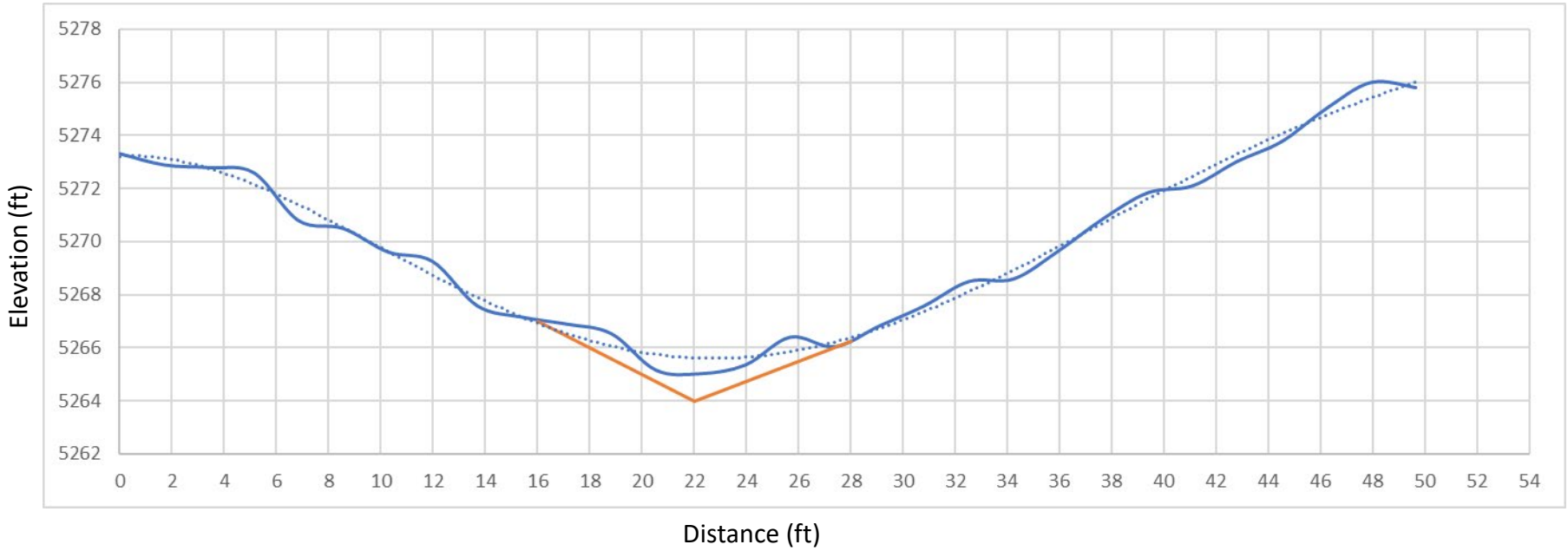
Plate C-25

Francis Fire Burn Scar Debris Flow Assessment
City of Fruit Heights
Fruit Heights, Utah
Project Number –1058-008

Cross-Section 25

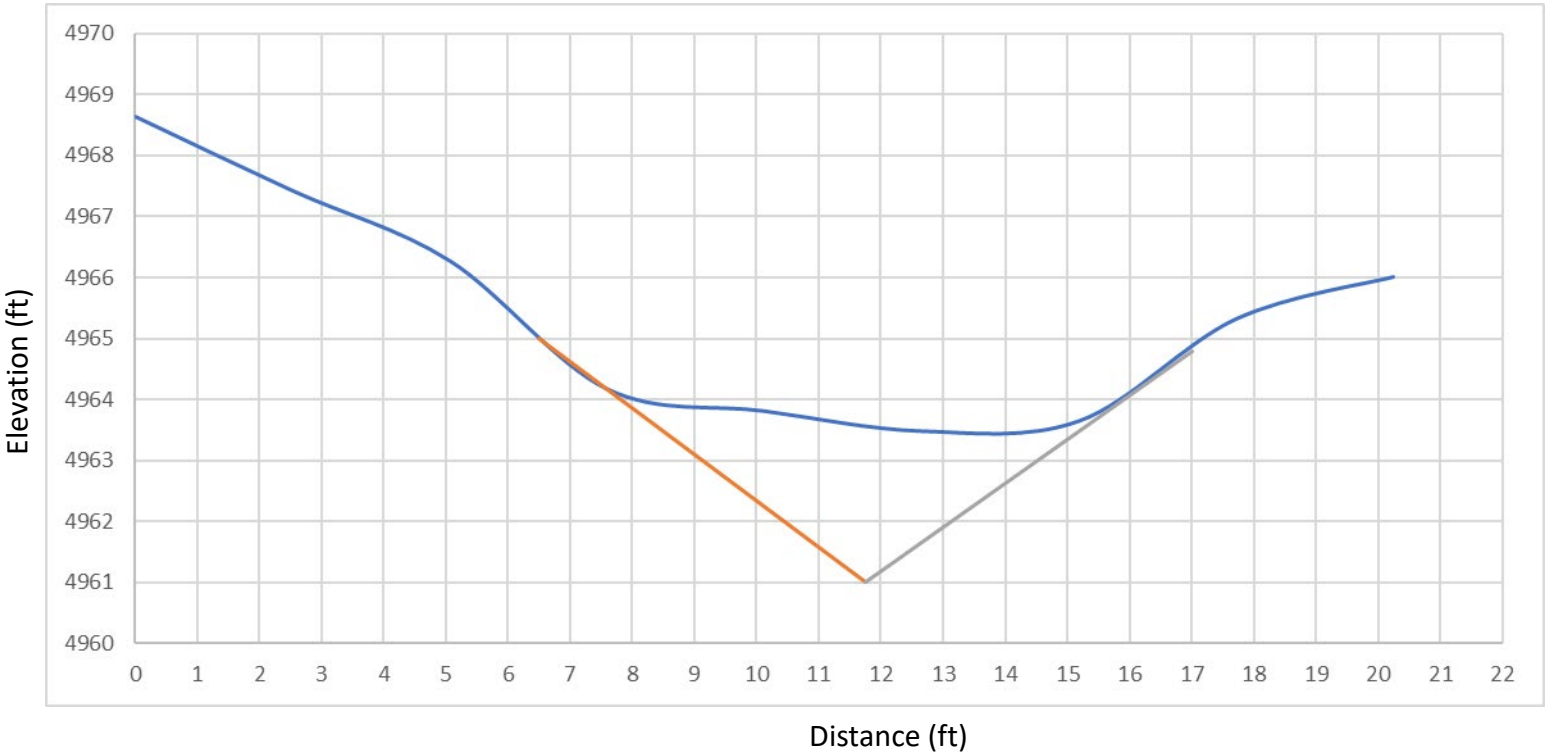
GeoStrata

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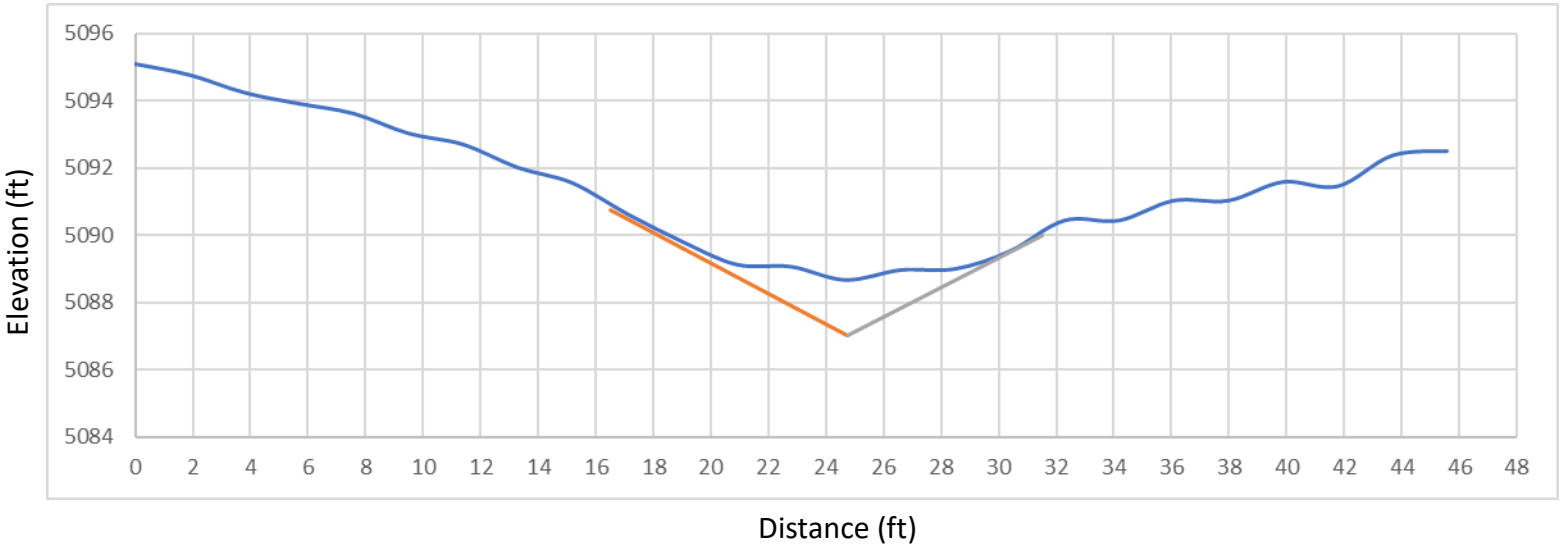
Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
81.84	6.82

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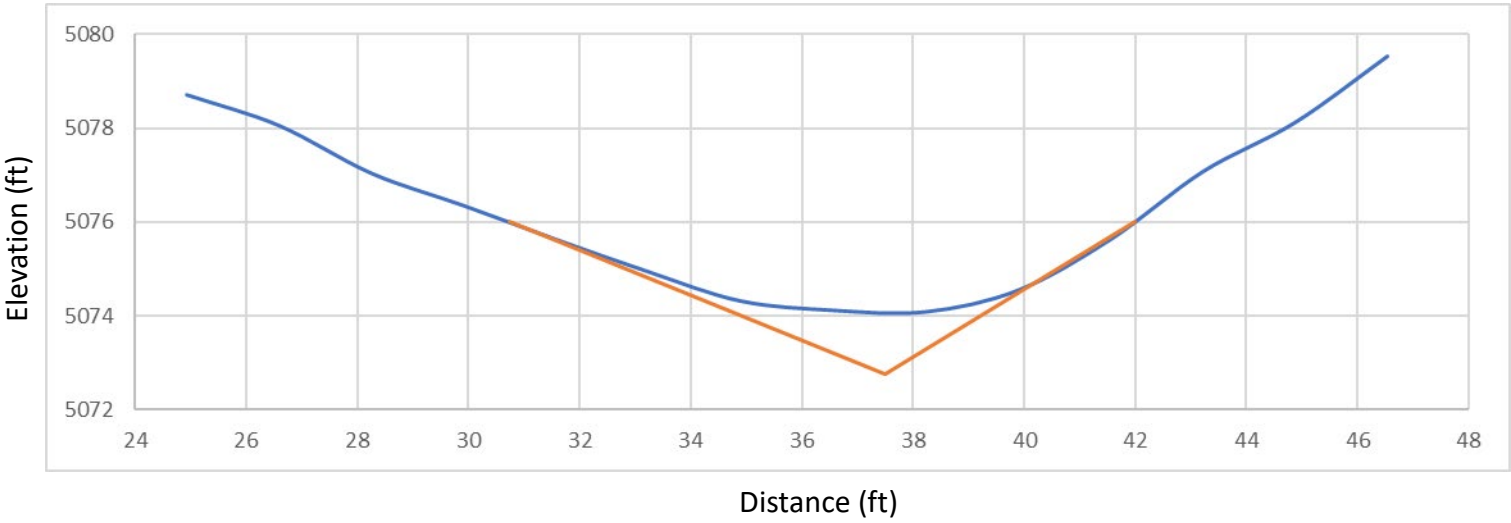
Area of Erodible Sediment (in2)	Area of Erodible Sediment (ft2)
109.24	9.10

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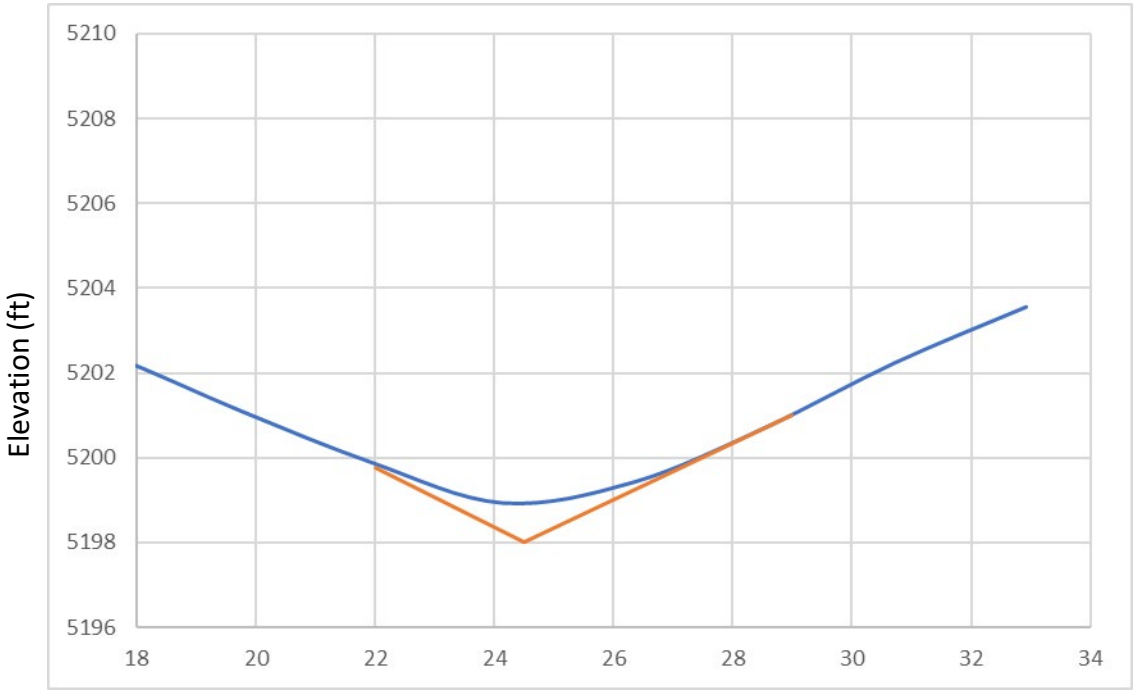
Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
111.72	9.31

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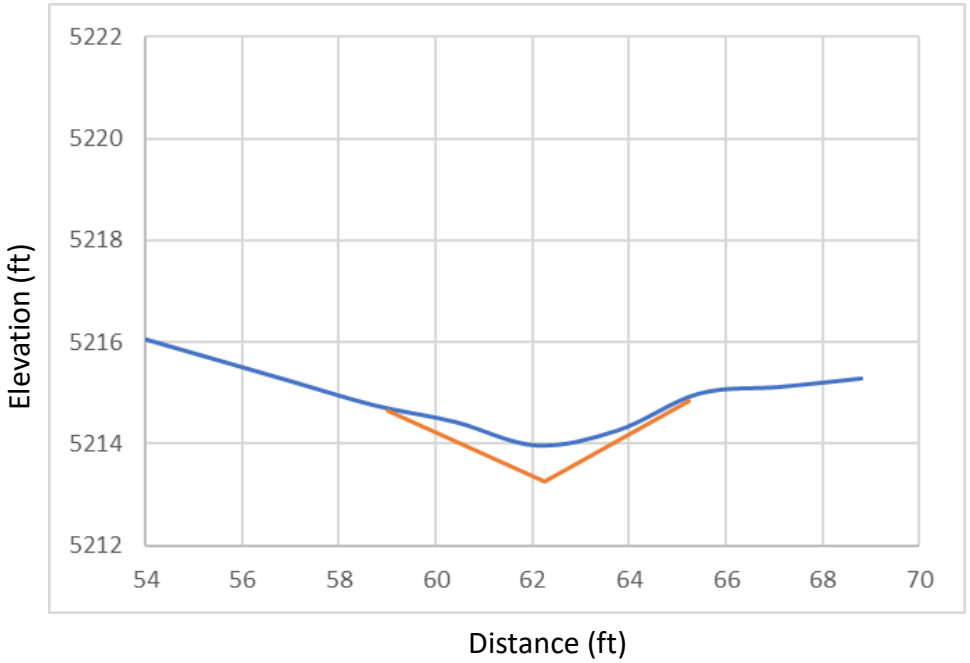
Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
27.15	2.26

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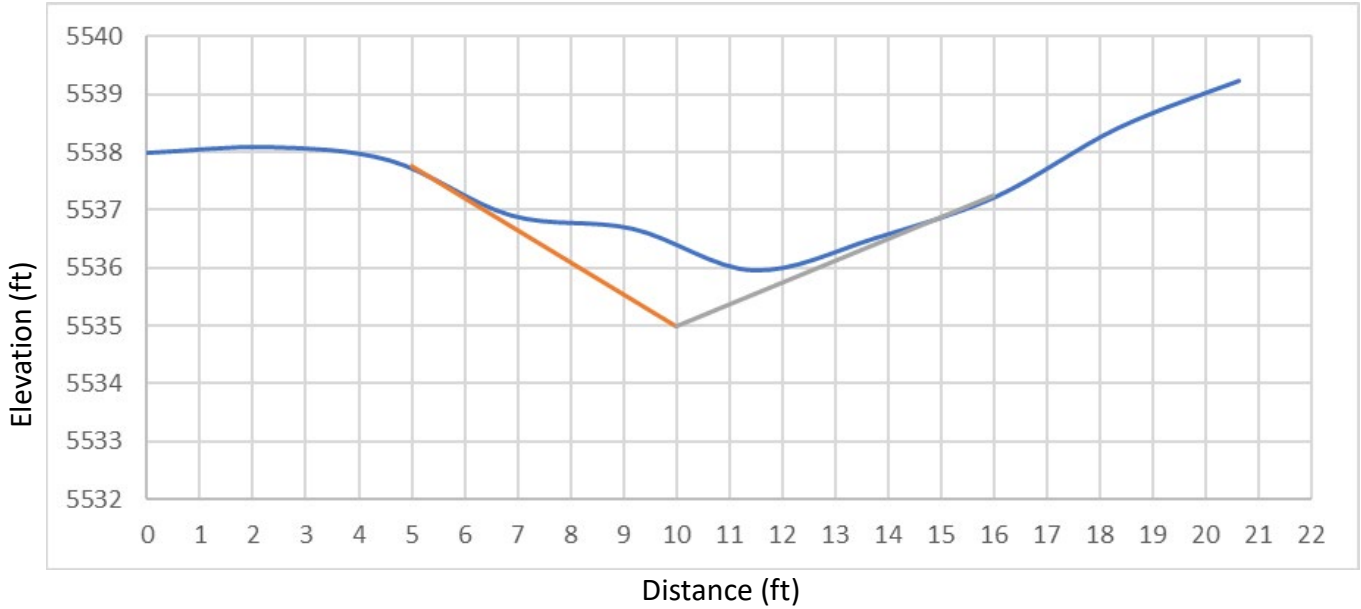
Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
22.78	1.90

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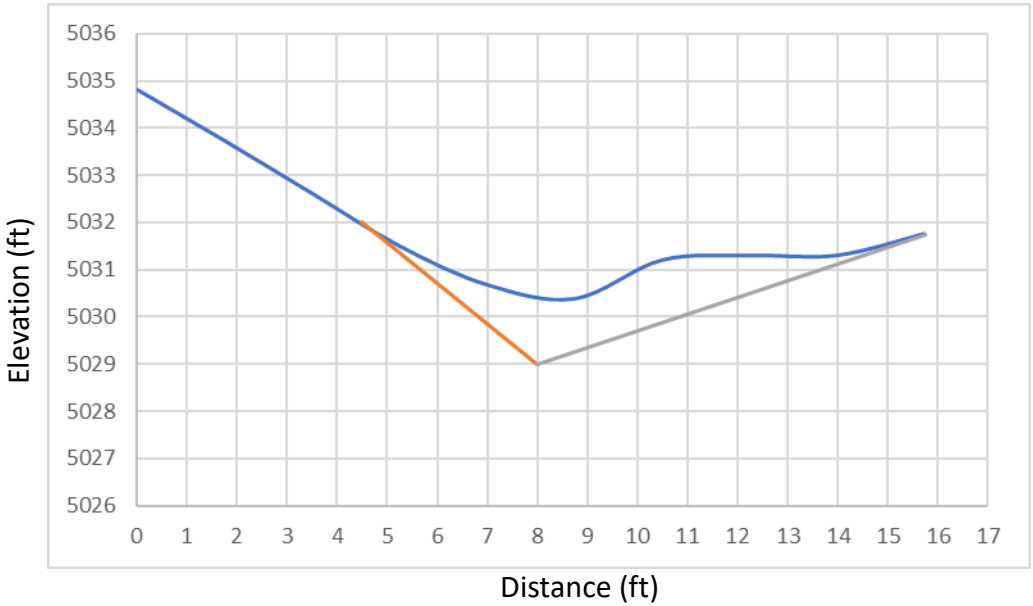
Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
21.17	1.76

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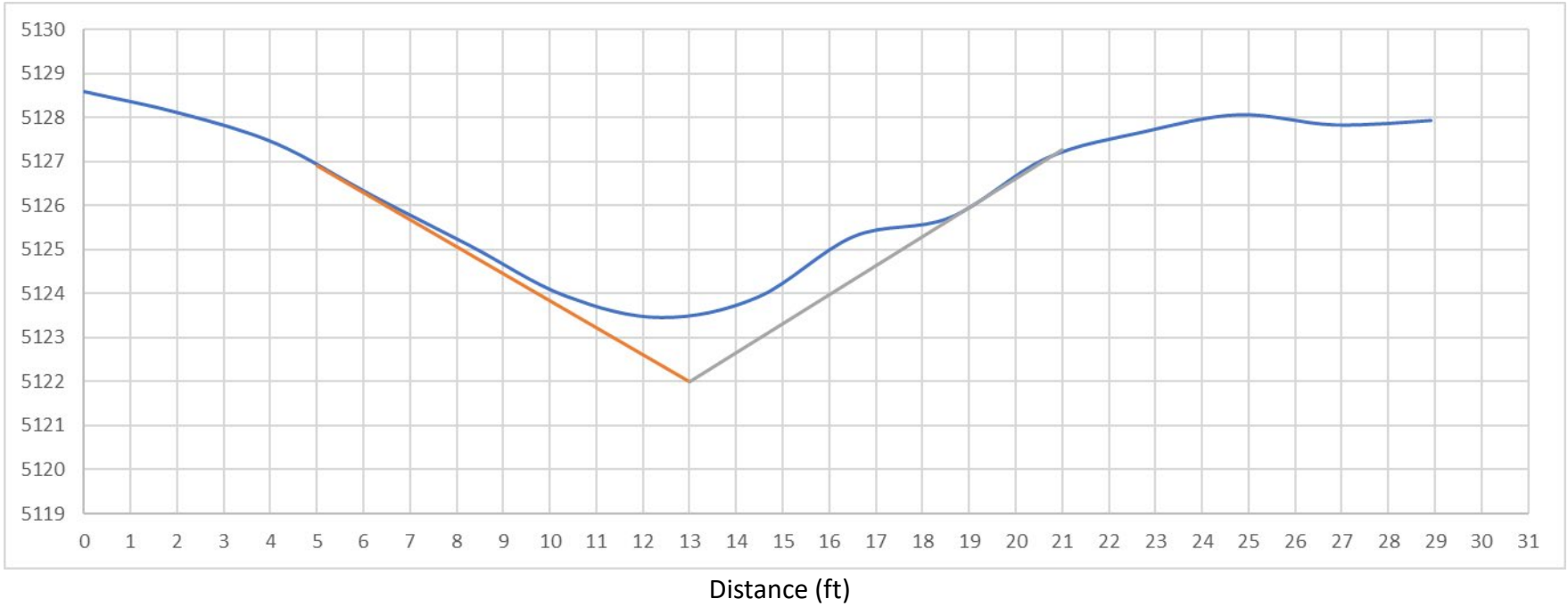
Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
52.99	4.42

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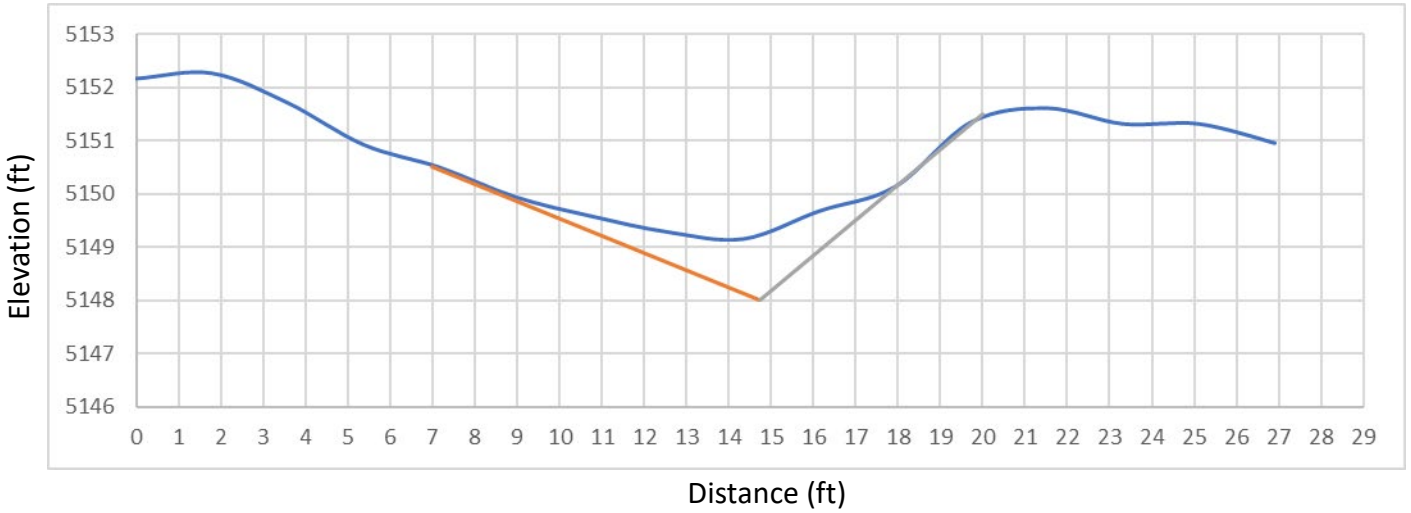
Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
92.41	7.70

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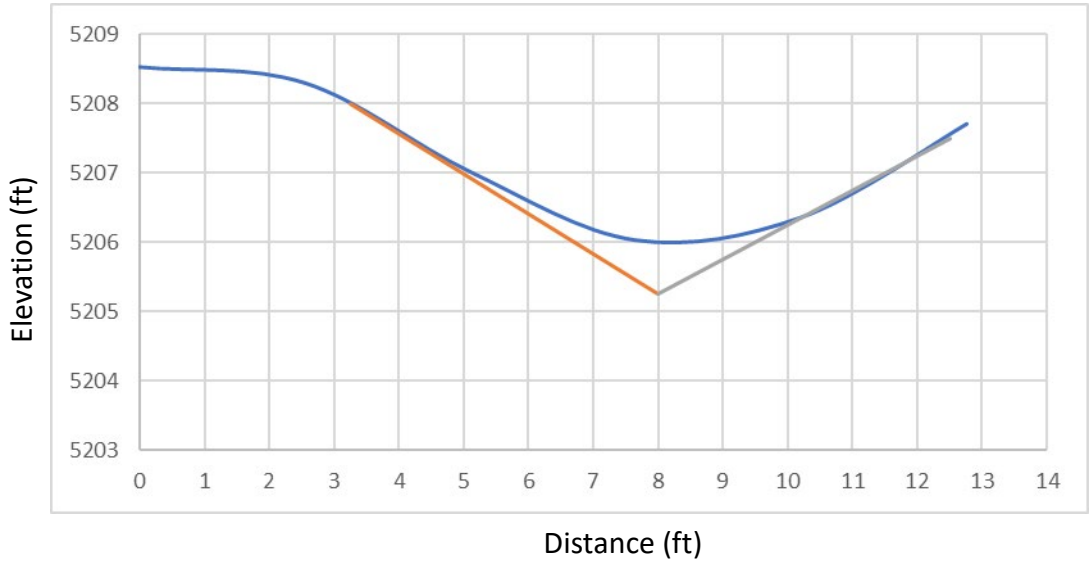
Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
91.86	7.65

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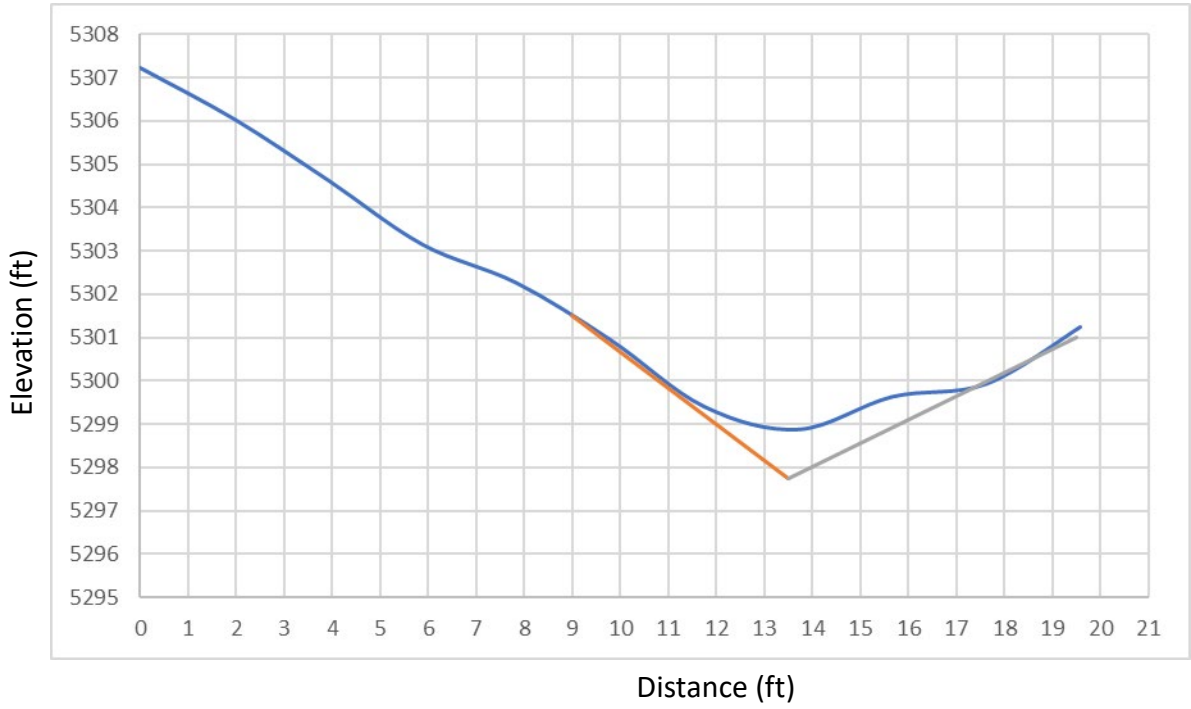
Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
58.43	4.87

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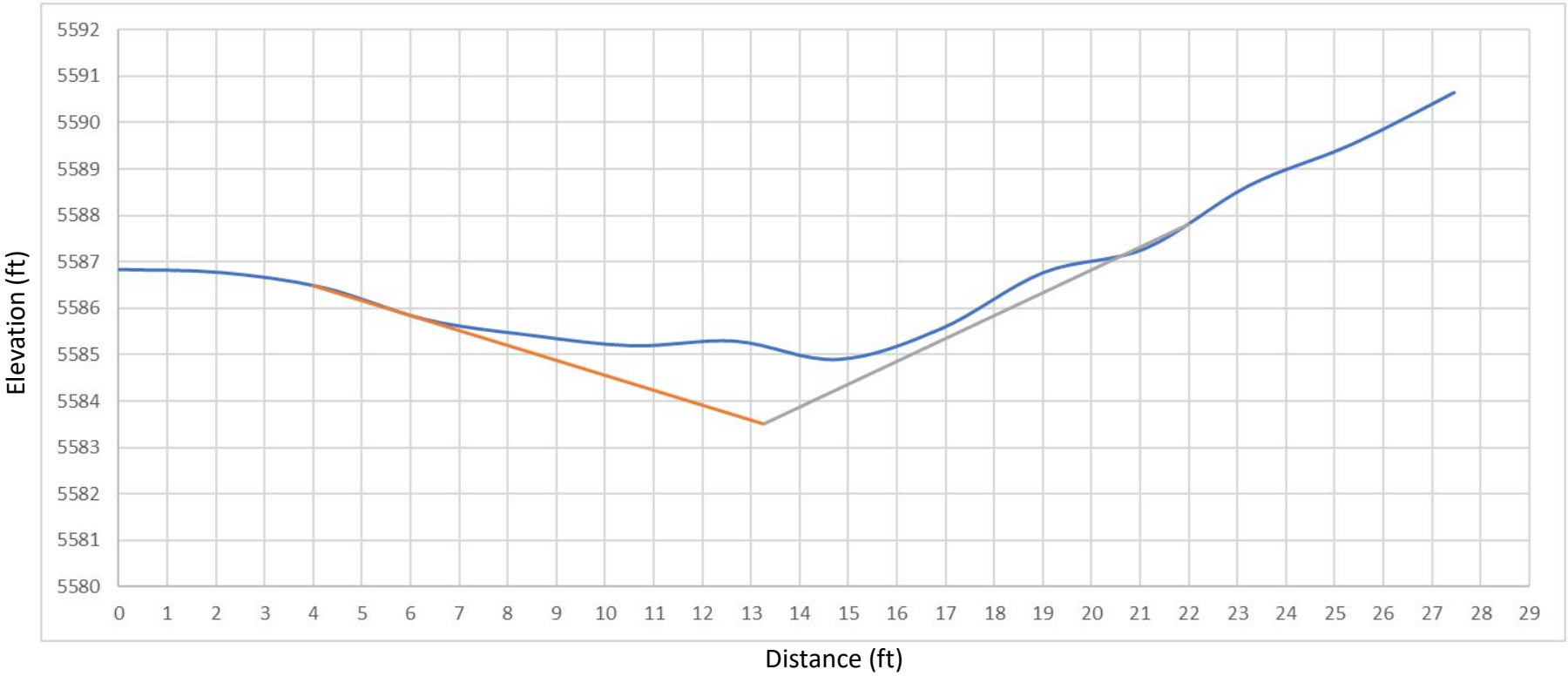
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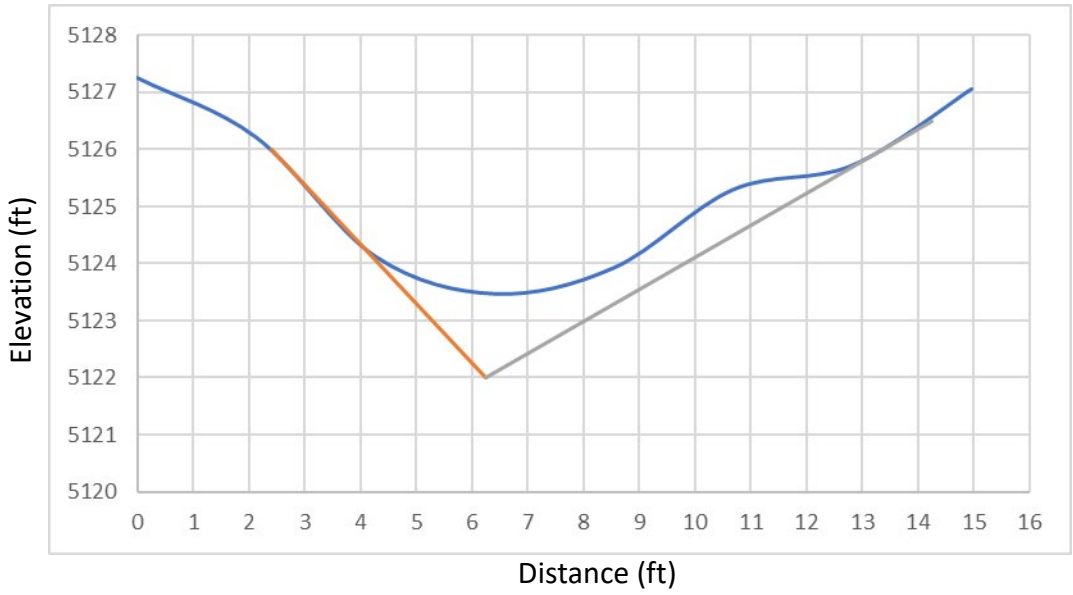
Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
48.24	4.02

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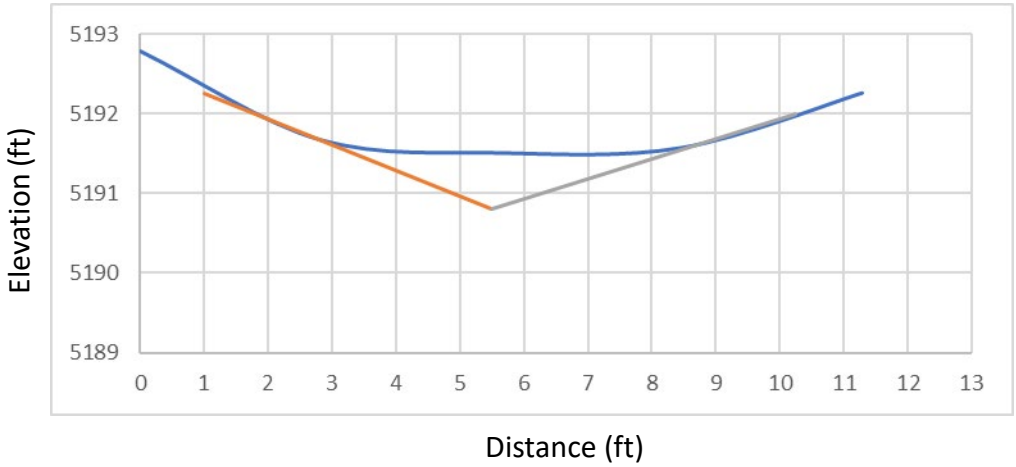
Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
105.97	8.83

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Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
74.37	6.20

GeoStrata
Copyright GeoStrata , 2020



Area of Erodible Sediment (in^2)	Area of Erodible Sediment (ft^2)
23.61	1.97

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April 12, 2017

Mr. Matt Lowe
Lowe Companies
6028 South Ridge Drive
South Ogden, Utah 84403

RE: Response to Review Comments
Raymond Property Development (Rock Loft Estates)
East of the Intersection of Mountain Road and Green Street
Fruit Heights, Utah
CMT Job No. 9502

Mr. Lowe:

As requested by Mr. Matt Lowe, this letter is to provide response to review comments provided by GeoStrata on behalf of Fruit Heights City dated March 25, 2016 related to the geotechnical study[1] and supplemental geotechnical study[2], prepared by GSH Geotechnical, Inc. (GSH) for the Raymond Property development (now known as Rock Loft Estates) in Fruit Heights, Utah. The undersigned engineer was the geotechnical engineer of record for the project. Review comments by Geostrata are shown in italics with responses given in plain text.

Comments and Response

Comment 1 – *3.2.4 Laboratory Direct Shear Test. GSH presents four directs shear tests in the Supplemental Geotechnical Study. One of these tests is from a previous report by GSH for the project dated September 24, 2014; however, the second direct shear in the previous report was not included in the supplemental report. The test that was not included represents a lower value than other tests performed on the same material. GeoStrata recommends that GSH explain why this test was not included in the supplemental report.*

Response – Following the completion of the September 24, 2014 geotechnical study, it came to the attention of the undersigned engineer that the results of the second direct shear test were actually results of testing for another project. These inaccurate results were omitted from the supplemental geotechnical study dated August 27, 2015 for this reason.

Comment 2 - *3.2.4 Laboratory Direct Shear Test. One of the direct shear tests performed on a sample from borings B-3 at a depth of 35 feet produced an angle of internal friction of 38 degrees and no cohesion. This value does not appear to represent a clay sample. GeoStrata recommends that GSH explain this discrepancy.*

[1] "Report, Geotechnical and Slope Stability Analysis, Proposed Raymond Property Residential Development, East of the Intersection of Mountain Road and Green Road, Fruit Heights, Utah," GSH Geotechnical, Inc, Job No. 1696-001-14, September 24, 2014.
[2] "Report, Supplemental Geotechnical Study, Proposed Raymond Property Residential Development, East of the Intersection of Mountain Road and Green Road, Fruit Heights, Utah," GSH Geotechnical, Inc, Job No. 1696-02N-15, August 27, 2015.

Response – The referenced supplemental geotechnical study contains an asterisk next to the sample description for this material and indicates "*Interbedded sand seams within sample." The higher internal friction angle and reduced cohesion are common in sand soils.

Comment 3 - *3.2.4 Laboratory Direct Shear Test. GSH presents test values for the direct shear testing performed, but does not provide plates showing the test results. GeoStrata recommends that GSH provide plates showing the direct shear testing results.*

Response – Please see the attached figures provided by GSH.

Comment 4 – *5.3 Slope Stability. Soil strength values used for the slope stability assessments in the GSH Supplemental Geotechnical Study are based on referenced values presented by the Bureau of Reclamation. While use of such values is useful where laboratory testing is not practical, it is preferable to use laboratory data specific to subject site. Several test values used for the sand soils at the site are lower than the reference values used in the GSH analyses. GeoStrata recommends that GSH use laboratory test values in their analyses where practical.*

Response – The results of the site specific testing were utilized where practical for the slope stability analyses. Due to the layered nature of the soil deposits at the site, interbedded sand seams were encountered within the clay samples recovered at the site. Direct shear tests performed on the predominately clay soils produced results more appropriate for sandy soils, which is indicative of the shear occurring on the thin interbedded sand seams within the sample. Therefore, assumed values based on the Bureau of Reclamation guidelines were used for the clay soils. The strength parameters for the sand soils used in the modeling were based largely on an average value of the site specific direct shear tests performed as part of the referenced geotechnical studies. Average values based on the direct shear tests performed are an internal friction angle of 36 degrees and a cohesion of 91 psf. It should be noted that the strength parameters for the sand soils utilized for the slope stability analyses in the referenced geotechnical studies are within the ranges indicated by the Bureau of Reclamation.

Comment 5 - *5.3 Slope Stability. The GSH Supplemental Geotechnical Study presents written descriptions of the two profiles assessed; however, Figure 2 does not present the location of the profiles. GeoStrata recommends that GSH update the Figure 2 to show the location of the profiles assessed.*

Response – The referenced supplemental geotechnical study indicates that further information related to the cross-section location is provided within the project geologic study contained in the appendix of the referenced supplemental geotechnical study. A copy of the figure from the project geologic study showing the location of the cross-sections is attached to this letter.

Comment 6 - *5.3 Slope Stability. The geologic hazards evaluation performed for GSH by Western GeoLogic dated August 15, 2015 identifies four landslides on near the subject property. Western GeoLogic identifies 3 landslides on the slope on the west side of the property and one landslide east of the site. The slope stability assessments presented in the GSH Supplemental Geotechnical Study does not appear to consider these landslides in their analyses. GeoStrata recommends that GSH perform additional slope stability assessments at the location of the landslides as well as other steep slopes at the site. These assessments should include if necessary:*

a. Additional subsurface investigations to adequately characterize subsurface conditions.

b. Coordination with Western GeoLogic to develop a reasonable geologic cross section for use in slope stability assessments.

c. Laboratory testing to adequately assess soil strengths. For landslide deposits residual direct shear or ring shear testing is typically considered suitable.

Response – We understand that lots within the landslide deposits will not be developed at this time; therefore, further response to this comment is not required at this time.

Comment 7 – In a recent meeting with representatives of Fruit Heights City, the front yard setback requirements for the development were discussed. The supplemental geotechnical report recommends using a 20-foot setback. The design team was asked to explain this recommendation.

Response - The 20-foot front yard setback was utilized in the slope stability modeling for the site as part of the referenced supplemental geotechnical study. The 20-foot front yard setback helps to minimize grading at the site by reducing cuts into existing slopes on uphill lots. Additionally, the 20-foot front yard setback creates a larger distance from the crest of the slopes on downhill lots, thus reducing the load imposed on the existing slopes. We understand from our discussions with representative of Fruit Heights City that the 10 foot reduction in the front yard setback would result in an increase of 10 feet in the rear yard setback, which is suitable from a geotechnical perspective.

Limitations

The recommendations contained herein are based solely on the soil characteristics determined during the referenced geotechnical study. It is possible that soil conditions other than those observed during the referenced study may exist, which could potentially be problematic. We cannot assume responsibility for conditions of which we are not aware or have not observed.

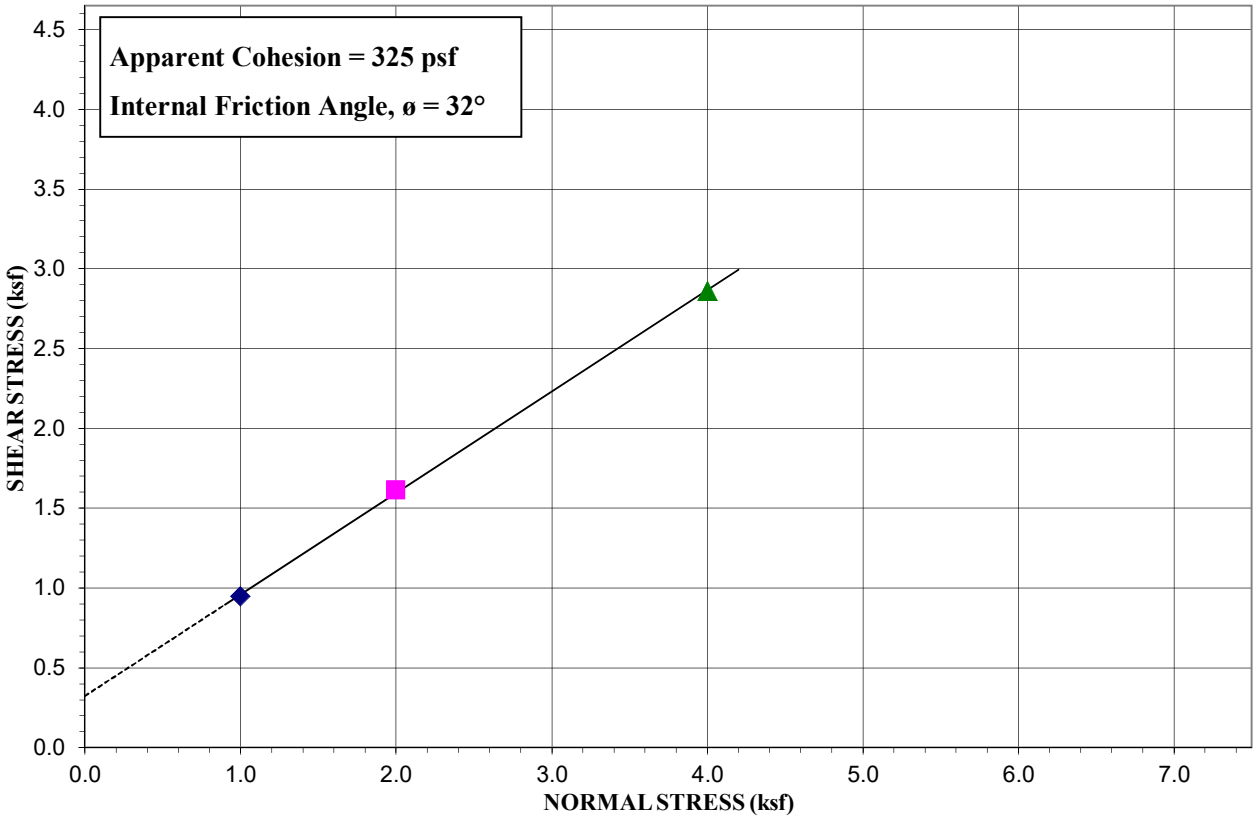
If you have any questions, please call.

Sincerely,
CMT Engineering Laboratories



Andrew M. Harris, P.E.
Senior Geotechnical Engineer

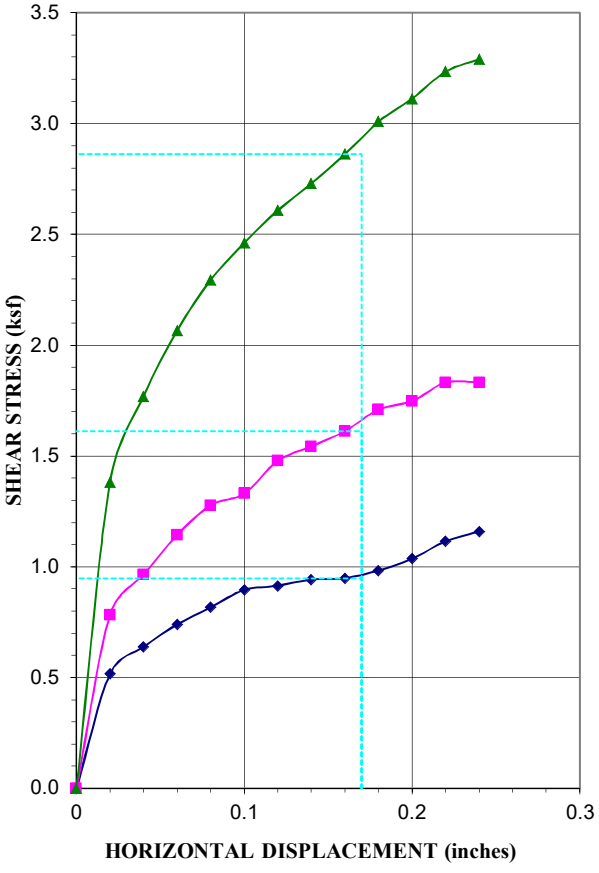
DIRECT SHEAR TEST



Source:	TP4	Depth:	3.5 ft	
Type of Test:	Consolidated Undrained			
Sample Type	TW			
Strain Rate	0.01 in/min			
Test No. (Symbol)		1 (◆)	2 (■)	3 (▲)
Initial Height, in.		1	1	1
Diameter, in.		2.42	2.42	2.42
Dry Density Before, pcf		96.0	90.7	94.6
Dry Density After, pcf		93.8	99.3	101.5
Moisture % Before		15.2	15.2	14.3
Moisture % After		15.2	15.2	14.3
Normal Load, ksf		1.0	2.0	4.0
Shear Stress, ksf		0.95	1.61	2.86
Vertical Deformation, in		1.0	2.0	4.0

Sample Properties	
Cohesion, psf	325
Friction Angle, ϕ	32
Liquid Limit, %	---
Plasticity Index, %	---
Percent Gravel	
Percent Sand	
Percent Passing #200 sieve	
Classification	SM/ML

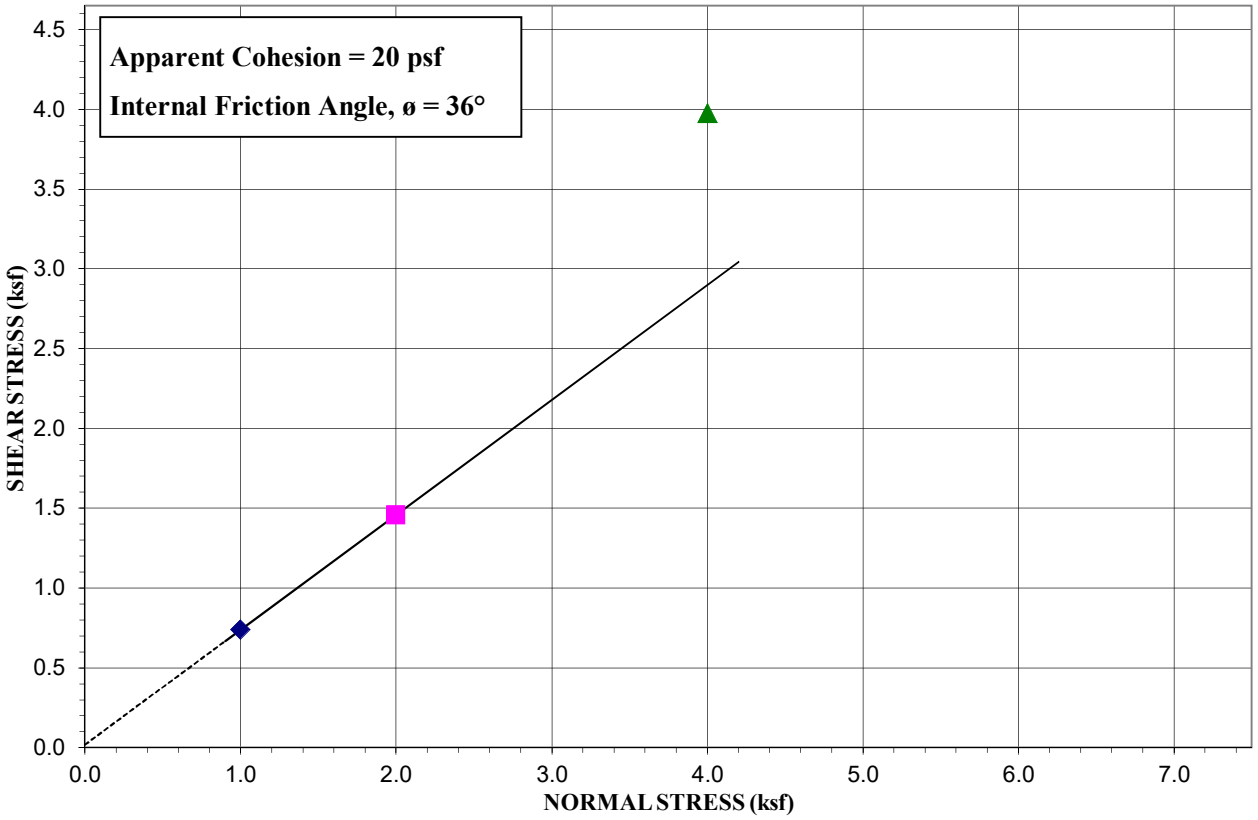
Testing Laboratory: AMEC, SLC, Utah



PROJECT: Raymond Property

PROJECT NO.:	1696-001-14	GSH	FIGURE NO.:	2

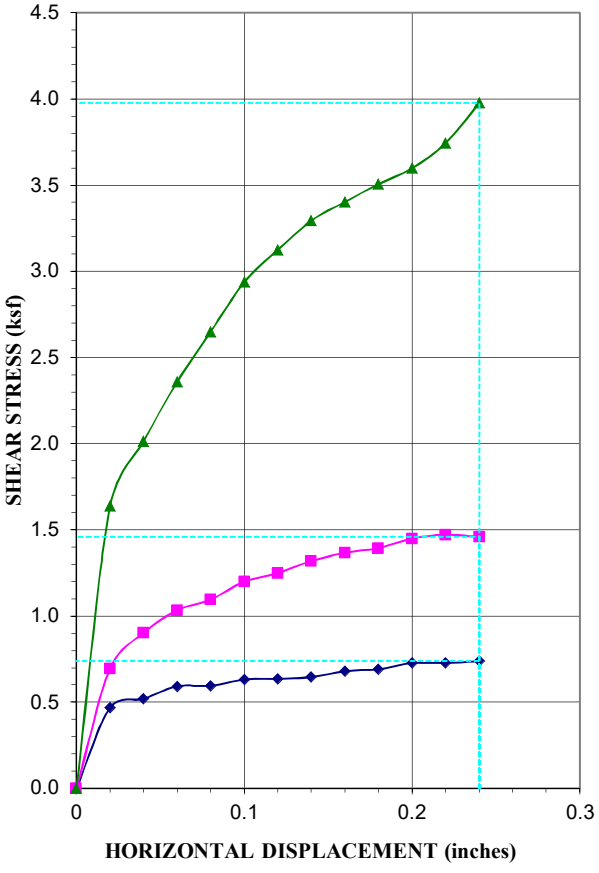
DIRECT SHEAR TEST



Source: TP8	Depth: 5.0 ft		
Type of Test:	Consolidated Undrained		
Sample Type	TW		
Strain Rate	0.01 in/min		
Test No. (Symbol)	1 (◆)	2 (■)	3 (▲)
Initial Height, in.	1	1	1
Diameter, in.	2.42	2.42	2.42
Dry Density Before, pcf	68.6	73.5	75.1
Dry Density After, pcf	71.6	85.2	87.0
Moisture % Before	27.8	27.0	26.3
Moisture % After	27.8	27.0	26.3
Normal Load, ksf	1.0	2.0	4.0
Shear Stress, ksf	0.74	1.46	3.98
Vertical Deformation, in	1.0	2.0	4.0

Sample Properties	
Cohesion, psf	20
Friction Angle, ϕ	36
Liquid Limit, %	---
Plasticity Index, %	---
Percent Gravel	
Percent Sand	
Percent Passing #200 sieve	
Classification	SM/ML

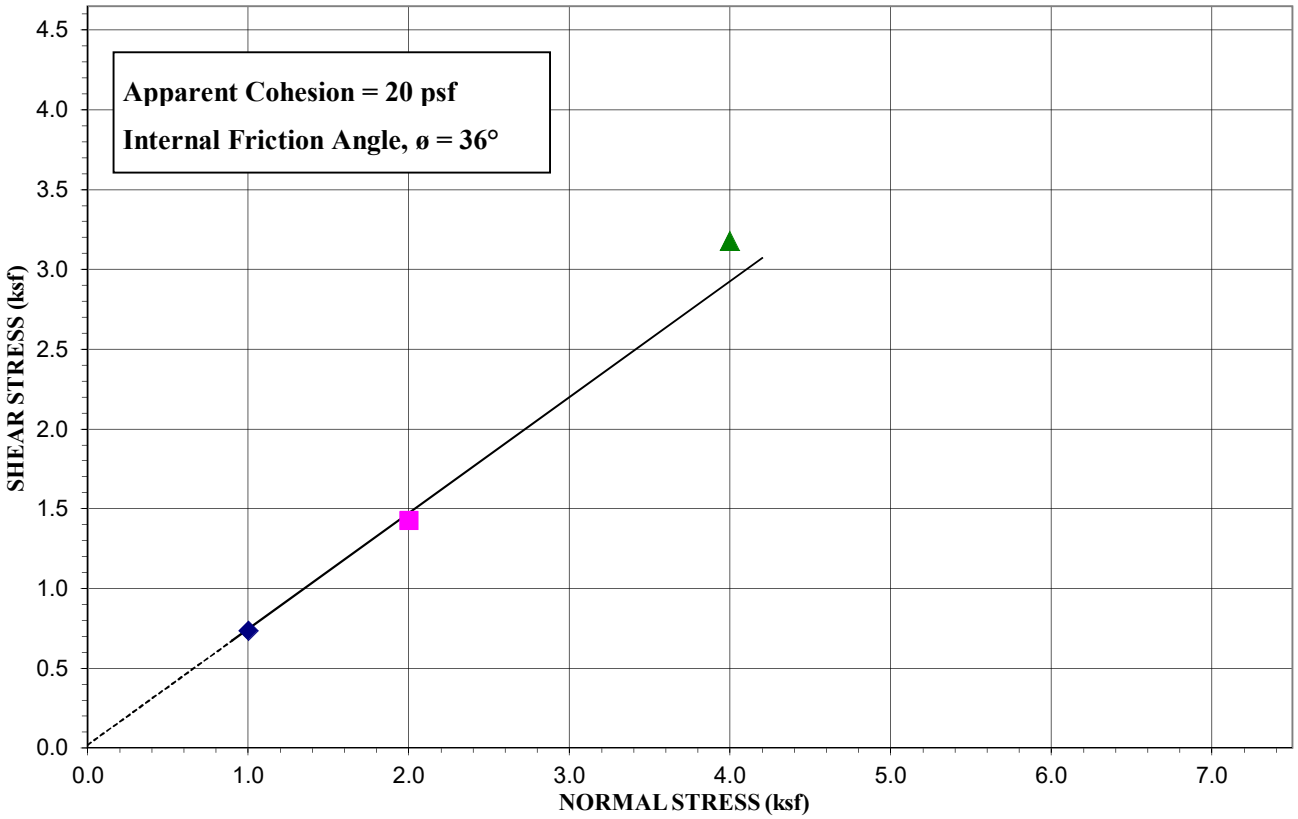
Testing Laboratory: AMEC, SLC, Utah



PROJECT: Raymond Property

PROJECT NO.: 1696-001-14	GSH	FIGURE NO.: 2

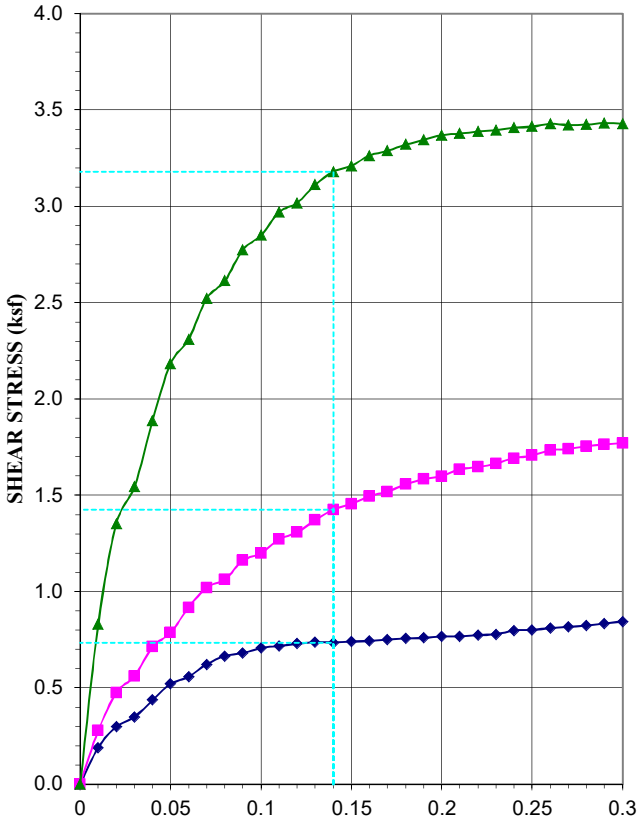
DIRECT SHEAR TEST



Source: B-2	Depth: 10.5 ft		
Type of Test:	Consolidated - Undrained		
Test No. (Symbol)	1 (◆)	2 (■)	3 (▲)
Sample Type	Remolded to In-Situ Moisture		
Initial Height, in.	1.06	1.16	1.10
Diameter, in.	2.45	2.45	2.45
Dry Density Before, pcf	104.0	107.7	108.1
Dry Density After, pcf	106.0	116.3	113.0
Moisture % Before	10.5	9.6	9.9
Moisture % After	15.9	16.2	16.5
Saturation % Before	47.3	47.3	49.5
Saturation % After	75.2	101.9	94.7
Normal Load, ksf	1.0	2.0	4.0
Shear Stress, ksf	0.74	1.43	3.18
Strain Rate	0.005 in/min		

Sample Properties	
Cohesion, psf	20
Friction Angle, ϕ	36
Liquid Limit, %	---
Plasticity Index, %	---
Percent Gravel	---
Percent Sand	---
Percent Passing No. 200 sieve	---
Classification	Silty Sand (SM)

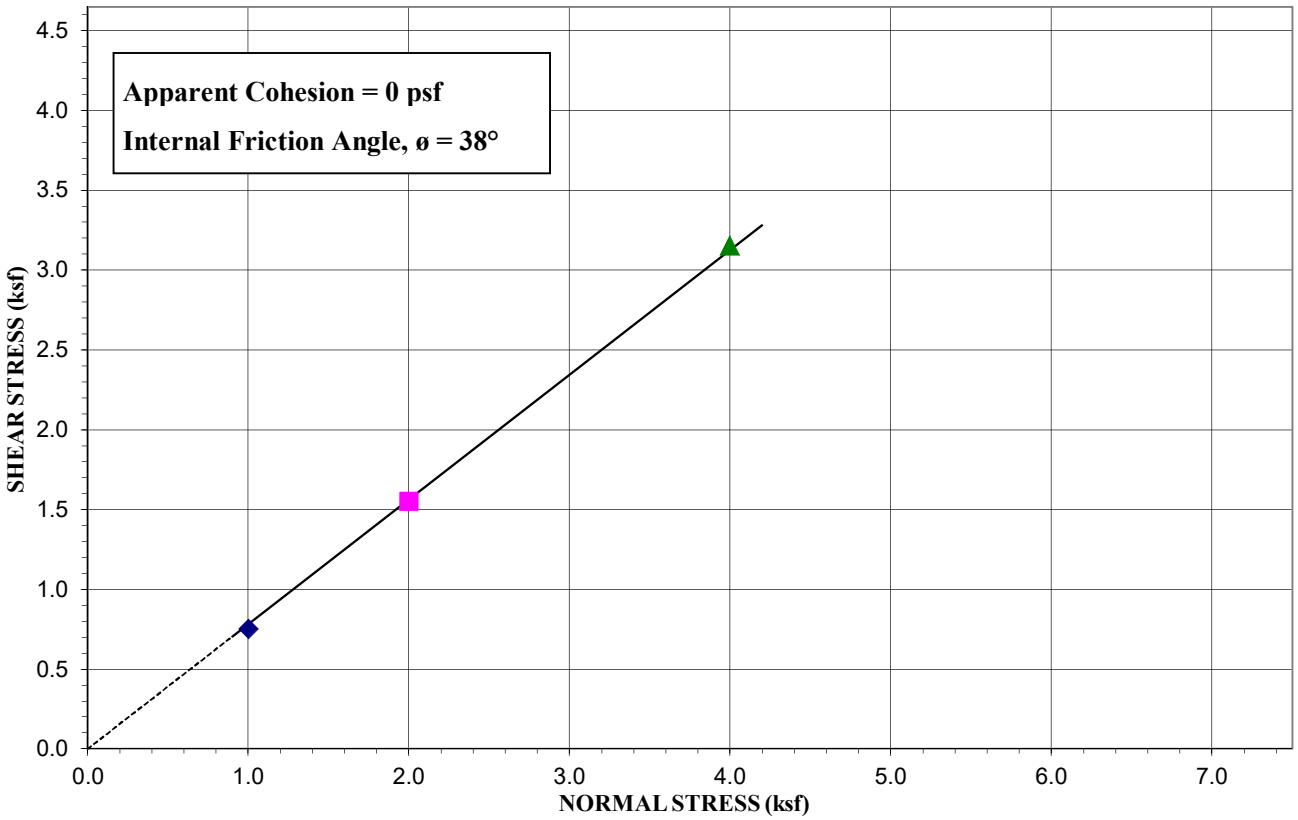
Testing Laboratory: GSH



PROJECT: Raymond Property

PROJECT NO.: 1696-02N-15	GSH	FIGURE NO.: 5A

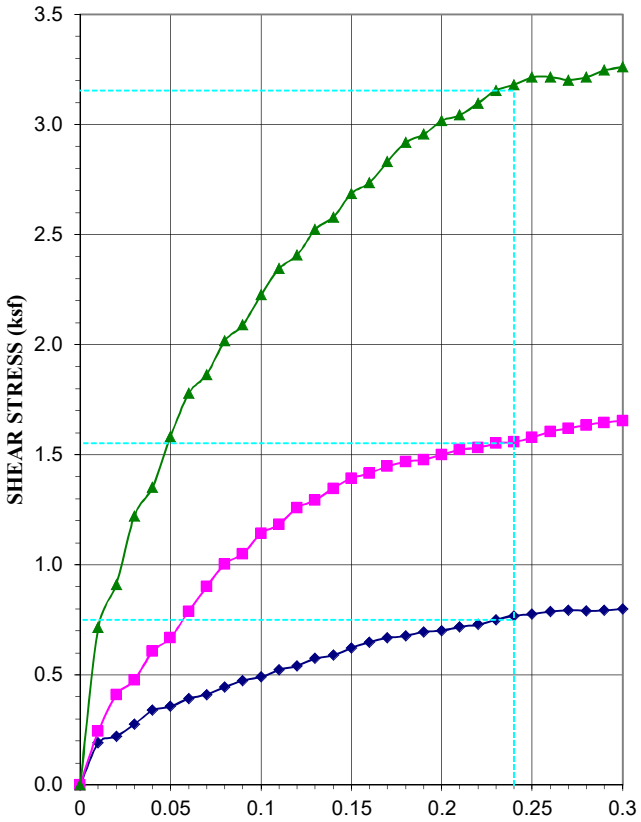
DIRECT SHEAR TEST



Source: B-3	Depth: 35 ft		
Type of Test:	Consolidated - Undrained		
Test No. (Symbol)	1 (◆)	2 (■)	3 (▲)
Sample Type	Relatively Undisturbed		
Initial Height, in.	1.06	1.16	1.10
Diameter, in.	2.45	2.45	2.45
Dry Density Before, pcf	87.1	90.6	85.3
Dry Density After, pcf	90.0	96.1	88.2
Moisture % Before	27.2	22.8	29.7
Moisture % After	31.9	28.4	33.4
Saturation % Before	77.2	70.0	80.7
Saturation % After	97.0	99.6	97.2
Normal Load, ksf	1.0	2.0	4.0
Shear Stress, ksf	0.75	1.55	3.15
Strain Rate	0.005 in/min		

Sample Properties	
Cohesion, psf	0
Friction Angle, ϕ	38
Liquid Limit, %	---
Plasticity Index, %	---
Percent Gravel	---
Percent Sand	---
Percent Passing No. 200 sieve	---
Classification	Silty Sand (SM)

Testing Laboratory: GSH



PROJECT: Raymond Property

PROJECT NO.: 1696-02N-15

GSH

FIGURE NO.: 6A

DIRECT SHEAR TEST

EXPLANATION

- Fault, bar and ball on downthrown side, dashed where approximately located.
- Setback zone. No structures designed for human occupancy.
- Great Basin Earth Science (GBES, 2008) trench locations.
- Slope failures along fault scarp.
- Alluvial fans.
- Trenches and test pits, this study.
- 1937 ephemeral drainage courses.
- A—A' Slope cross section.
- Steep slope boundary.

N W E S

0 50 100 feet

Scale 1:1,200
(1 inch = 100 feet)

Air photo base, site plan, topography, and surveyed trench and fault locations from Reeve & Associates, Inc.; Rock Loft Estates Subdivision preliminary plan dated May 12, 2016.

SITE PLAN

GEOLOGIC HAZARDS EVALUATION
Raymond Property
About 175 South Mountain Road
Fruit Heights, Utah

FIGURE 4

APPENDIX C

SUPPORTING
DOCUMENTATION

CMT ENGINEERING LABORATORIES

April 27, 2021

Mr. Matt Lowe
Lowe Companies
6028 South Ridge Drive, #203
South Ogden, Utah 84403

Subject: Addendum to Geotechnical Engineering and Geologic Study
Infinite Slope with Seepage Analysis

Proposed Rock Loft Ridge Subdivision
About 250 North 1450 East
Fruit Heights, Utah
CMT Project No. 12188

Gentlemen,

Recently, a geotechnical and geologic study[1] were completed for the site by CMT. As part of the review process, review comments were provided by GeoStrata, dated March 3, 2021. As requested per paragraph 6 of review comment number 7, we are presenting our analyses for an infinite slope with seepage for the existing fault scarp.

The analysis requirements consist of a 4-foot slip surface depth, fully saturated, using infinite slope analysis methods. Three of the steepest cross sections, previously analyzed for global stability, were utilized herein and were comprised of cross sections E-E, F-F, and G-G.

The soil properties at the fault scarp are tabulated below and consist of the previously designated Holocene to Late Pleistocene mass wasting colluvium.

Material	Internal Friction Angle (degrees)	Apparent Cohesion (psf)	Unit Weight (pcf)*
Mass Wasting Colluvium	33	0*	120

* Cohesion omitted

[1] "Combined Geotechnical Engineering and Geologic Study, Proposed Rock Loft Ridge Subdivision, About 250 North 1450 East, Fruit Heights, Utah," CMT Job No. 12188, November 12, 2020 (Revised February 3, 2021).

INFINITE SLOPE ANALYSIS RESULTS

The results of our infinite slope stability are tabulated below.

Slope Cross Section	Slope Angle in Degrees	Calculated Factor of Safety
E-E	32.53	0.489
F-F	26.39	0.628
G-G	28.52	0.5736

The factor of safety for an infinite slope with a saturated, 4-foot slip surface, utilizing the previously accepted strength values for the mass wasting colluvium soil, are below 1.0 at each location analyzed and indicate some failure/movement may occur under these conditions. The results of our analyses are attached.

However, based on prior setbacks (derived for existing slope steepness, fault setback, and global stability requirements), the proposed individual home setbacks will be multiple times greater than the 4-foot slip surface analyzed and in our opinion these results do not impact the setbacks utilized in the final design of the site property.

CLOSURE

We appreciate the opportunity to work with you for this project. Please let us know if you have any question concerning this addendum.

Sincerely,

CMT Engineering Laboratories

Bryan N. Roberts, P.E.
Senior Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
276478
BRYAN N. ROBERTS
4/27/2022
STATE OF UTAH

Reviewed by:

Andrew M. Harris, P.E.
Geotechnical Division Manager

SURFICIAL SLOPE STABILITY (INFINITE SLOPE)

Project Name: Rock Loft Ridge Scarp section from Cross Section E-E
Job Number: 12188

friction angle:	33	deg (ϕ')
cohesion:	0	psf (c')
soil unit weight:	120	pcf (γ)
slip surface depth:	4	feet (z)
groundwater depth:	0	feet
slope angle:	32.5	deg (B)

m = 1

Factor of Safety, F = {0 psf + (120 pcf - [1] * 62.4 pcf) * 4 ft * $\cos^2$(32.5) * tan(33) } / [120 pcf * 4 ft * sin(32.5) * cos(32.5)]
0.489296

Reference:
Blake et al, 2002, Guidelines for Analyzing and Mitigating Landslide Hazards in California.
(Equation is as follows:)

$$F = \frac{c' + (\gamma - m\gamma_w) z \cos^2 B \tan\phi'}{\gamma z \sin B \cos B} \qquad (9.5)$$

where F is the factor of safety, c' is the cohesion intercept, z is the vertical depth of the slip surface, B is the slope angle, γ is the unit weight (density) of the soil, γ_w is the unit weight of the water, m is the fraction of z such that mz is the vertical height of the groundwater table above the slip surface, and ϕ' is the angle of shearing resistance.

SURFICIAL SLOPE STABILITY (INFINITE SLOPE)

Project Name: Rock Loft Ridge Scarp section from Cross Section F-F
Job Number: 12188

friction angle:	33	deg (ϕ')
cohesion:	0	psf (c')
soil unit weight:	120	pcf (γ)
slip surface depth:	4	feet (z)
groundwater depth:	0	feet
slope angle:	26.39	deg (B)

m = 1

Factor of Safety, F = {0 psf + (120 pcf - [1] * 62.4 pcf) * 4 ft * $\cos^2$(26.39) * tan(33) } / [120 pcf * 4 ft * sin(26.39) * cos(26.39)]
0.628222

Reference:
Blake et al, 2002, Guidelines for Analyzing and Mitigating Landslide Hazards in California.
(Equation is as follows:)

$$F = \frac{c' + (\gamma - m\gamma_w) z \cos^2 B \tan\phi'}{\gamma z \sin B \cos B} \qquad (9.5)$$

where F is the factor of safety, c' is the cohesion intercept, z is the vertical depth of the slip surface, B is the slope angle, γ is the unit weight (density) of the soil, γ_w is the unit weight of the water, m is the fraction of z such that mz is the vertical height of the groundwater table above the slip surface, and ϕ' is the angle of shearing resistance.

SURFICIAL SLOPE STABILITY (INFINITE SLOPE)

Project Name: Rock Loft Ridge Scarp section from Cross Section G-G
Job Number: 12188

friction angle:	33	deg (ϕ')
cohesion:	0	psf (c')
soil unit weight:	120	pcf (γ)
slip surface depth:	4	feet (z)
groundwater depth:	0	feet
slope angle:	28.52	deg (B)

m = 1

Factor of Safety, F = {0 psf + (120 pcf - [1] * 62.4 pcf) * 4 ft * $\cos^2$(28.52) * tan(33) } / [120 pcf * 4 ft * sin(28.52) * cos(28.52)]
0.573631

Reference:
Blake et al, 2002, Guidelines for Analyzing and Mitigating Landslide Hazards in California.
(Equation is as follows:)

$$F = \frac{c' + (\gamma - m\gamma_w) z \cos^2 B \tan\phi'}{\gamma z \sin B \cos B} \qquad (9.5)$$

where F is the factor of safety, c' is the cohesion intercept, z is the vertical depth of the slip surface, B is the slope angle, γ is the unit weight (density) of the soil, γ_w is the unit weight of the water, m is the fraction of z such that mz is the vertical height of the groundwater table above the slip surface, and ϕ' is the angle of shearing resistance.