1497 West 40 South
Lindon, Utah - 84042
Phone (801) 225-5711

3662 West 2100 South
Salt Lake City, Utah - 84120
Phone (801) 787-9138

1596 W. 2650 S. #108
Ogden, Utah - 84401
Phone (801) 399-9516

Geotechnical Study
Hidden Cove Subdivision
6260 South 2125 East
Ogden, Utah

Project No. 177078

December 4, 2017

Prepared For:

Blue Mountain Homes, LLC
Attention: Mr. Donald Fulton
P.O. Box 65999
Salt Lake City, UT 84165

Prepared By:

EARTHTEC ENGINEERING
Ogden Office

TABLE OF CONTENTS

TABLE OF CONTENTS (CONTINUED)

*Earth Tech, LLC d.b.a. Earthtec Engineering (Earth Tech, LLC) is a separate business entity, and has no relation to Earthtec Testing & Engineering, P.C.. Earth Tech, LLC assumes no liability or responsibility over the contents contained within Earthtec Testing & Engineering, P.C.'s report. Earth Tech, LLC did not review, confirm, or verify any portion of the attached reports and makes no assurances to the completeness or correctness of their findings. The findings, conclusions, and recommendations presented in the attached reports were provided by others and Earth Tech, LLC does not make any warrantee, guarantee, or representation presented in the reports included in the appendix.

Appendix A includes previously completed reports by other companies provided to Earthtec Engineering by Mr. Donald Fulton. The reports are included in this appendix as requested by Weber County. These reports are not used or referenced in Earth Tech, LLC's Geotechnical Study Job No. 177078, dated December 4, 2017.

1.0 EXECUTIVE SUMMARY

This entire report presents the results of Earthtec Engineering's completed geotechnical study for the Hidden Cove Subdivision in Ogden, Utah. This executive summary provides a general synopsis of our recommendations and findings. Details of our findings, conclusions, and recommendations are provided within the body of this report.

- The subject property is approximately 3.38 acres and is proposed to be developed with three residential lots. The proposed structures will consist of conventionally framed two-story, houses. We anticipate foundation loads for the proposed structures will not exceed 3,000 pounds per linear foot for bearing wall, 20,000 pounds for column loads, and 100 pounds per square foot for floor slabs. (see Section 3)
- Our field exploration included the boring of two (2) test pits to depths of 12 feet below the existing ground surface. Groundwater was not encountered within the excavations to the depths explored. (see Section 5)
- The subsurface soils encountered generally consisted of topsoil overlying near-surface loose to dense sand and gravel. All topsoil should be removed beneath the entire building footprints, exterior flatwork, and pavements prior to construction. (see Section 7)
- Conventional strip and spread footings may be used to support the structure, with foundations placed entirely on a minimum of 24 inches of properly placed, compacted, and tested structural fill extending to undisturbed native soils. (see Sections 10 and 14)
- Minimum roadway section consists of 3 inches of asphalt over 6 inches of road-base. Areas that are soft or deflect under construction traffic should be removed and replaced with granular material or structural fill. (see Section 13)

Based on the results of our field exploration, laboratory testing, and engineering analyses, it is our opinion that the subject site may be suitable for the proposed development, provided the recommendations presented in this report are followed and implemented during design and construction.

Failure to consult with Earthtec Engineering (Earthtec) regarding any changes made during design and/or construction of the project from those discussed herein relieves Earthtec from any liability arising from changed conditions at the site. We also strongly recommend that Earthtec observes the building excavations to verify the adequacy of our recommendations presented herein, and that Earthtec performs materials testing and special inspections for this project to provide continuity during construction.

2.0 INTRODUCTION

The project is located at approximately 6260 South 2125 East in Ogden, Utah. The general location of the site is shown on Figure No. 1, *Vicinity Map* and Figure No. 2, *Aerial Photograph Showing Location of Test Pits*, at the end of this report. The purposes of this study are to:

- Evaluate the subsurface soil conditions at the site,
- Assess the engineering characteristics of the subsurface soils, and
- Provide geotechnical recommendations for general site grading and the design and construction of foundations, concrete floor slabs, miscellaneous concrete flatwork, and asphalt paved residential streets.

The scope of work completed for this study included field reconnaissance, subsurface exploration, field and laboratory soil testing, geotechnical engineering analysis, and the preparation of this report.

3.0 PROPOSED CONSTRUCTION

We understand that the proposed project, as described to us by Mr. Donald Fulton of Blue Mountain Homes, LLC, consists of developing the approximately 3.38-acre existing parcel into 3 residential lots and an associated residential street. The proposed structures will consist of conventionally framed two-story, houses. We have based our recommendations in this report on the assumption that or anticipated foundation loads for the proposed structures will not exceed 3,000 pounds per linear foot for bearing wall, 20,000 pounds for column loads, and 100 pounds per square foot for floor slabs. If structural loads will be greater Earthtec should be notified so that we may review our recommendations and make modifications, if necessary.

In addition to the construction described above, we anticipate that

- Utilities will be installed to service the proposed buildings,
- Exterior concrete flatwork will be placed in the form of curb, gutter, and sidewalks, and
- An asphalt paved residential street will be constructed.

4.0 GENERAL SITE DESCRIPTION

4.1 Site Description

At the time of our subsurface exploration the site was an undeveloped lot vegetated with grass, weeds, and scrub oak. The subject site is located on a southeast facing slope with an access dirt road from Jared Road on the northwest, and several dirt roads on the south. The ground surface appears to slope more than 15 percent grade, we anticipate up to 6 of cut and fill may be required for site grading. The site was bounded on the north, east, and west by developed residential lots, and on the south by undeveloped land and the State Highway 89.

4.2 Geologic Setting

The subject property is located near the eastern shore of the Great Salt Lake in a valley between the Great Salt Lake Basin and the Wasatch Mountain Range. The valley and Great Salt Lake Basin were formed by extensional tectonics during the Tertiary and Quaternary geologic periods. The valley and Great Salt Lake Basin, and much of western Utah, were previously covered by Lake Bonneville, a large, Pleistocene age, fresh water lake that reached a high-stand surface elevation of approximately 5,170 feet above sea level. The Great Salt Lake is a remnant of Lake Bonneville. The valleys and lake basin to the west of the Wasatch Range have been partially filled with several thousand feet of lake (lacustrine) sediment during Lake Bonneville time, and post-Bonneville (Holocene) deltaic, lacustrine, alluvial, and colluvial deposits. The Wasatch Mountains to the east of the subject property are comprised of the early Proterozoic Farmington Canyon Complex consisting primarily of schist and gneiss. The subject site is located just below the Provo Shoreline level of Lake Bonneville Shoreline level of Lake Bonneville in an area where lacustrine deposits have been dissected by downcutting and erosion of gullies. The surficial geology at the location of the subject site has been mapped as "ldp: Deltaic deposits related to regressive phase (uppermost Pleistocene)-Clast-supported pebble and cobble gravel interbedded with this sand bed, and matrix-supported gravelly sands; moderate to well sorted, clasts subround to round, with weak carbonate cementation common. Deposited as foreset beds with original dips of 30-35 degrees, Commonly capped with <5m of topset alluvium (unit alp), which is less well sorted, silty to sandy, pebble and gravel. Mapped at the mouth of north Ogden, Ogden, Weber, and Ward canyons, and the canyon of Mill Creek" by Nelson and Personius (1993)[1].

A geotechnical/geologic study[2] was performed at the subject site by Earthtec Testing & Engineering, P.C. in 2003. The report has recommended a minimum of 30 feet setback from the toe of slopes where grade measures 20 percent or steeper for structures during the future development, due to potential mass movement of the slopes. The recommendations in the above-referenced report should be followed for the new development. A copy of the report is maintained in our files and can be provided upon request.

5.0 SUBSURFACE EXPLORATION

5.1 Soil Exploration

Under the direction of a qualified member of our geotechnical staff, subsurface explorations

---

[1] U.S. Geological Survey Map I-2199: Surficial Geologic Map of the Weber Segment, Wasatch Fault Zone, Weber and Davis Counties, Utah, by Nelson, A. R., and Personius, S. F., 1993.

[2] Geotechnical/Geological Study, Kunzler Subdivision, 6260 South 2125 East, Weber County, Utah, by Earthtec Testing & Engineering, P.C., ETE Job No. 03E-064, February 4, 2003

were conducted at the site on November 16, 2017 by the excavation of two (2) test pits to depths of 12 feet below the existing ground surface using a a rubber-tire backhoe. The approximate locations of the test pits are shown on Figure No. 2, *Aerial Photograph Showing Location of Test Pits.* Graphical representations and detailed descriptions of the soils encountered are shown on Figure Nos. 3 and 4, *Test Pit Log* at the end of this report. The stratification lines shown on the logs represent the approximate boundary between soil units; the actual transition may be gradual. Due to potential natural variations inherent in soil deposits, care should be taken in interpolating between and extrapolating beyond exploration points. A key to the symbols and terms on the logs is presented on Figure No. 5, *Legend.*

Disturbed bag samples were collected at various depths in each test pit. The soil samples collected were classified by visual examination in the field following the guidelines of the Unified Soil Classification System (USCS). The samples were transported to our Ogden, Utah laboratory where they will be retained for 30 days following the date of this report and then discarded, unless a written request for additional holding time is received prior to the 30-day limit.

6.0 LABORATORY TESTING

Representative soil samples collected during our field exploration were tested in the laboratory to assess pertinent engineering properties and to aid in refining field classifications, if needed. Tests performed included natural moisture content, dry density tests, liquid and plastic limits determinations, mechanical (partial) gradation analyses, a one-dimensional consolidation test, and a direct shear test. The table below summarizes the laboratory test results, which are also included on the attached *Test Pit Logs* at the respective sample depths, on Figure Nos. 3 and 4, *Consolidation-Swell Test,* and *Direct Shear Test,* on Figure No. 6.

Table 1: Laboratory Test Results

Test Pit No.	Depth (ft.)	Natural Moisture (%)	Natural Dry Density (pcf)	Atterberg Limits		Grain Size Distribution (%)			Soil Type
				Liquid Limit	Plasticity Index	Gravel (+ #4)	Sand	Silt/Clay (- #200)	
1	6	3	105	22	NP*	13	66	21	SM
1	8	5	---	---	---	11	73	16	SM
2	4	4	---	---	---	41	40	19	GM

NP* = Non-Plastic

As part of the consolidation test procedure, water was added to a sample to assess moisture sensitivity when the sample was loaded to an equivalent pressure of approximately 1,000 psf. The native sand soils have a slight potential for collapse (settlement) and a slight potential for compressibility under increased moisture contents and anticipated load conditions.

7.0 SUBSURFACE CONDITIONS

7.1 Soil Types

On the surface of the site, we encountered topsoil which is estimated to extend about one foot in depth at the test pit locations. Below the topsoil we encountered layers of sand and gravel extending to depths of 12 feet below the existing ground surface. Graphical representations and detailed descriptions of the soils encountered are shown on Figure Nos. 3 and 4, *Test Pit Log* at the end of this report. Based on our experience and observations during field exploration, the consistency and the sand and gravel soils visually had a relative density varying from loose to dense. Variation in topsoil depths may occur at the site.

7.2 Groundwater Conditions

Groundwater was not encountered within the excavations to the depths explored. Note that groundwater levels will fluctuate in response to the season, precipitation, snow melt, irrigation, and other on and off-site influences. Quantifying these fluctuations would require long term monitoring, which is beyond the scope of this study. The contractor should be prepared to dewater excavations as needed.

8.0 SITE GRADING

8.1 General Site Grading

All surface vegetation and unsuitable soils (such as topsoil, organic soils, undocumented fill, soft, loose, or disturbed native soils, and any other inapt materials) should be removed from below foundations, floor slabs, exterior concrete flatwork, and pavement areas. We encountered topsoil on the surface of the site. The topsoil (including soil with roots larger than about ¼ inch in diameter) should be completely removed, even if found to extend deeper, along with any other unsuitable soils that may be encountered. Over-excavations below footings and slabs also may be needed, as discussed in Section 10.0.

Fill placed over large areas, even if only a few feet in depth, can cause consolidation in the underlying native soils resulting in settlement of the fill. Because there is more than 100 feet of relief from northwest to the southeast, we anticipate that more than 6 feet of fill may be placed in some areas of the site during grading. If more than 6 feet of grading fill will be placed above the existing surface (to raise site grades), Earthtec should be notified so that we may provide additional recommendations, if required. Such recommendations will likely include placing the fill several weeks (or possibly more) prior to construction to allow settlement to occur.

8.2 Temporary Excavations

Temporary excavations that are less than 4 feet in depth and above groundwater should have side slopes no steeper than ½H:1V (Horizontal:Vertical). Temporary excavations where water is encountered in the upper 4 feet or that extend deeper than 4 feet below site grades should be sloped or braced in accordance with OSHA[2] requirements for Type C soils.

8.3 Fill Material Composition

The native soils within the upper 12 feet appear to be suitable for use as placed and compacted structural fill provided the material meets the requirements for structural fill and any existing debris and particles larger than 6 inches in diameter are removed prior to use. Excavated soils, including silt, may be stockpiled for use as fill in landscape areas.

Structural fill is defined as fill material that will ultimately be subjected to any kind of structural loading, such as those imposed by footings, floor slabs, pavements, etc. We recommend that a professional engineer or geologist verify that the structural fill to be used on this project meets the requirements, stated below. We recommend that structural fill consist of imported sandy/gravelly soils meeting the following requirements in the table below:

Table 2: Structural Fill Recommendations

Sieve Size/Other	Percent Passing (by weight)
4 inches	100
3/4 inches	70 – 100
No. 4	40 – 80
No. 40	15 – 50
No. 200	0 – 20
Liquid Limit	35 maximum
Plasticity Index	15 maximum

In some situations, particles larger than 4 inches and/or more than 30 percent coarse gravel may be acceptable, but would likely make compaction more difficult and/or significantly reduce the possibility of successful compaction testing. Consequently, stricter quality control measures than normally used may be required, such as using thinner lifts and increased or full-time observation of fill placement.

We recommend that utility trenches below any structural load be backfilled using structural fill. Note that most local governments and utility companies require Type A-1-a or A-1-b (AASHTO classification) soils (which overall is stricter than our recommendations for structural fill) be used as backfill above utilities in certain areas. In other areas or situations, utility trenches may be backfilled with the native soil, but the contractor should be aware that native soils (as observed in the explorations) may be time consuming to compact due to potential difficulties in controlling the moisture content needed to obtain optimum compaction. All backfill soil should have a maximum particle size of 4 inches, a maximum Liquid Limit of 35 and a maximum Plasticity Index of 15.

If required, we recommend that free draining granular material (clean sand and/or gravel) meet

---

[2] OSHA Health And Safety Standards, Final Rule, CFR 29, part 1926.

the following requirements in the table below:

Table 3: Free-Draining Fill Recommendations

Sieve Size/Other	Percent Passing (by weight)
3 inches	100
No. 10	0 – 25
No. 40	0 – 15
No. 200	0 – 5
Plasticity Index	Non-plastic

Three-inch minus washed rock (sometimes called river rock or drain rock) and pea gravel materials usually meet these requirements and may be used as free draining fill. If free draining fill will be placed adjacent to soil containing a significant amount of sand or silt/clay, precautions should be taken to prevent the migration of fine soil into the free draining fill. Such precautions should include either placing a filter fabric between the free draining fill and the adjacent soil material, or using a well-graded, clean filtering material approved by the geotechnical engineer.

8.4 Fill Placement and Compaction

Fill should be placed on level, horizontal surfaces. Where fill will be placed on existing slopes steeper than 5H:1V, the existing ground should be benched prior to placing fill. We recommend bench heights of 1 to 4 feet, with the lowest bench being a minimum 3 feet below adjacent grade and at least 10 feet wide.

The thickness of each lift should be appropriate for the compaction equipment that is used. We recommend a maximum lift thickness prior to compaction of 4 inches for hand operated equipment, 6 inches for most "trench compactors" and 8 inches for larger rollers, unless it can be demonstrated by in-place density tests that the required compaction can be obtained throughout a thicker lift. The full thickness of each lift of structural fill placed should be compacted to at least the following percentages of the maximum dry density, as determined by ASTM D-1557:

In landscape and other areas not below structurally-loaded areas:	90%
Less than 5 feet of fill below structurally loaded areas:	95%
Greater than 5 feet of fill below structurally loaded areas:	98%

Generally, placing and compacting fill at moisture contents within ±2 percent of the optimum moisture content, as determined by ASTM D-1557, will facilitate compaction. Typically, the further the moisture content deviates from optimum the more difficult it will be to achieve the required compaction.

Fill should be tested frequently during placement and we recommend early testing to demonstrate that placement and compaction methods are achieving the required compaction. The contractor is responsible to ensure that fill materials and compaction efforts are consistent so that tested areas are representative of the entire fill.

8.5 Stabilization Recommendations

Near surface layers of silty sand soils may rut and pump during grading and construction. The likelihood of rutting and/or pumping, and the depth of disturbance, is proportional to the moisture content in the soil, the load applied to the ground surface, and the frequency of the load. Consequently, rutting and pumping can be minimized by avoiding concentrated traffic, minimizing the load applied to the ground surface by using lighter equipment, partially loaded equipment, tracked equipment, by working in dry times of the year, and/or by providing a working surface for equipment.

During grading the soil in any obvious soft spots should be removed and replaced with granular material. If rutting or pumping occurs traffic should be stopped in the area of concern. The soil in rutted areas should be removed and replaced with granular material. In areas where pumping occurs the soil should either be allowed to sit until pore pressures dissipate (several hours to several days) and the soil firms up, or be removed and replaced with granular material. Typically, we recommend removal to a minimum depth of 24 inches.

For granular material, we recommend using angular well-graded gravel, such as pit run, or crushed rock with a maximum particle size of four inches. We suggest that the initial lift be approximately 12 inches thick and be compacted with a static roller-type compactor. A finer granular material such as sand, gravelly sand, sandy gravel or road base may also be used. Materials which are more angular and coarse may require thinner lifts in order to achieve compaction. We recommend that the fines content (percent passing the No. 200 sieve) be less than 15%, the liquid limit be less than 35, and the plasticity index be less than 15.

Using a geosynthetic fabric, such as Mirafi 600X or equivalent, may also reduce the amount of material required and avoid mixing of the granular material and the subgrade. If a fabric is used, following removal of disturbed soils and water, the fabric should be placed over the bottom and up the sides of the excavation a minimum of 24 inches. The fabric should be placed in accordance with the manufacturer's recommendations, including proper overlaps. The granular material should then be placed over the fabric in compacted lifts. Again, we suggest that the initial lift be approximately 12 inches thick and be compacted with a static roller-type compactor.

9.0 SEISMIC AND GEOLOGIC CONSIDERATIONS

9.1 Seismic Design

The residential structures should be designed in accordance with the 2015 International Residential Code (IRC). The IRC designates this area as a seismic design class D_2.

The site is located at approximately 41.149 degrees latitude and -111.926 degrees longitude from the approximate center of the site. The IRC site value for this property is 0.878g. The design spectral response acceleration parameters are given below.

Table 4: Design Acceleration for Short Period

S_S	F_a	Site Value (S_{DS})
		2/3 S_S*F_a
1.316g	1.00	0.878g

S_S = Mapped spectral acceleration for short periods
F_a = Site coefficient from Table 1613.3.3(1)
$S_{DS} = \frac{2}{3}S_{MS} = \frac{2}{3}(F_a \cdot S_s)$ = 5% damped design spectral response acceleration for short periods

9.2 Faulting

The subject property is located within the Intermountain Seismic Belt where the potential for active faulting and related earthquakes is present. Based upon published geologic maps[3], no active faults traverse through or immediately adjacent to the site and the site is not located within local fault study zones. The nearest mapped fault trace is the Wasatch Fault located about 1 mile east of the site.

9.3 Liquefaction Potential

According to current liquefaction maps[4] for Weber County, the site is located within an area designated as "Low to Moderate" in liquefaction potential. Liquefaction can occur when saturated subsurface soils below groundwater lose their inter-granular strength due to an increase in soil pore water pressures during a dynamic event such as an earthquake.

Loose, saturated sands are most susceptible to liquefaction, but some loose, saturated gravels and relatively sensitive silt to low-plasticity silty clay soils can also liquefy during a seismic event. Subsurface soils were composed of sand and gravel soils. The soils encountered at this project do not appear liquefiable, but the liquefaction susceptibility of underlying soils (deeper than our explorations) is not known and would require deeper explorations to quantify

10.0 FOUNDATIONS

10.1 General

The foundation recommendations presented in this report are based on the soil conditions encountered during our field exploration, the results of laboratory testing of samples of the native soils, the site grading recommendations presented in this report, and the foundation loading conditions presented in Section 3.0, *Proposed Construction*, of this report. If loading conditions and assumptions related to foundations are significantly different, Earthtec should be notified so that we can re-evaluate our design parameters and estimates (higher loads may cause more settlement), and to provide additional recommendations if necessary.

Conventional strip and spread footings may be used to support the proposed structures after

---

[3] U.S. Geological Survey, Quaternary Fault and Fold Database of the United States, November 3, 2010
[4] Utah Geological Survey, Liquefaction-Potential Map for a Part of Weber County, Utah, Public Information Series 28, August 1994

appropriate removals as outlined in Section 8.1. Foundations should not be installed on topsoil, undocumented fill, debris, combination soils, organic soils, frozen soil, or in ponded water. If foundation soils become disturbed during construction, they should be removed or compacted.

10.2 Strip/Spread Footings

We recommend that conventional strip and spread foundations be constructed entirely on a minimum of 24 inches of properly placed, compacted, and tested structural fill extending to undisturbed native soils. See Section 14 for further information regarding foundation support relating to the stability of the slope. For foundation design we recommend the following:

- Footings founded on a minimum 24 inches of structural fill may be designed using a maximum allowable bearing capacity of 1,500 pounds per square foot. The values for vertical foundation pressure can be increased by one-third for wind and seismic conditions per Section 1806.1 when used with the Alternative Basic Load Combinations found in Section 1605.3.2 of the 2015 International Building Code.
- Continuous and spot footings should be uniformly loaded and should have a minimum width of 20 and 30 inches, respectively.
- Exterior footings should be placed below frost depth which is determined by local building codes. In general, 30 inches of cover is adequate for most sites; however local code should be verified by the end design professional. Interior footings, not subject to frost (heated structures), should extend at least 18 inches below the lowest adjacent grade.
- Foundation walls and footings should be properly reinforced to resist all vertical and lateral loads and differential settlement.
- The bottom of footing excavations should be compacted with at least 4 passes of an approved non-vibratory roller prior to erection of forms or placement of structural fill to densify soils that may have been loosened during excavation and to identify soft spots. If soft areas are encountered, they should be stabilized as recommended in Section 8.5.
- Footing excavations should be observed by the geotechnical engineer prior to beginning footing construction to evaluate whether suitable bearing soils have been exposed and whether excavation bottoms are free of loose or disturbed soils.
- Structural fill used below foundations should extend laterally a minimum of 6 inches for every 12 vertical inches of structural fill placed. For example, if 18 inches of structural fill is required to bring the excavation to footing grade, the structural fill should extend laterally a minimum of 9 inches beyond the edge of the footings on both sides.

10.3 Estimated Settlements

If the proposed foundations are properly designed and constructed using the parameters provided above, we estimate that total settlements should not exceed one inch and differential settlements should be one-half of the total settlement over a 25-foot length of continuous foundation, for non-earthquake conditions. Additional settlement could occur during a seismic event due to ground shaking, if more than 3 feet of grading fill is placed above the existing ground surface, if loading conditions are greater than anticipated in Section 3, and/or if foundation soils are allowed to become wetted.

10.4 Lateral Earth Pressures

Below grade walls act as soil retaining structures and should be designed to resist pressures induced by the backfill soils. The lateral pressures imposed on a retaining structure are dependent on the rigidity of the structure and its ability to resist rotation. Most retaining walls that can rotate or move slightly will develop an active lateral earth pressure condition. Structures that are not allowed to rotate or move laterally, such as subgrade basement walls, will develop an at-rest lateral earth pressure condition. Lateral pressures applied to structures may be computed by multiplying the vertical depth of backfill material by the appropriate equivalent fluid density. Any surcharge loads in excess of the soil weight applied to the backfill should be multiplied by the appropriate lateral pressure coefficient and added to the soil pressure. For static conditions the resultant forces are applied at about one-third the wall height (measured from bottom of wall). For seismic conditions, the resultant forces are applied at about two-third times the height of the wall both measured from the bottom of the wall. The lateral pressures presented in the table below are based on drained, horizontally placed structural fill (as outlined in this report) as backfill material using a 35° friction angle and a dry unit weight of 108 pcf.

Table 5: Lateral Earth Pressures (Static and Dynamic)

Condition	Case	Lateral Pressure Coefficient	Equivalent Fluid Pressure (pcf)
Active	Static	0.27	29
	Seismic	0.47	50
At-Rest	Static	0.43	46
	Seismic	0.70	76
Passive	Static	3.69	399
	Seismic	5.63	608

*Seismic values combine the static and dynamic values

These pressure values do not include any surcharge, and are based on a relatively level ground surface at the top of the wall and drained conditions behind the wall. It is important that water is not allowed to build up (hydrostatic pressures) behind retaining structures. Retaining walls should incorporate drainage behind the walls as appropriate, and surface water should be directed away from the top and bottom of the walls.

Lateral loads are typically resisted by friction between the underlying soil and footing bottoms. Resistance to sliding may incorporate the friction acting along the base of foundations, which may be computed using a coefficient of friction of soils against concrete of 0.55 for native gravels or structural fill meeting the recommendations presented herein. Concrete or masonry walls shall be selected and constructed in accordance to the provision of Section R404 of the 2015 International Residential Code or sections referenced therein. Retaining wall lateral resistance design should further reference Section R404.4 for reference of Safety Factors.

The pressure and coefficient values presented above are ultimate; therefore, an appropriate factor of safety may need to be applied to these values for design purposes. The appropriate factor of safety will depend on the design condition and should be determined by the project structural engineer.

11.0 FLOOR SLABS AND FLATWORK

Concrete floor slabs and exterior flatwork may be supported on 6 inches of properly placed and compacted structural fill after appropriate removals and grading as outlined in Section 8.1 are completed. We recommend placing a minimum 4 inches of free-draining fill material (see Section 8.3) beneath floor slabs to facilitate construction, act as a capillary break, and aid in distributing floor loads. For exterior flatwork, we recommend placing a minimum 4 inches of road-base material. Prior to placing the free-draining fill or road-base materials, the native sub-grade should be proof-rolled to identify soft spots, which should be stabilized as discussed above in Section 8.5.

For slab design, we recommend using a modulus of sub-grade reaction of 135 pounds per cubic inch. The thickness of slabs supported directly on the ground shall not be less than 3½ inches. A 6-mil polyethylene vapor retarder with joints lapped not less than 6 inches shall be placed between the ground surface and the concrete, as per Section R506 of the 2015 International Residential Code.

To help control normal shrinkage and stress cracking, we recommend that floor slabs have adequate reinforcement for the anticipated floor loads with the reinforcement continuous through interior floor joints, frequent crack control joints, and non-rigid attachment of the slabs to foundation and bearing walls. Special precautions should be taken during placement and curing of all concrete slabs and flatwork. Excessive slump (high water-cement ratios) of the concrete and/or improper finishing and curing procedures used during hot or cold weather conditions may lead to excessive shrinkage, cracking, spalling, or curling of slabs. We recommend all concrete placement and curing operations be performed in accordance with American Concrete Institute (ACI) codes and practices.

12.0 DRAINAGE

12.1 Surface Drainage

As part of good construction practice, precautions should be taken during and after construction to reduce the potential for water to collect near foundation walls. Accordingly, we recommend the following:

- The contractor should take precautions to prevent significant wetting of the soil at the base of the excavation. Such precautions may include: grading to prevent runoff from entering the excavation, excavating during normally dry times of the year, covering the base of the excavation if significant rain or snow is forecast, backfill at the earliest possible date, frame floors and/or the roof at the earliest possible date, other precautions that might become evident during construction.
- Adequate compaction of foundation wall backfill should be provided i.e. a minimum of 90% of ASTM D-1557. Water consolidation methods should not be used.
- The ground surface should be graded to drain away from the building in all directions. We recommend a minimum fall of 6 inches in the first 10 feet.
- Roof runoff should be collected in rain gutters with down spouts designed to discharge well outside of the backfill limits, or at least 10 feet from foundations, whichever is greater.
- Sprinkler nozzles should be aimed away, and all sprinkler components kept at least 5 feet, from foundation walls. A drip irrigation system may be utilized in landscaping areas within 10 feet of foundation walls. Also, sprinklers should not be placed at the top or on the face of slopes. Sprinkler systems should be designed with proper drainage and well maintained. Over-watering should be avoided.
- Any additional precautions which may become evident during construction.

12.2 Subsurface Drainage

Groundwater was not encountered during our field exploration; thus, it is our opinion that perimeter foundation drains are not needed for this project. However, if foundation drains are constructed for the proposed homes, the recommendations presented below should be followed during design and construction of the foundation drains.

Section R405.1 of the 2015 International Residential Code states, "Drains shall be provided around all concrete and masonry foundations that retain earth and enclose habitable or usable spaces located below grade." Section R310.2.3.2 of the 2015 International Residential Code states, "Window wells shall be designed for proper drainage by connecting to the building's foundation drainage system." An exception is allowed when the foundation is installed on well drained ground consisting of Group 1 soils, which include those defined by the Unified Soil Classification System as GW, GP, SW, SP, GM, and SM. The soils observed in the explorations at the depth of foundation consisted primarily of Silty Sand (SM) which is not a Group 1 soil. The recommendations presented below should be followed during design and construction of the foundation drains:

- A perforated 4-inch minimum diameter pipe should be enveloped in at least 12 inches of free-draining gravel and placed adjacent to the perimeter footings. The perforations should be oriented such that they are not located on the bottom side of the pipe, as much as possible. The free-draining gravel should consist of primarily ¾- to 2-inch size gravel having less than 5 percent passing the No. 4 sieve, and should be wrapped with a separation fabric such as Mirafi 140N or equivalent.

- The highest point of the perforated pipe bottom should be equal to the bottom elevation of the footings. The pipe should be uniformly graded to drain to an appropriate outlet (storm drain, land drain, other gravity outlet, etc.) or to one or more sumps where water can be removed by pumping.
- A perforated 4-inch minimum diameter pipe should be installed in all window wells and connected to the foundation drain.
- To facilitate drainage beneath basement floor slabs we recommend that the minimum thickness of free-draining fill beneath the slabs be increased to at least 10 inches (approximately equal to the bottom of footing elevations). A separation fabric such as Mirafi 140N or equivalent should be placed beneath the free-draining gravel. Connections should be made to allow any water beneath the slabs to reach the perimeter foundation drain.
- The drain system should be periodically inspected and clean-outs should be installed for the foundation drain to allow occasional cleaning/purging, as needed. Proper drain operation depends on proper construction and maintenance.

13.0 PAVEMENT RECOMMENDATIONS

We understand that an asphalt paved residential street will be constructed as part of the development. The native soils encountered beneath the fill and topsoil during our field exploration were predominantly composed of sands. We estimate that a California Bearing Ratio (CBR) value of 5 is appropriate for these soils. If the fill material and topsoil is left beneath concrete flatwork and pavement areas, increased maintenance costs over time should be anticipated.

We anticipate that the traffic volume will be about 200 vehicles a day or less for the residential streets, consisting of mostly cars and pickup trucks, with a daily delivery truck and a weekly garbage truck. Based on these traffic parameters, the estimated CBR given above, and the procedures and typical design inputs outlined in the UDOT Pavement Design Manual (1998), we recommend the minimum asphalt pavement section presented below.

Table 6: Pavement Section Recommendations

Asphalt Thickness (in)	Compacted Roadbase Thickness (in)	Compacted Subbase Thickness (in)
3	6*	0

* Stabilization may be required

If the pavement will be required to support construction traffic, more than an occasional semi-tractor or fire truck, or more traffic than listed above, our office should be notified so that we can re-evaluate the pavement section recommendations.

14.0 SLOPE STABILITY

We evaluated the overall stability by evaluating slope two cross-section for the proposed slope at the subject property. The properties of the native soils at the site were estimated using laboratory testing on samples recovered during our field investigations and our experience with similar soils. The Bureau of Reclamation5, estimates silty sand soils have an internal friction angle between 33 and 35 degrees. Our direct shear testing on the native silty sand (SM) the soils encountered during our field investigation indicated the soils have an internal friction angle of about 35 degrees and cohesion of about 175 psf (See Figure No. 7, Direct Shear Test). Completed direct shear test result from a previous study indicated that native silt soils have internal friction angle of about 26 degrees and cohesion of about 186 psf. For the soil parameters used in the slope stability analysis see the table below.

Table 7: Soil Parameters

Material	Internal Friction Angle (degrees)	Apparent Cohesion (psf)	Moist Unit Weight (pcf)	Saturated Unit Weight (pcf)
Silty Sand (SM)	35	175	120	130
Silt (ML)	26	185	102	120
Silty Gravel (GM)	35	150	125	130

For the seismic (pseudostatic) analysis, a peak horizontal ground acceleration of 0.586g for the 2% probability of exceedance in 50 years was obtained for site (grid) locations of 41.149 degrees north latitude and -111.926 degrees west longitude. Typically, one-third to one-half this value is utilized in analysis. Accordingly, a value of 0.195 was used as the pseudostatic coefficient for the stability analysis.

We evaluated the stability of the site with the proposed grading elevations using the computer program XSTABLE. This program uses a limit equilibrium (Bishop's modified) 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. The configuration analyzed was based on our observations during the field investigation and the provided site plan. The site plan was provided by the client, Mr. Donald Fulton. See Figure No. 8, *Slope Cross-Section*, for location of slopes analyzed.

---

[5] US Bureau of Reclamation, 1987, "Design Standards No. 13, Embankment Dams, Denver Colorado"

The configuration of the proposed slope was analyzed at Cross-Section A-A' and starts near the south side of the cul-de-sac and goes uphill to the northwest through Lot 1-R. The configuration of the proposed slope was analyzed at Cross-Section B-B' and starts near the east side of the cul-de-sac and goes uphill through Lot 3-R nearly parallel to the Lot 2-R property line. A water table was conservatively placed at approximately 15 feet below the ground surface with signs of the past groundwater at approximately this depth. 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 slope configuration described for Cross-Section A-A' meets both these requirements. Cross-Section B-B' does not meets both these requirements and will require further modifications to the slope, other than those detailed on Figure No. 8, to satisfy minimum factors of safety. The slope stability data is attached as Figure Nos. 9 through 12, *Stability Results*. Any modifications to the slope, including the construction of residence and retaining walls, should be properly designed and engineered.

It should be clearly understood that slope movements or even failure can occur if the slope is undermined, overloaded, or the slope soils become saturated. The silt layer encountered at the site is a likely slip plane which may cause for slope movements or even failure. An engineered solution that reinforces the slope may provide the required factors of safety for the residential construction. Further investigation including a deeper boring may be required to determine slope stability based on the residence locations, the depth of the silt slip plane, and if the final grade placed has provided a stable slope. 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 soils, all recommendation for place fill under the roadway should be keyed into the native soils and placed as recommended in Section 8 of this report. Surface water should be directed away from the top and bottom of the slope, the slope should be vegetated with drought resistant plants, and sprinklers should not be placed on the face of the slope. Watering of landscape areas should be limited to reduce the amount of water introduced into the slope. Overwatering should be avoided. Any broken or leaking pipes should be fixed immediately.

15.0 GENERAL CONDITIONS

The exploratory data presented in this report was collected to provide geotechnical design recommendations for this project. The explorations may not be indicative of subsurface conditions outside the study area or between points explored and thus have a limited value in depicting subsurface conditions for contractor bidding. Variations from the conditions portrayed in the explorations may occur and which may be sufficient to require modifications in the design. If during construction, conditions are different than presented in this report, Earthtec should be advised immediately so that the appropriate modifications can be made.

The findings and recommendations presented in this geotechnical report were prepared in accordance with generally accepted geotechnical engineering principles and practice in this area of Utah at this time. No warranty or representation is intended in our proposals, contracts, letters, or reports.

This geotechnical report is based on relatively limited subsurface explorations and laboratory testing. Subsurface conditions may differ in some locations of the site from those described herein, which may require additional analyses and possibly modified recommendations. Thus we strongly recommend consulting with Earthtec regarding any changes made during design and construction of the project from those discussed herein. Failure to consult with Earthtec regarding any such changes relieves Earthtec from any liability arising from changed conditions at the site.

To maintain continuity, Earthtec should also perform materials testing and special inspections for this project. The recommendations presented herein are based on the assumption that an adequate program of tests and observations will be followed during construction to verify compliance with our recommendations. We also assume that we will review the project plans and specifications to verify that our conclusions and recommendations are incorporated and remain appropriate (based on the actual design). Earthtec should be retained to review the final design plans and specifications so comments can be made regarding interpretation and implementation of our geotechnical recommendations in the design and specifications. Earthtec also should be retained to provide observation and testing services during grading, excavation, foundation construction, and other earth-related construction phases of the project.

We appreciate the opportunity of providing our services on this project. If we can answer questions or be of further service, please contact Earthtec at your convenience.

Respectfully;

EARTHTEC ENGINEERING

Frank F.

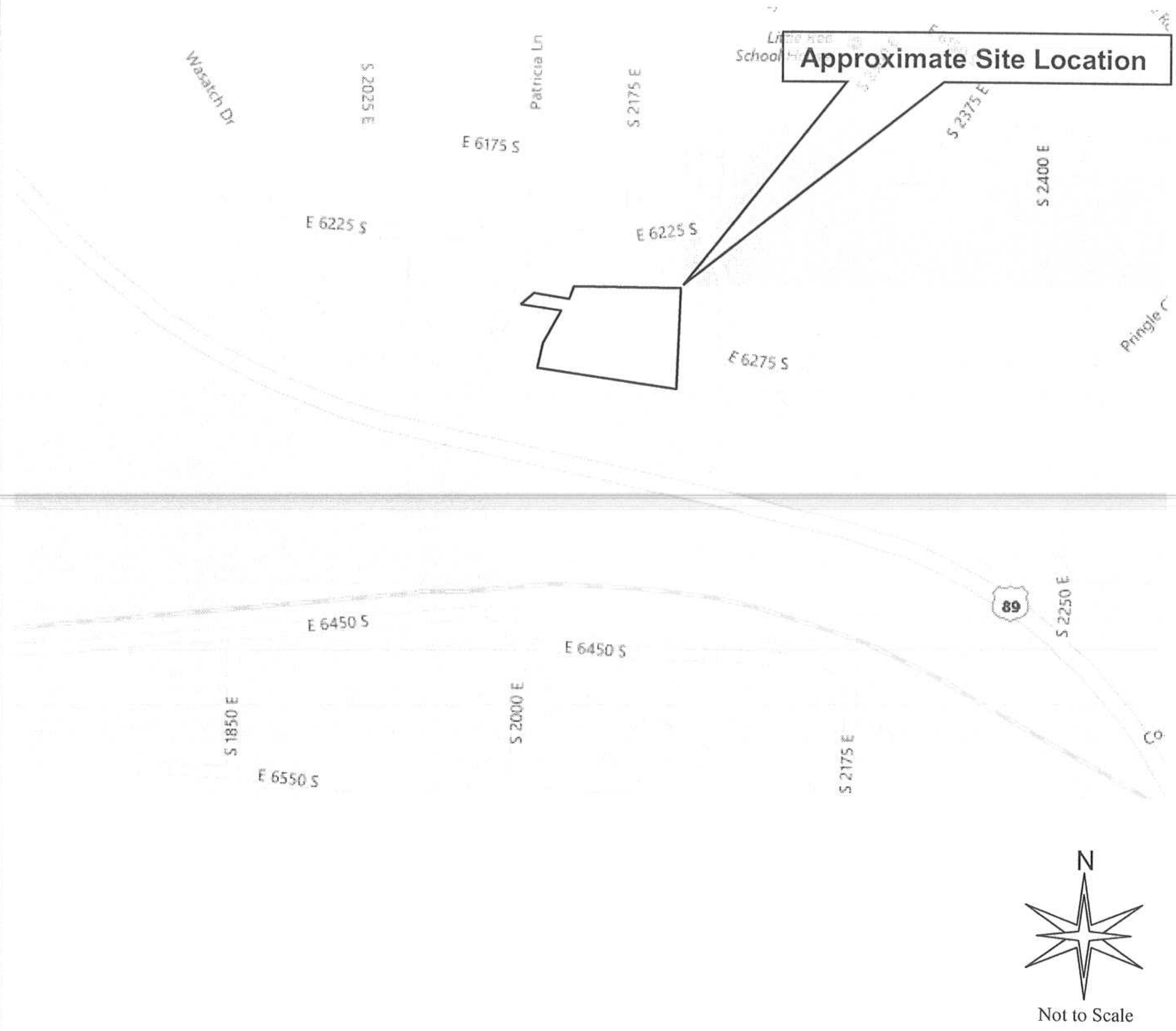
Frank Namdar, P.G., E.I.T.
Project Engineer

No. 5567049-2202
TIMOTHY ALLAN
MITCHELL
12/4/17

Timothy A. Mitchell, P.E.
Senior Geotechnical Engineer

VICINITY MAP

HIDDEN COVE SUBDIVISION
6260 SOUTH 2125 EAST
OGDEN, UTAH



PROJECT NO.: 177078	EARTHTEC ENGINEERING	**FIGURE NO.:** 1

AERIAL PHOTOGRAPH SHOWING LOCATION OF TEST PITS

HIDDEN COVE SUBDIVISION
6260 SOUTH 2125 EAST
OGDEN, UTAH

☒ Approximate Test Pit Location

— Approximate Site Perimeter

PROJECT NO.: 177078	EARTHTEC ENGINEERING	FIGURE NO.: 2

TEST PIT LOG

NO.: TP-1

PROJECT: Hidden Cove Subdivision
CLIENT: Blue Mountain Homes, LLC
LOCATION: See Figure 2
OPERATOR: C.E. Butters Construction
EQUIPMENT: Rubber-tire Backhoe
DEPTH TO WATER; INITIAL ▽ :

PROJECT NO.: 177078
DATE: 11/16/17
ELEVATION: Not Determined
LOGGED BY: F. Namdar
AT COMPLETION ▼ :

Depth (Ft.)	Graphic Log	USCS	Description	Samples	TEST RESULTS							
					Water Cont. (%)	Dry Dens. (pcf)	LL	PI	Gravel (%)	Sand (%)	Fines (%)	Other Tests
0			TOPSOIL, silty sand, slightly moist, brown, organics									
1			Silty SAND, loose to medium dense (estimated), slightly moist, light brown									
2				☒								
3												
4				☒								
5												
6		SM		☒	3	105	22	NP	13	66	21	C
7												
8				☒	5				11	73	16	DS
9												
10				☒								
11												
12			MAXIMUM DEPTH EXPLORED APPROXIMATELY 12 FEET	☒								
13												
14												
15												

Notes: No groundwater encountered.

Tests Key
CBR = California Bearing Ratio
C = Consolidation
R = Resistivity
DS = Direct Shear
SS = Soluble Sulfates
B = Burnoff

PROJECT NO.: 177078

EARTHTEC ENGINEERING

FIGURE NO.: 3

TEST PIT LOG

NO.: TP-2

PROJECT: Hidden Cove Subdivision
CLIENT: Blue Mountain Homes, LLC
LOCATION: See Figure 2
OPERATOR: C.E. Butters Construction
EQUIPMENT: Rubber-tire Backhoe
DEPTH TO WATER; INITIAL ▽ :

PROJECT NO.: 177078
DATE: 11/16/17
ELEVATION: Not Determined
LOGGED BY: F. Namdar

AT COMPLETION ▼ :

Depth (Ft.)	Graphic Log	USCS	Description	Samples	Water Cont. (%)	Dry Dens. (pcf)	LL	PI	Gravel (%)	Sand (%)	Fines (%)	Other Tests
0			TOPSOIL, silty sand, slightly moist, brown, organics									
1		SM	Silty SAND, loose to medium dense (estimated), slightly moist, reddish brown									
2		GM	Silty GRAVEL with sand, dense (estimated), slightly moist, light brown, some gravel and cobbles	☒								
3												
4				☒	4				41	40	19	
5												
6				☒								
7												
8				☒								
9												
10				☒								
11				☒								
12			MAXIMUM DEPTH EXPLORED APPROXIMATELY 12 FEET									
13												
14												
15												

Notes: No groundwater encountered.

Tests Key

CBR = California Bearing Ratio
C = Consolidation
R = Resistivity
DS = Direct Shear
SS = Soluble Sulfates
B = Burnoff

PROJECT NO.: 177078

EARTHTEC ENGINEERING

FIGURE NO.: 4

LEGEND

PROJECT: Hidden Cove Subdivision
CLIENT: Blue Mountain Homes, LLC
DATE: 11/16/17
LOGGED BY: F. Namdar

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR SOIL DIVISIONS			USCS SYMBOL	TYPICAL SOIL DESCRIPTIONS
COARSE GRAINED SOILS (More than 50% retaining on No. 200 Sieve)	GRAVELS (More than 50% of coarse fraction retained on No. 4 Sieve)	CLEAN GRAVELS (Less than 5% fines)	GW	Well Graded Gravel, May Contain Sand, Very Little Fines
			GP	Poorly Graded Gravel, May Contain Sand, Very Little Fines
		GRAVELS WITH FINES (More than 12% fines)	GM	Silty Gravel, May Contain Sand
			GC	Clayey Gravel, May Contain Sand
	SANDS (50% or more of coarse fraction passes No. 4 Sieve)	CLEAN SANDS (Less than 5% fines)	SW	Well Graded Sand, May Contain Gravel, Very Little Fines
			SP	Poorly Graded Sand, May Contain Gravel, Very Little Fines
		SANDS WITH FINES (More than 12% fines)	SM	Silty Sand, May Contain Gravel
			SC	Clayey Sand, May Contain Gravel
FINE GRAINED SOILS (More than 50% passing No. 200 Sieve)	SILTS AND CLAYS (Liquid Limit less than 50)		CL	Lean Clay, Inorganic, May Contain Gravel and/or Sand
			ML	Silt, Inorganic, May Contain Gravel and/or Sand
			OL	Organic Silt or Clay, May Contain Gravel and/or Sand
	SILTS AND CLAYS (Liquid Limit Greater than 50)		CH	Fat Clay, Inorganic, May Contain Gravel and/or Sand
			MH	Elastic Silt, Inorganic, May Contain Gravel and/or Sand
			OH	Organic Clay or Silt, May Contain Gravel and/or Sand
HIGHLY ORGANIC SOILS			PT	Peat, Primarily Organic Matter

SAMPLER DESCRIPTIONS

◪ SPLIT SPOON SAMPLER
(1 3/8 inch inside diameter)

⧗ MODIFIED CALIFORNIA SAMPLER
(2 inch outside diameter)

▥ SHELBY TUBE
(3 inch outside diameter)

◧ BLOCK SAMPLE

☒ BAG/BULK SAMPLE

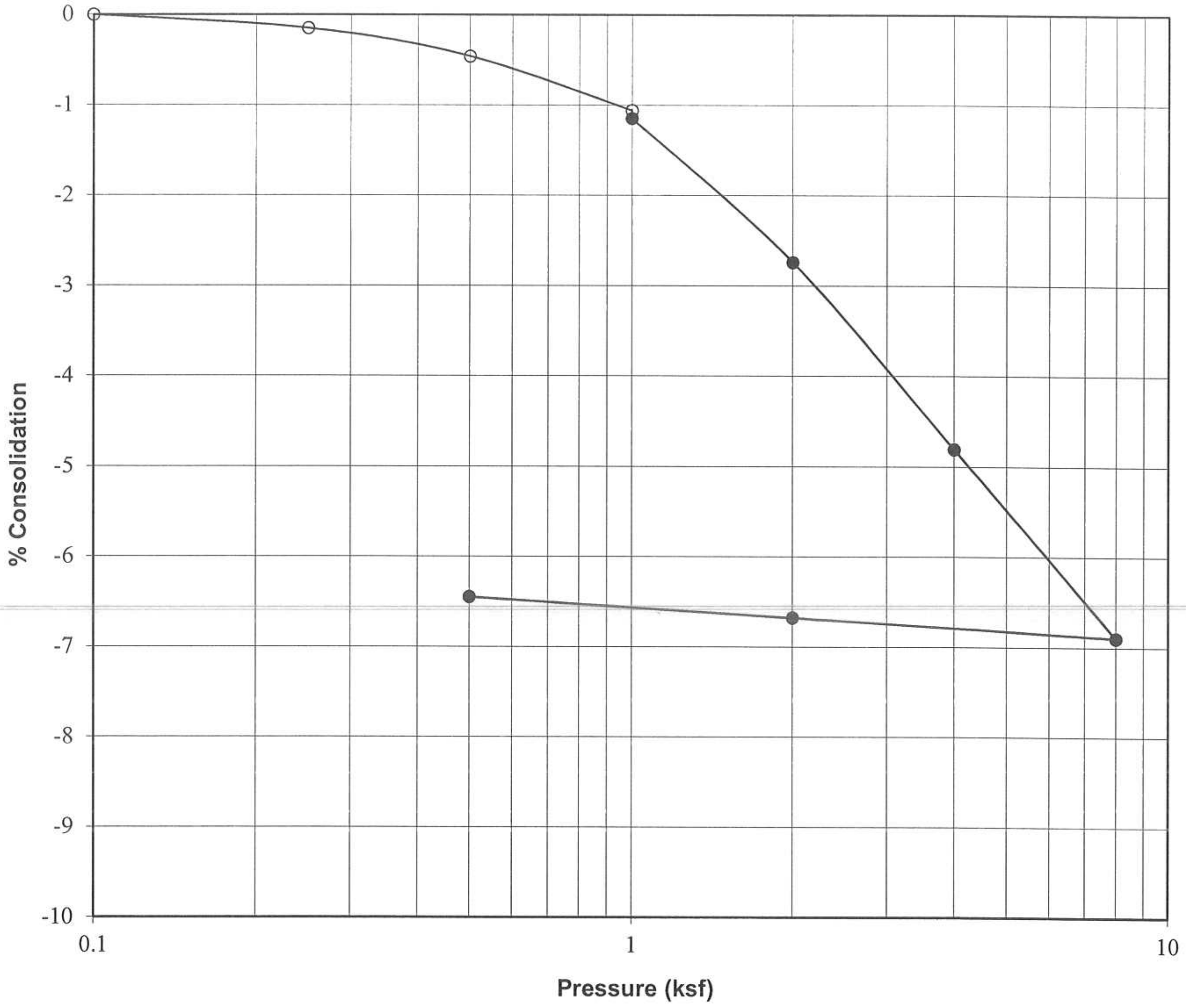
WATER SYMBOLS

∇ Water level encountered during field exploration

▼ Water level encountered at completion of field exploration

NOTES:
1. The logs are subject to the limitations, conclusions, and recommendations in this report.
2. Results of tests conducted on samples recovered are reported on the logs and any applicable graphs.
3. Strata lines on the logs represent approximate boundaries only. Actual transitions may be gradual.
4. In general, USCS symbols shown on the logs are based on visual methods only: actual designations (based on laboratory tests) may vary.

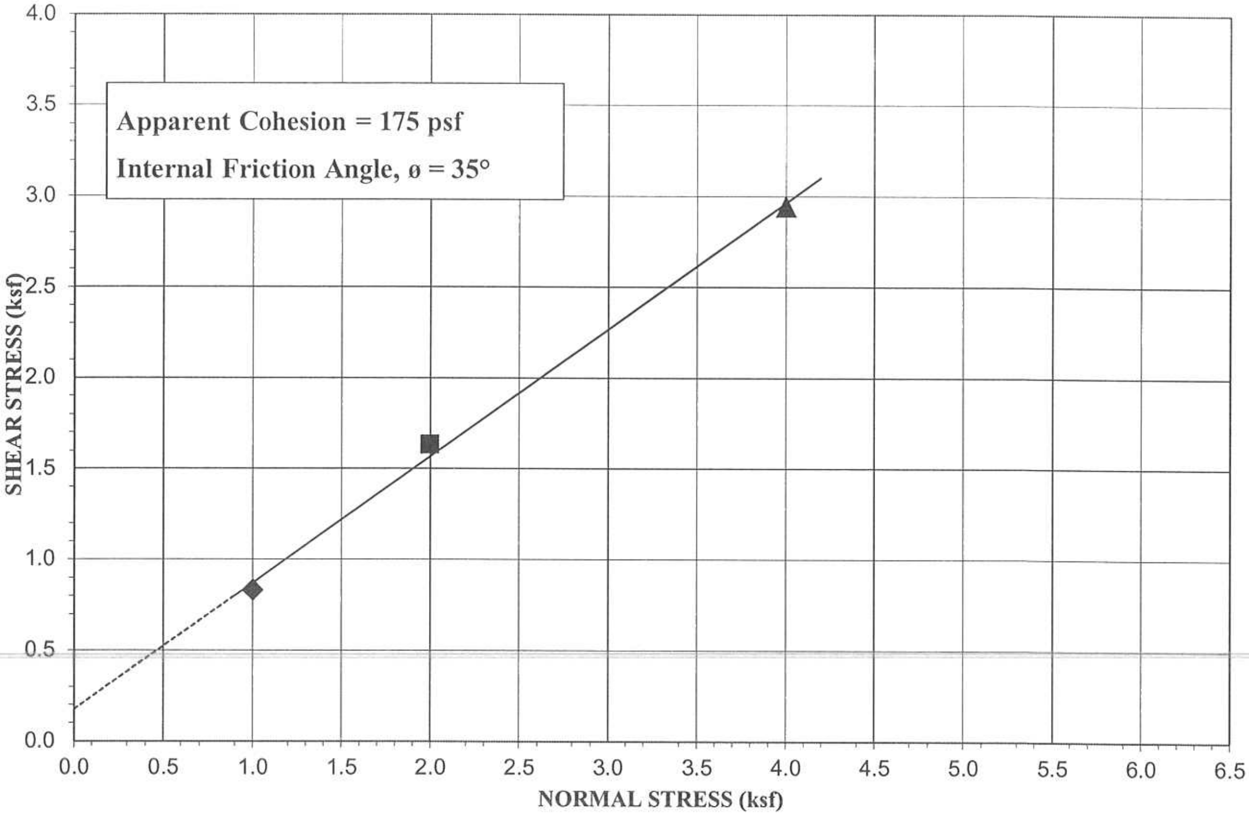
CONSOLIDATION - SWELL TEST

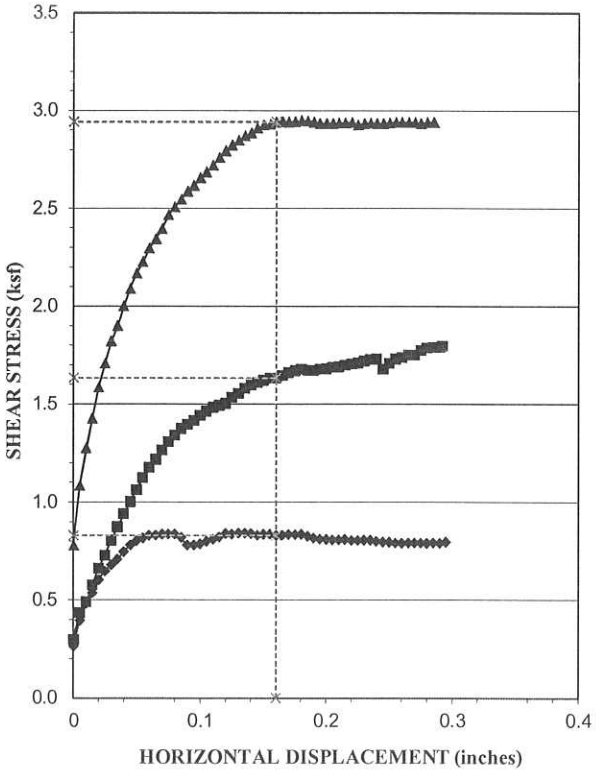


Project:	Hidden Cove Subdivision
Location:	TP-1
Sample Depth, ft:	6
Description:	Bag
Soil Type:	Silty SAND (SM)
Natural Moisture, %:	3
Dry Density, pcf:	105
Liquid Limit:	22
Plasticity Index:	NP
Water Added at:	1 ksf
Percent Collapse:	0.1

PROJECT NO.: 177078	EARTHTEC ENGINEERING	**FIGURE NO.:** 6

DIRECT SHEAR TEST





Source: TP-1	**Depth:** 8.0 ft		
Type of Test:	Consolidated Drained/Saturated		
Test No. (Symbol)	**1 (◆)**	**2 (■)**	**3 (▲)**
Sample Type	Remolded		
Initial Height, in.	1	1	1
Diameter, in.	2.4	2.4	2.4
Dry Density Before, pcf	105.2	105.7	104.9
Dry Density After, pcf	104.5	106.4	105.5
Moisture % Before	4.5	4.5	4.5
Moisture % After	21.1	21.1	21.1
Normal Load, ksf	1.0	2.0	4.0
Shear Stress, ksf	0.83	1.63	2.94
Strain Rate	.00005085 IN/SEC		

Sample Properties	
Cohesion, psf	175
Friction Angle, ϕ	35
Liquid Limit, %	-
Plasticity Index, %	-
Percent Gravel	11
Percent Sand	73
Percent Passing No. 200 sieve	16
Classification	SM

PROJECT: Hidden Cove Subdivision

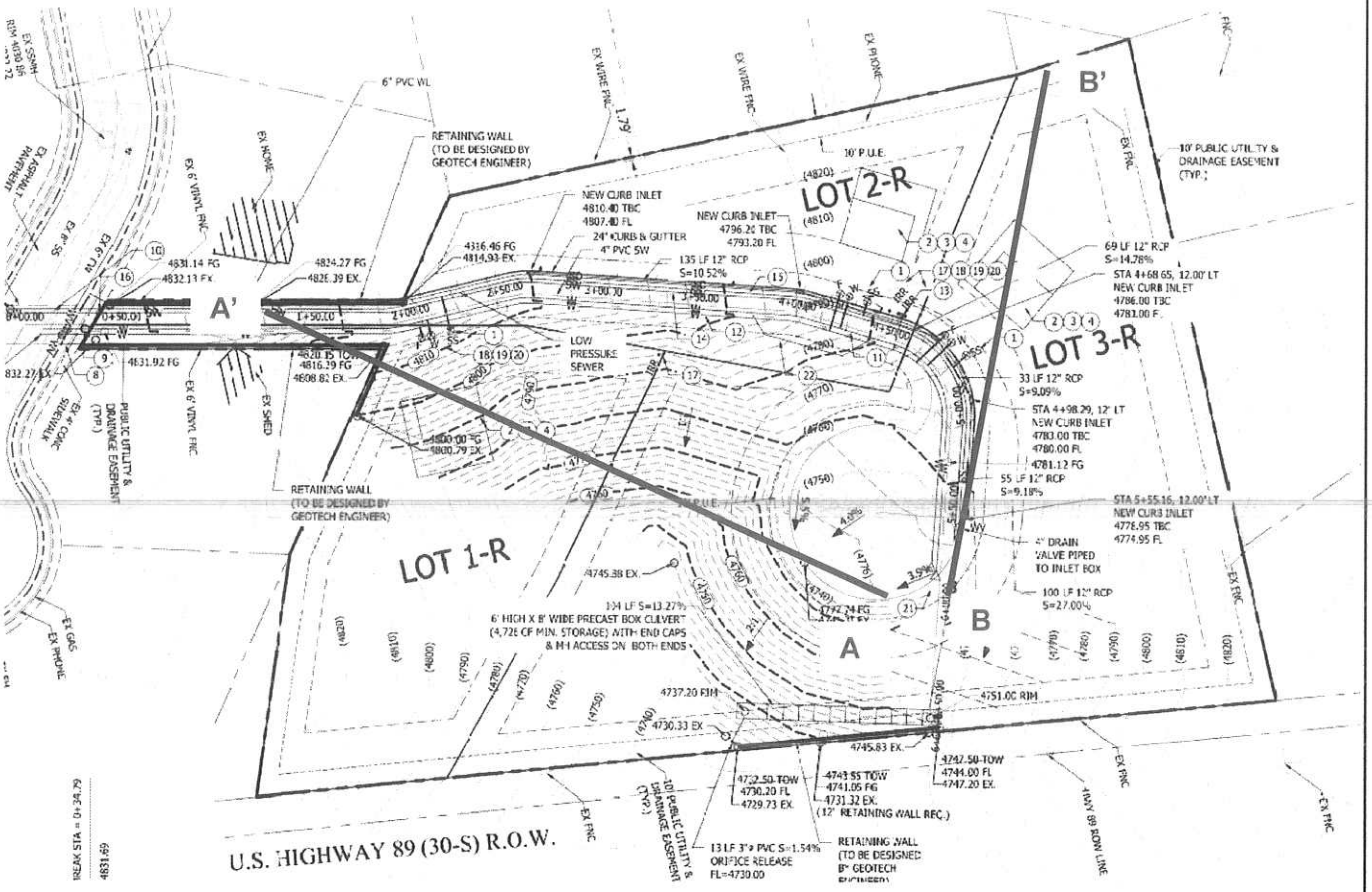
PROJECT NO.: 177078

EARTHTEC ENGINEERING

FIGURE NO.: 7

SLOPE CROSS-SECTION

HIDDEN COVE SUBDIVISION
6260 SOUTH 2125 EAST
OGDEN, UTAH



*Site Plan Provided by Pinnacle Engineering

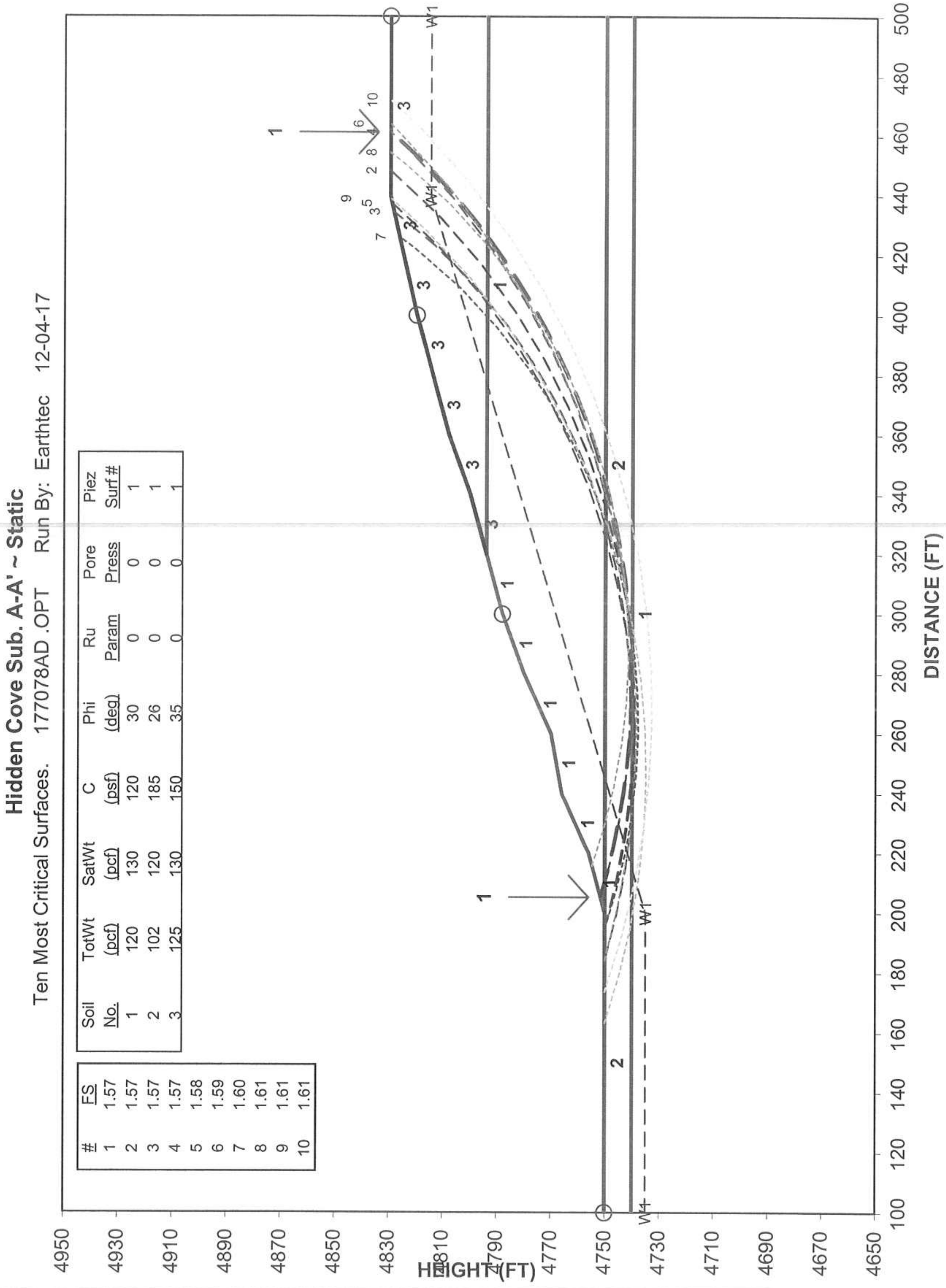
— **Approximate Slope Cross-section**

Not to Scale

PROJECT NO.: 177078

FIGURE NO.: 8

STABILITY RESULTS



Hidden Cove Sub. A-A' ~ Static

Ten Most Critical Surfaces. 177078AD .OPT Run By: Earthtec 12-04-17

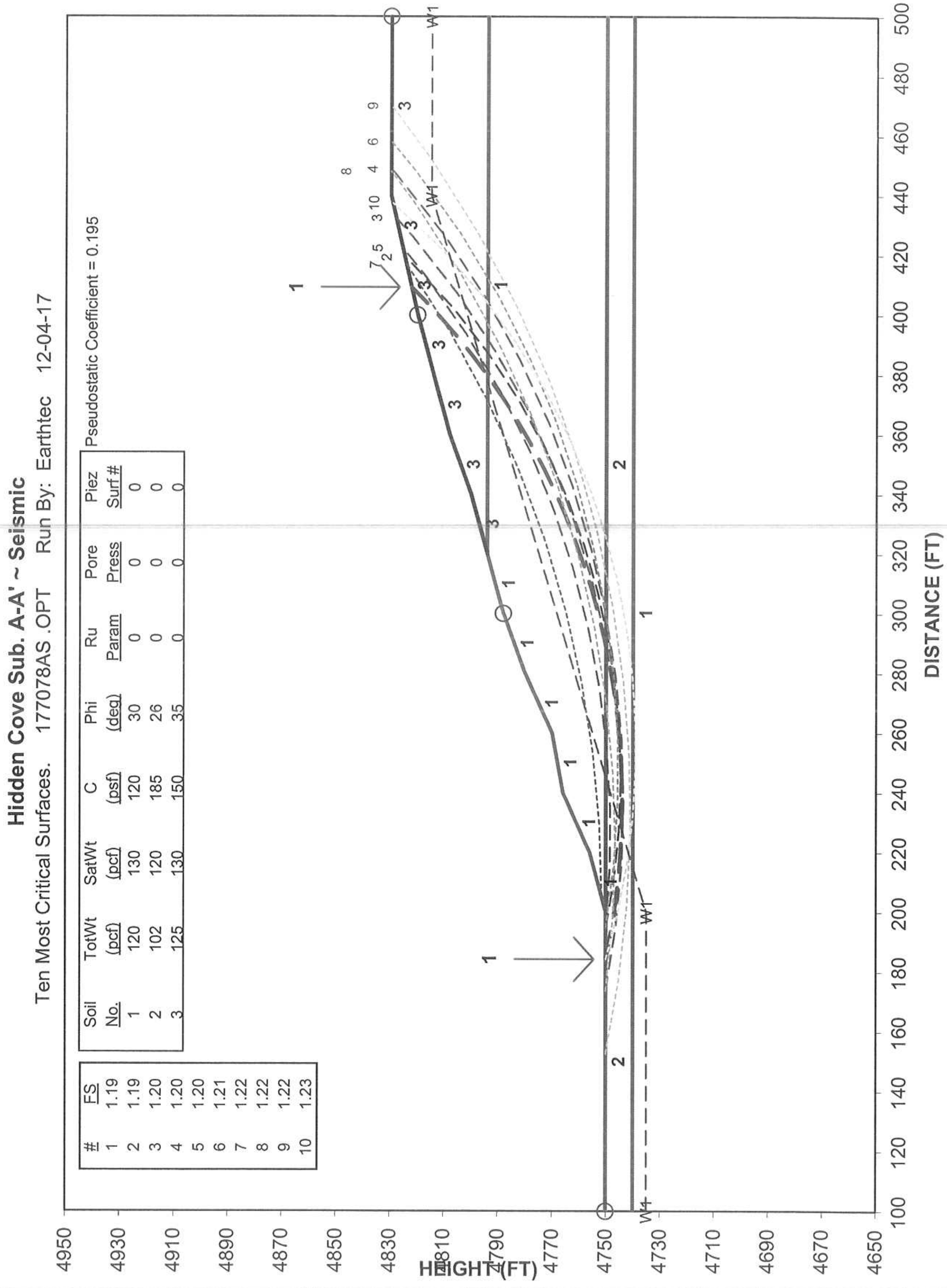
#	FS
1	1.57
2	1.57
3	1.57
4	1.57
5	1.58
6	1.59
7	1.60
8	1.61
9	1.61
10	1.61

Soil No.	TotWt (pcf)	SatWt (pcf)	C (psf)	Phi (deg)	Ru Param	Pore Press	Piez Surf #
1	120	130	120	30	0	0	1
2	102	120	185	26	0	0	1
3	125	130	150	35	0	0	1

PROJECT NO.: 177078

FIGURE NO.: 9

STABILITY RESULTS



Hidden Cove Sub. A-A' ~ Seismic

Ten Most Critical Surfaces. 177078AS .OPT Run By: Earthtec 12-04-17

#	FS
1	1.19
2	1.19
3	1.20
4	1.20
5	1.20
6	1.21
7	1.22
8	1.22
9	1.22
10	1.23

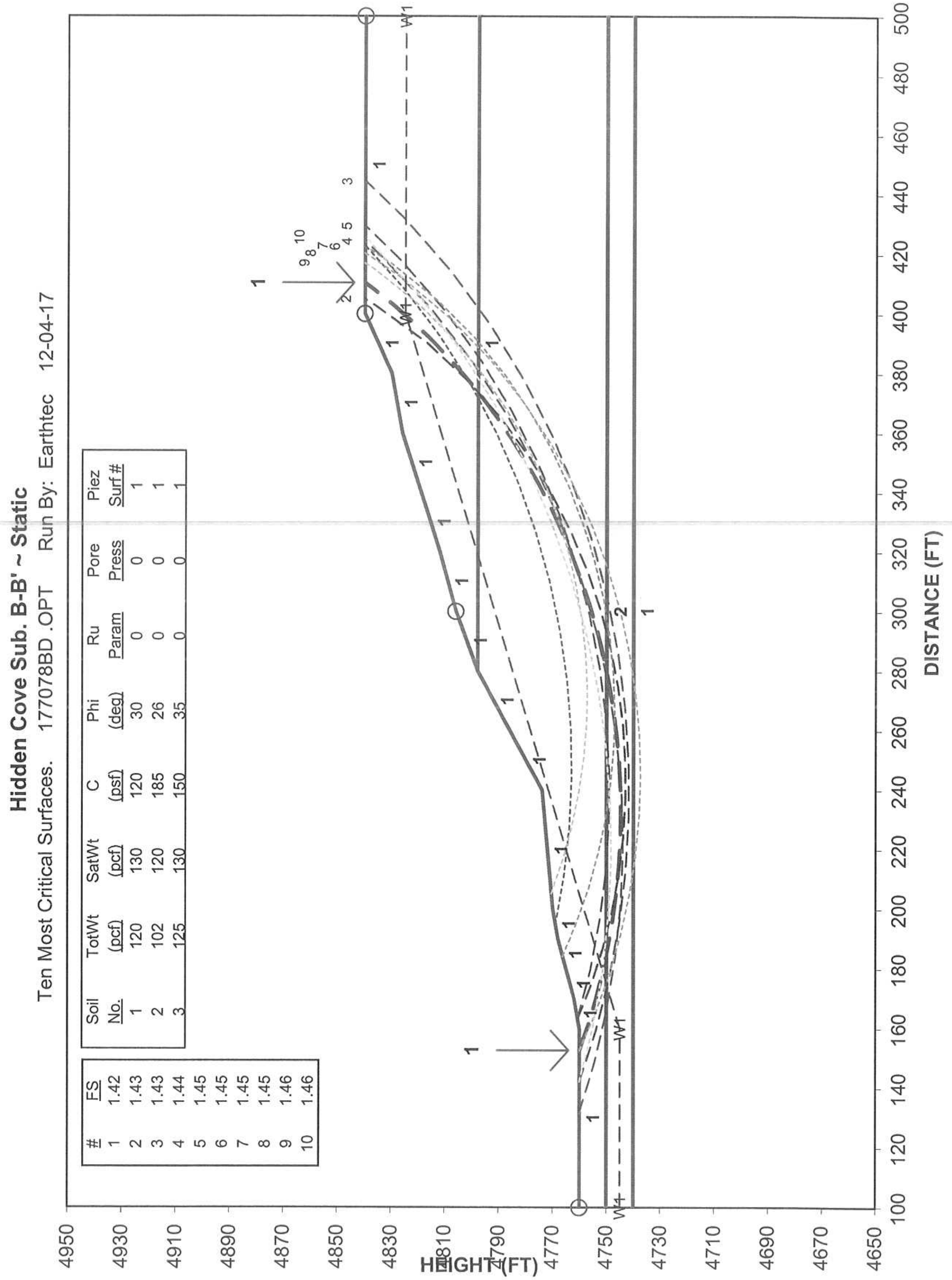
Soil No.	TotWt (pcf)	SatWt (pcf)	C (psf)	Phi (deg)	Ru Param	Pore Press	Piez Surf #
1	120	130	120	30	0	0	0
2	102	120	185	26	0	0	0
3	125	130	150	35	0	0	0

Pseudostatic Coefficient = 0.195

PROJECT NO.: 177078

FIGURE NO.: 10

STABILITY RESULTS



Hidden Cove Sub. B-B' ~ Static

Ten Most Critical Surfaces. 177078BD .OPT Run By: Earthtec 12-04-17

#	FS
1	1.42
2	1.43
3	1.43
4	1.44
5	1.45
6	1.45
7	1.45
8	1.45
9	1.46
10	1.46

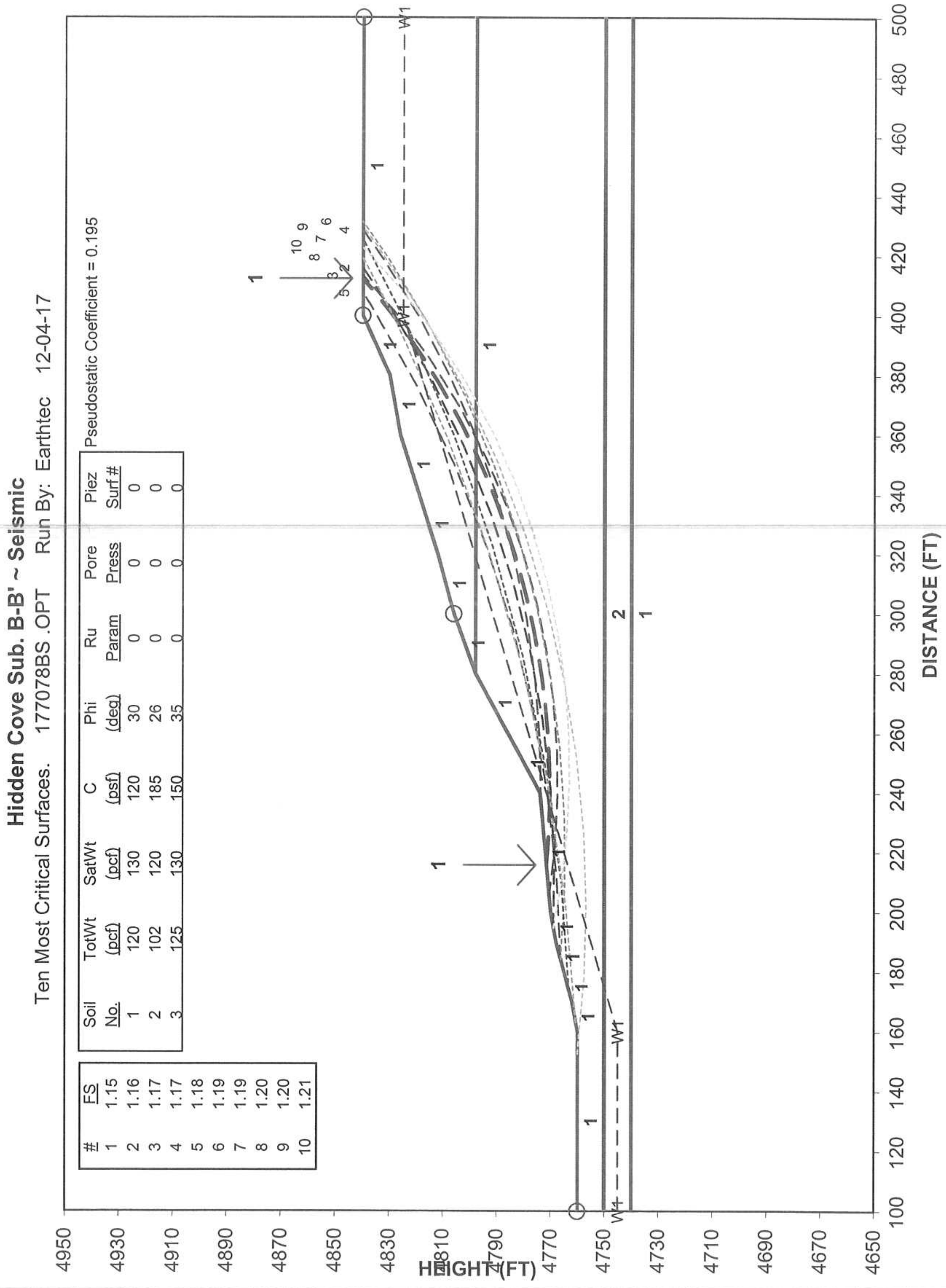
Soil No.	TotWt (pcf)	SatWt (pcf)	C (psf)	Phi (deg)	Ru Param	Pore Press	Piez Surf #
1	120	130	120	30	0	0	1
2	102	120	185	26	0	0	1
3	125	130	150	35	0	0	1

PROJECT NO.: 177078

EARTHTEC ENGINEERING

FIGURE NO.: 11

STABILITY RESULTS



PROJECT NO.: 177078

FIGURE NO.: 12

APPENDIX A

Earth Tech, LLC d.b.a. Earthtec Engineering (Earth Tech, LLC) is a separate business entity, and has no relation to Earthtec Testing & Engineering, P.C.. Earth Tech, LLC assumes no liability or responsibility over the contents contained within Earthtec Testing & Engineering, P.C.'s report. Earth Tech, LLC did not review, confirm, or verify any portion of the attached reports and makes no assurances to the completeness or correctness of their findings. The findings, conclusions, and recommendations presented in the attached reports were provided by others and Earth Tech, LLC does not make any warrantee, guarantee, or representation presented in the reports included in the appendix.

Appendix A includes previously completed reports by other companies provided to Earthtec Engineering by Mr. Donald Fulton. The reports are included in this appendix as requested by Weber County. These reports are not used or referenced in Earth Tech, LLC's Geotechnical Study Job No. 177078, dated December 4, 2017.

Earthtec Testing & Engineering, P.C.

133 North 1330 West
Orem, Utah 84057
Phone (801) 225-5711
Fax (801) 225-3363

1596 West 2650 South #10
Ogden, Utah 84401
Phone (801) 399-9516
Fax (801) 399-9842

475-0866

GEOTECHNICAL/GEOLOGICAL STUDY
KUNZLER SUBDIVISION
6260 SOUTH 2125 EAST
WEBER COUNTY, UTAH

PREPARED FOR: Mike

SEAN KUNZLER
c/o SUN PLAY POOLS & SPAS
5690 S HARRISON BLVD.
OGDEN, UT 84403

Spoke with
mark ~~Christ Hansen~~ CROOKSTON

ETE JOB NO.: 03E-064

FEBRUARY 4, 2003

TABLE OF CONTENTS
ETE JOB NUMBER: 03E-064

TABLE OF CONTENTS - CONTINUED

1.0 INTRODUCTION

We understand that a one lot residential subdivision is planned for a parcel of land located at approximately 6260 South 2125 East in Weber County, Utah as shown on the Vicinity Map, Figure 1. The site is primarily located in the SW ¼ SW ¼ of Section 23, Township 5 North, Range 1 West, at an elevation of approximately 4,720 feet above mean sea level (MSL).

This study was made to assist in evaluating geologic hazards, the subsurface conditions and engineering characteristics of the foundation soils, and in developing our opinions and recommendations concerning appropriate foundation types and floor slabs. This report presents the results of a reconnaissance-level engineering geology and geologic hazards review and evaluation performed by Western GeoLogic, LLC (Western GeoLogic) and a geotechnical study performed by Earthtec Testing and Engineering, PC (Earthtec) which includes a site reconnaissance conducted by an experienced certified engineering geologist to assess the site, review of available geologic maps and reports, field exploration, laboratory testing, an evaluation of available data, and our opinions and recommendations. Data from the studies is summarized on Figures 2 through 7. The engineering geology section of this report has been prepared in general accordance with the Guidelines for Preparing Engineering Geologic reports in Utah (Utah Section of the Association of Engineering Geologists, 1986).

2.0 CONCLUSIONS

1. Based on the two test pits excavated for this study, the site is covered with 6 to 12 inches of topsoil. Native soils below the topsoil generally consist of medium dense silty sand to silty

sand with gravel (SM) which extends beyond the maximum depth investigated (11 ft). No groundwater was encountered within the test pits.

2. The site is considered suitable for the proposed use given the geologic conditions characterized in this report. There are no geologic hazards or engineering geology constraints that would impose unacceptable risks to the construction of a residence at the site, if the recommendations provided in this report are followed.

3. Due to the collapsible nature of the native soils at the site, spread footings should be found on at least 24 inches of structural fill. A maximum allowable bearing capacity of 2000 psf should be used for footing designs.

4. Cut and fill slopes should be graded no steeper than 2 to 1 (horizontal to vertical). The proposed structure should be placed at least 30 feet from the existing slopes where the grade measures 20 percent or steeper.

3.0 PROPOSED CONSTRUCTION

We understand that the planned construction will consist of a single family residence on a single lot subdivision. The proposed structure will be 1 to 2 stories in height with a basement. For design purposes, it was assumed that structural loads would be on the order of 3 to 5 klf for wall loads. If structural loads are different than those assumed, we should be notified and allowed to reevaluate our recommendations.

4.0 SITE CONDITIONS

The subject lot is undeveloped land on the south facing slope above the Weber River. The proposed house pad is located in an amphitheater shaped area covered with scattered stands of scrub oak and low grasses and weeds. The sides of the amphitheater slopes were measured to be 24 to 28 percent. The site is bound by US 89 to the south and residential property on all other sides. The existing homes in the area generally appear to be performing satisfactorily from a foundation viewpoint, based solely on limited exterior visual inspection.

5.0 FIELD INVESTIGATION

The field investigation consisted of excavating two test pits to depths of about 11 feet at the approximate locations shown on Figure 4. The soils encountered were continuously logged by the undersigned engineer and geologist. Disturbed and undisturbed samples were obtained and returned to our laboratory for testing.

6.0 LABORATORY TESTING

The samples obtained during the field investigation were sealed and returned to our laboratory where each one was inspected to verify field classification and to select representative samples for laboratory testing. Laboratory testing consisted of moisture and density determinations, a sieve analyses, and a collapse test. The results of these tests are shown on Figure 6, attached.

7.0 SUBSURFACE CONDITIONS

Based on the two test pits excavated for this study, the site is covered with 6 to 12 inches of topsoil. Native soils below the topsoil generally consist of medium dense silty sand to silty sand with gravel (SM) which extends beyond the maximum depth investigated (11 ft). No groundwater was encountered within the test pits. Graphical representations of the soil conditions encountered in test pits are shown on the Test Pit Logs, Figures 5 and 6. A key to the symbols used on the test pit logs are shown on Figure 7.

8.0 ENGINEERING GEOLOGY

8.1 Site Reconnaissance

On January 31, 2002, Mr. Craig V Nelson of Western GeoLogic conducted site reconnaissance of the project property and surrounding area. Weather at the time of the visit was clear with temperatures in the high 50s. There was no snow cover on the ground.

The site is accessed by a 2125 East Street and consists of undeveloped land covered with scattered stands of scrub oak and low grasses and weeds. The building pad lies within a south-facing amphitheater-shaped parcel. The slopes surrounding the property to the east, north, and west were measured at between 25 and 28 percent.

No evidence of slope instability was observed along the cut and fill slopes (up to five feet in height) along the existing unpaved access driveway. A recent appearing landslide head scarp was observed on the slope to the southwest of the site (Figure 3).

8.2 Aerial Photographs

Aerial photographs were reviewed to obtain information about the geomorphology of the project and surrounding property. Older landslide head scarps combine to form the scallop-shaped slope crest around the project (Figure 3). The most recent scarp appears to be to the southwest of the site. No evidence of other active slope instability was observed on the site. No fault scarps, debris flow levees, or evidence of other geologic hazards on the site was observed in the photos.

8.3 Geologic Setting

The site is located within the Wasatch Front Valley System, a deep sediment-filled, structural basin flanked by two uplifted range blocks; the Wasatch Range to the east, and the Lakeside Mountains to the west. The project is located just below the Provo Shoreline level of Lake Bonneville in an area where lacustrine deposits have been dissected by downcutting and erosion of gullies. Nelson and Personius (1993) mapped the surficial geology at the project site as (Figure 2):

lpd – Deltaic deposits related to regressive phase (uppermost Pleistocene) – Clast-supported pebble and cobble gravel interbedded with thin sand beds, and matrix-supported gravelly sands; moderate to well sorted; clasts subround to round, with weak carbonate cementation common. Deposited as foreset beds with original dips of 30? -35?. Commonly capped with <5 m of topset alluvium (unit alp), which is less well sorted, silty to sandy, pebble and cobble gravel. Mapped at the mouths of North Ogden, Ogden, Weber, and Ward canyons, and the canyon of Mill Creek.

Landslide deposits have not been mapped at the site although the crest of the slope has been mapped as a scarp. The amphitheater shape of the parcel appears to have been created by past headward erosion of a large gulley into the lacustrine delta. Construction of U.S. 89 placed a massive buttress of road fill across the former gulley drainage. Given the field and air photo evidence it appears that no active landslide deposits are located in the area of the building footprint. Future reactivation of movement along the surrounding slopes is likely to be shallow rotational slumps and translational sliding with the primary hazard to upslope properties from headward erosion of the head scarps.

Given the relatively small size of the subject site and the simple surficial geology, the 1:50,000-scale geologic map (enlarged to 1:24,000-scale in Figure 2) shows sufficient detail to adequately portray the geology of the project site.

The Nelson and Personius (1993) map indicates that the nearest bedrock outcrop lies about 1¼ miles east of the site, along the upthrown block of the Wasatch Fault. Given the steep dip and total offset of the Wasatch Fault, bedrock on the downthrown side of the fault in the area of the project would be expected to be quite deep. No bedrock was observed or has been mapped cropping out within the site area. Consequently no bedrock related structural features such as fractures, foliation, schistosity, or folds have been mapped.

8.4 Lake Bonneville History

Deposits from Lake Bonneville represent the majority of the surficial deposits in the vicinity of the site. Given the local importance of Lake Bonneville sediments a brief discussion of Lake Bonneville history follows.

During late-Quaternary time, nearly 100 basins in the western United States contained lakes. The largest of these basins, the Bonneville Basin located in northwestern Utah, was created by extensional tectonism about 15 million years ago (Gwynn, 1980; Miller, 1990). Lake Bonneville consisted of numerous topographically closed basins, including the Salt Lake and Cache Valleys (Oviatt et al., 1992).

Approximately 30,000 years ago, Lake Bonneville began a slow transgression before reaching its highest level of 5,160 to 5,200 feet above mean sea level. The lake culminated at this elevation around 16,000 years before present, creating the geomorphic feature commonly referred to as the Bonneville Shoreline. The water level in Lake Bonneville catastrophically fell roughly 360 feet approximately 14,500 years ago as a result of overtopping a natural dam at Red Rocks Pass in southeastern Idaho; thereby forming a lower shoreline referred to as the Provo Shoreline. The Qpsf deposits of the Weber Delta were deposited during this time. Between 13,000 and 14,000 years ago the lake fell again due to climatic factors, and finally by about 11,000 years ago Lake Bonneville was at the current elevation of the Great Salt Lake (Oviatt et al., 1990 and 1992).

8.5 Seismotectonic Setting

The project site is located along the eastern margin of the Basin and Range physiographic province (Stokes, 1977). The Basin and Range is characterized by a series of generally north-trending elongate mountain ranges separated by predominately alluvial and lacustrine sediment filled valleys. The mountain ranges of the Basin and Range are typically bounded on one, or less frequently, both sides by major normal faults

(Stewart, 1978). The boundary between the Basin and Range and Middle Rocky Mountains provinces is the prominent, west-facing escarpment along the Wasatch fault zone at the base of the Wasatch Range. The late Cenozoic normal faulting, which is characteristic of the Basin and Range, was initiated between approximately 17 and 10 million years (m.y.) ago in the Nevada (Stewart, 1980) and Utah (Anderson, 1989) portions of the province as a result of a roughly east-west directed, regional extensional stress regime. This stress regime has continued to the present (Zoback and Zoback, 1989; Zoback, 1989).

The subject site is also situated in the central portion of the Intermountain seismic belt (ISB). The ISB is a north-south trending zone of historical seismicity along the eastern margin of the Basin and Range province which extends for approximately 1,500km from northern Arizona to northwestern Montana (Sbar et al., 1972; Smith and Sbar, 1974). At least 16 earthquakes of magnitude 6.0 or greater have occurred within the ISB since 1850, with the largest of these events the MS 7.5 1959 Hebgen Lake, Montana earthquake. However, none of these events have occurred along the Wasatch fault or other known late Quaternary faults (Arabasz et al., 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 near the town of Snowville.

Major normal faults in the region include the Wasatch fault, West Valley fault zone (principally comprised of the Taylorsville and Granger faults), and East Great Salt Lake fault zone (Hecker, 1993). The Wasatch fault zone is comprised of multiple (six to ten) segments and is one of the most widely studied faults in the world.

The site is located along the Weber Segment, which is in the central, most active portion of the fault zone. Previous studies (e.g., Black et al., 1995; Lund et al., 1991; Machette et al., 1992, 1991; McCalpin et al., 1994; Personius, 1992; Schwartz and Coppersmith, 1984; Swan et al., 1980) have documented evidence for multiple surface rupture events on each of the segments in the central portion of the fault zone during the latest Pleistocene and Holocene. These surface ruptures are interpreted to be associated with paleo-earthquakes of surface wave magnitude (MS) 6.5 to 7.5. In the past 5,600 years, the estimated average recurrence intervals for these events are on the order of 350 years for the central five segments of the fault zone and 1,275 to 2,800 years for individual segments. The most recent documented surface rupture events on these five segments have occurred between approximately 620 +/- 30 and 2,120 +/- 100 calendar years before present (Machette et al., 1992; McCalpin and Nishenko, 1996). Evidence for Holocene displacement on the East Great Salt Lake fault is based on interpretation of seismic reflection profiles and stratigraphic data for the lake basin (e.g., Mikulich and Smith, 1974; Pechmann et al., 1987; Viveiros, 1986).

8.6 Hydrology and Hydrogeology

No streams, springs, ponds, or marshes were observed on the site. The U.S.G.S. topographic map of the Ogden Quadrangle (Figure 1) indicates that the nearest perennial stream is the Weber River located about ¾ miles to the south of the site.

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. 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 (Clark and others, 1990). Based on topography the regional groundwater flow is expected to be to the south.

Elevation of the shallow aquifer varies somewhat based on seasonal and climatic fluctuations. No groundwater was observed in any of the test pit excavations.

We understand that the subdivision will connect to the sanitary sewer and no septic systems are planned. We also understand that others are performing hydrology and runoff analysis.

9.0 GEOLOGIC HAZARDS

Assessment of potential geologic hazards and the resulting risks imposed is critical in determining the suitability of the site for the proposed land use. A discussion and analysis of geologic hazards follows.

9.1 Earthquake Groundshaking and Seismic Design Criteria

Groundshaking refers to the ground surface acceleration caused by seismic waves generated during an earthquake. Strong ground motion is only likely to present a significant risk during moderate to large earthquakes located within a 60 mile radius of the project area (Boore et al., 1993). A number of seismic sources have been identified within this distance, including active faults such as: East and West Cache Faults; East Bear Lake Fault; the Collinston, Brigham City, Weber, and Salt Lake segments of the Wasatch Fault; East Great Salt Lake Fault; Antelope Island Fault; Morgan Fault; and North Oquirrh Fault; as well as a random or "floating" earthquake source.

The severity of groundshaking at the site will vary with the magnitude of the earthquake, the distance from the earthquake epicenter, and the ground response of the soil. Improperly designed structures can fail during earthquakes delivering strong ground motions. The risk from this hazard can be adequately mitigated by design and construction of the proposed residential structure to appropriate building codes. The proposed Residential structure should be designed in accordance with the IRC building code. According to the IRC maps, this site is classified as Site Class E; however, in accordance with section R301.2.2.1.2 the site may be reclassified as site class D_2 since the area is a site class D according to the IBC.

9.2 Surface Fault Rupture

Surface fault rupture is the hazard related to differential movement of the ground surface along a fault during large earthquakes. Faults generating earthquakes with measured Richter magnitudes of less than 6½ typically do not express rupture at the ground surface. Large earthquakes (Richter magnitudes 7 to 7½) have been associated with over 6 feet of vertical surface rupture along normal faults. The ground rupture may be expressed either as one large displacement or several smaller ruptures comprising a fault zone. Ground displacement from surface fault rupture can cause significant damage or even collapse to structures located across the rupture zone.

Based on the field reconnaissance and review of published references there is no evidence that known active faults pass through the subject site. Given the evidence from paleoseismic studies, ground displacement associated with surface fault rupture will likely be restricted to areas adjacent to the active faults mapped by Nelson and Personius (1993) about a mile to the east of the site.

Based on this information and our current understanding that surface fault rupture and deformation tend to follow past patterns, it is believed that the proposed home may be developed without undue risk from surface fault rupture. Given this information the risk posed by surface fault rupture to the subject property is rated as low.

9.3 Liquefaction and Lateral-Spread Ground Failure

Liquefaction is defined as the condition when saturated, loose, cohesionless, soils lose their support capabilities during a seismic event because of the development of excessive pore pressure within the soil. Earthquake-induced soil liquefaction can present a significant risk to structures. Groundshaking may cause saturated, sandy soils to "liquefy" due to increased pore pressure between soil grains. Earthquakes of Richter magnitude 5 are generally regarded as the lower threshold for liquefaction. Liquefaction can cause bearing capacity failures of structural footings and foundations and damage roadway embankments by triggering lateral spread landslides. Lateral spread-type failures can occur on nearly flat slopes and indeed lateral spread deposits from prehistoric lateral spread landslides have been mapped along the Wasatch Front to the north and south of the project site.

According to the Utah Geologic Survey's liquefaction map for Weber County, this site is in an area classified as having a low to moderate potential for liquefaction (UGS, undated). Due to the type of subsurface investigation conducted for this report, we are unable to perform a liquefaction analysis for this site. It is possible that there are sand lenses at this site which are susceptible to liquefaction and significant settlement in excess of one inch should be expected during a strong seismic event. To adequately evaluate the liquefaction potential at the site, a boring at least 30 feet deep would need to be drilled. Earthtec would be happy to provide this service upon request.

9.4 Tectonic Subsidence

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 fault block (west of the fault). Given the distance of the property from the fault, the risk to the subject property from tectonic subsidence is rated as low.

9.5 Seismic Seiche and Storm Surge

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

9.6 Landslide and Slope Failures

Slope stability hazards such as landslides, slumps, and other mass movements can develop along moderate to steep slopes where the slope has been disturbed, the head of the slope loaded, or where an increase in groundwater pore pressures result in driving forces within the slope exceeding the restraining forces. Evidence of recent slope instability was observed on the slope to the southwest of the site (Figure 3). The surrounding slopes to the east, north, and west were likely formed by past movement along shallow, rotational slumps and shallow translational slides that are likely the primary mechanism for slope retreat along the Weber River. Given the stability characteristics of unconsolidated lacustrine deposits combined with the moderate to fairly steep slopes there is a potential risk of continued slope retreat and related landslide movement. However, given that the majority of future movement will develop along the slope crests, the landslide hazard should not present a significant risk to the proposed residence if constructed on the relatively flat area at the base of the amphitheater with an adequate set-back from the toe of the slope. We recommend that the proposed structure be placed at least 30 feet from the existing slope where the grade measures 20 percent or steeper.

9.7 Debris Flows

Debris flow hazards are typically associated with surficial units mapped as alluvial fan deposits at the mouths of large range front drainages. No alluvial fan deposits and no debris flow deposits have been mapped at the site. No evidence of debris flow deposition was observed during the site reconnaissance. Given the distance of the site from the range front, the absence of debris flow deposition features and large drainage channels, the hazard from debris flows in the project area is rated as low.

9.8 Rockfall

No large rocks were observed in the deposits comprising the slope south of the project and no evidence of fallen rock accumulations was observed on the site. Given this information and the distance of the site from the range front, the rockfall hazard for the project area is rated as low.

9.9 Snow Avalanche

Given the relatively low elevation (about 4,720 feet above MSL) and distance from the range front, the hazard from avalanche at the site is rated as low.

9.10 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 within a "Moderate" radon-hazard potential area (Black, 1996). A moderate hazard rating indicates that indoor radon concentrations would be expected to be in the range of 2 to 4 picocuries per liter of air (pCi/L). Actual indoor radon levels can be affected by non-geologic factors such as building construction, maintenance, and weather. Indoor testing following construction is the best method to characterize the radon hazard and determine if mitigation measures are required.

9.11 Volcanic Eruption

No active volcanoes, vents, or fissures have been mapped in the region. Nor have igneous rocks been mapped at the site or in the vicinity. Given the location of the development there is likely no volcanic hazard at the site and the risk to the project is low.

9.12 Swelling and/or Collapsible Soils

Surficial soils that contain certain characteristics can swell or collapse when subjected to alternating cycles of wetting and drying. A collapse test performed on an undisturbed sample obtained in one of our test pits, indicated a moderately high collapse potential. The recommendation presented in Section 11 of this report should followed to prevent excessive differential settlement.

9.13 Conclusions

The site is considered suitable for the proposed use given the geologic conditions characterized in this report. There are no geologic hazards or engineering geology constraints that would impose unacceptable risks to the construction of a residence at the site, if the recommendations provided in this report are followed.

10.0 SITE GRADING

10.1 General Site Grading

Topsoil, man-made fill, and soils loosened by construction activities should be removed (stripped) from the building pad and concrete flatwork areas prior to foundation excavation and placement of site grading fills. Following stripping and any additional excavation required to achieve design grades, the subgrade should be proof rolled to a firm, non-yielding surface. Soft areas detected during the proof-rolling operation should be removed and replaced with structural fill.

10.2 Structural Fill and Compaction

All fill placed below the building and concrete flatwork should be structural fill. All other fills should be considered as backfill. Structural fill should consist of native sands at the site or imported material. Imported material should consist of well-graded sandy gravels with a maximum particle size of 3 inches and 5 to 15 percent fines (materials passing the No. 200 sieve). The liquid limit of the fines should not exceed 35 and the plasticity index should be below 15. All fill soils should be free from topsoils, highly organic material, frozen soils, and other deleterious materials. Structural fill should be placed in maximum 8-inch thick loose lifts at a moisture content within 2 percent of optimum and compacted to at least 95 percent of maximum density (ASTM D 1557) under the building and 90 percent under concrete flatwork.

10.3 Backfill

The native soils may be used as backfill in utility trenches and against outside foundation walls. Backfill should be placed in lift heights suitable to the compaction equipment used and compacted to at least 90 percent of the maximum dry density(ASTM D 1557).

10.4 Excavations

Temporary construction excavations at the site that are less than five feet deep should have slopes no steeper than ½ to 1 (horizontal to vertical). Excavations which are advanced deeper than five feet below site grades or where water is encountered should be sloped or braced in accordance with OSHA Health and Safety Standards, final rule, CFR 29, part 1926 for a type C soil.

11.0 FOUNDATIONS

11.1 Footing Design

To prevent excessive differential settlement due to the collapse potential at the site, we recommend that footings be founded on at least 24 inches of structural fill. The recommendations presented below should be utilized during design and construction of this project:

1. Spread footings founded on at least 24 inches of structural fill should be designed for a maximum allowable soil bearing capacity of 2000 psf. A one-third increase is allowed for short term transient loads such as wind and seismic events. Footings should be uniformly loaded.
2. Continuous footings should have a minimum width of 18 inches.

3. Exterior footings should be placed below frost depth which is determined by local building codes. Generally 30 inches is adequate in this area. Interior footings, not subject to frost, should extend at least 18 inches below the lowest adjacent grade.

4. Foundation walls on continuous footings should be well reinforced both top and bottom. We suggest a minimum amount of steel equivalent to that required for a simply supported span of 12 feet.

5. The bottom of footing excavations should be cleaned of all loose and disturbed soils and should be tested with non-vibratory compaction equipment to identify soft areas. If soft areas are encountered, they should be removed and replaced as recommended in Section 10.1.

11.2 Estimated Settlement

If footings are designed and constructed in accordance with the recommendations presented above, the risk of total settlement exceeding 1 inch and differential settlement exceeding 0.5 inch for a 25-foot span will be low. Additional settlement should be expected during a strong seismic event.

12.0 BELOW GRADE WALLS

Buried structures should be designed to resist the lateral loads imposed by the soils retained. The lateral earth pressures on the buried structures and the distribution of those pressures depends upon the type of structure, hydrostatic pressures, in-situ soils, backfill, and tolerable movements. Retaining and basement walls are usually designed with triangular stress distributions known as equivalent fluid pressure based on lateral earth pressure coefficients. Buried structures may be designed using the following ultimate values:

Condition	Lateral Pressure Coefficient	Equivalent Fluid Weight (PCF)
At Rest	0.50	65
Active	0.27	35
Passive	3.69	480

We recommend that the lateral earth pressures for walls which allow little or no wall movement be based on "at rest" conditions. Walls allowed to rotate 0.4 percent of the wall height may be designed with "active" pressures. These values assume level backfill extending horizontally for a distance at least as far as the wall height and that water will not accumulate behind walls. Backfill should be placed in accordance with the requirements discussed in Section 10.3. Lateral pressures approximately 30 percent higher will occur during backfill placement and bracing may be called for until the backfilling operation is completed.

Lateral building loads will be resisted by frictional resistance between the footings and the foundation soils and by passive pressure developed by backfill against the wall. For footings on structural fill we recommend a friction coefficient of 0.35 be used. The lateral earth coefficients presented above are ultimate values; therefore, an appropriate factor of safety should be applied to the values presented above.

13.0 FLOOR SLABS

A minimum 4-inch thick layer of free-draining gravel should be placed immediately below the floor slab to help distribute floor loads, break the rise of capillary water, and aid in the concrete curing process. For slab design, we recommend a modulus of subgrade reaction of 250 psi/in be used. To help control normal shrinkage and stress cracking, the floor slabs should have adequate reinforcement for the anticipated floor loads with the reinforcement continuous through interior floor joints and contain frequent crack control joints.

Special precautions should be taken during placement and curing of all concrete slabs and flatwork. Excessive slump (high water-cement ratios) of the concrete and/or improper finishing and curing procedures used during hot or cold weather conditions may lead to excessive shrinkage, cracking, spalling, or curling of slabs. We recommend all concrete placement and curing operations be performed in accordance with American Concrete Institute (ACI) codes and practices.

14.0 SURFACE DRAINAGE

Wetting of the foundation soils will likely cause some degree of volume change within the soil and should be prevented both during and after construction. We recommend that the following precautions be taken at this site:

1. The ground surface should be graded to drain away from the structure in all directions. We recommend a minimum fall of 8 inches in the first 10 feet.
2. Roof runoff should be collected in roof drains with down spouts designed to discharge well outside of the backfill limits.
3. Sprinkler heads should be aimed away and kept at least 12 inches from foundation walls.
4. Other precautions which may become evident during design and construction should be taken.

15.0 GENERAL CONDITIONS

The exploratory data presented in this report were collected to provide geotechnical design recommendations for this project. Test pits were widely spaced and may not be indicative of subsurface conditions between the points explored or outside the study area and thus have limited value in depicting subsurface conditions for contractor bidding. If it is necessary to define subsurface conditions in sufficient detail to allow accurate bidding we recommend an additional study be conducted which is designed for that purpose.

Variations from the conditions portrayed in the test pits often occur which are sometimes sufficient to require modifications in the design. If during construction, conditions are found to be different than those presented in this report, please advise us so that the appropriate modifications can be made. An experienced geotechnical engineer or technician should observe fill placement and conduct testing as required to confirm the use of proper structural fill materials and placement procedures.

The geotechnical study as presented in this report was conducted within the limits prescribed by our client, with the usual thoroughness and competence of the engineering profession in the area. No other warranty or representation, either expressed or implied, is intended in our proposals, contracts or reports.

We appreciate the opportunity of providing our services on this project. If we can answer questions or be of further service, please call.

Respectfully;
EARTHTEC ENGINEERING, P

No. 188039
MARK I.
CHRISTENSEN

Mark I. Christensen, P.E.
Project Geotechnical Engineer

WESTERN GEOLOGIC, LLC

CRAIG V
NELSON
5251804

Craig V Nelson, P.G., R.G., C.E.G.
Professional Geologist, State of Utah

3 copies sent

16.0 REFERENCES

Anderson, R.E., 1989. Tectonic evolution of the intermontane system--Basin and Range, Colorado Plateau, and High Lava Plains: in Pakiser, L.C., and Mooney, W.D., editors, Geophysical framework of the continental United States: Geological Society of America Memoir 172, p. 163-176.

Arabasz, W.J., Pechmann, J.C., and Brown, E.D., 1992. Observational Seismology and evaluation of earthquake hazards and risk in the Wasatch Front area, Utah: in Assessment of Regional and Earthquake Hazards and Risk Along the Wasatch Front, Utah, edited by P.L. Gori and W.W. Hays, U.S. Geological Survey Professional Paper 1500-D, D1-D36.

Black, B.D., 1996, Radon-hazard potential in the lower Weber River area, Weber and Davis Counties, Utah: Utah Geological Survey Public Information Series 45.

Black, B.D., Lund, W.R., and Mayes, B.H., 1995. Summary of new information from the South Fork Dry Creek site, Salt Lake County, Utah: in Environmental and Engineering Geology of the Wasatch Front Region, edited by W.R. Lund, Utah Geol. Assoc. Pub. 24, p. 11-30.

Boore, D.M., Joyner, W.B., and Fumal, T.E., 1993. Estimation of Response Spectra and Peak Acceleration from Western North America Earthquakes: An Interim Report: U.S. Geological Survey Open-File Report 93-509.

Clark, D.W., Appel, C.L., Lambert, P.M., and Puryear, R.L., 1990, Ground-water resources and simulated effects of withdrawals in the East Shore area of Great Salt Lake, Utah: Utah Department of Natural Resources Technical Publication 93, 150p.

Gwynn, J.W. (Editor), 1980. Great Salt Lake: a scientific, historical, and economic overview: Utah Geological and Mineral Survey Bulletin 166, p. 1-400.

Hecker, Suzanne, 1993, Quaternary tectonics of Utah with emphasis on earthquake-hazard characterization: Utah Geological Survey Bulletin 127.

Lund, W.R., Schwartz, D.P., Mulvey, W.E., Budding, K.E., and Black, B.D., 1991. Fault behavior and earthquake recurrence on the Provo segment of the Wasatch fault zone at Mapleton, Utah County, Utah, in Paleoseismology of Utah, v. 1, Utah Geological Survey Special Study 75, 41 pp.

Lund, W.R. (Editor), 1990. Engineering geology of the Salt Lake City metropolitan area, Utah: Utah Geological and Mineral Survey Bulletin 126, 66 p.

Machette, M.N., Personius, S.F., Nelson, A.R., Schwartz, D.P., and Lund, W.R., 1991. Segmentation and history of Holocene earthquakes, Wasatch fault zone, Utah: Journal of Structural Geology, v. 13, p. 137-149.

Machette M.N., Personius, S.F., and Nelson, A.R., 1992. Paleoseismology of the Wasatch fault zone: A summary of recent investigations, interpretations, and conclusions, in Assessment of Regional Earthquake

Hazards and Risk Along the Wasatch Front, Utah, edited by P.L. Gori and W.W. Hayes, U.S. Geological Survey Professional Paper 1500-A, A1-A71.

McCalpin, J.P., Forman, S.L, and Lowe, M., 1994, Reevaluation of Holocene faulting at the Kaysville site, Weber segment of the Wasatch fault zone, Utah: Tectonics, v. 13, p. 1-16.

McCalpin, J.P. and Nishenko, S.P., 1996. Holocene paleoseismicity, temporal clustering, and probabilities of future large (M > 7) earthquakes on the Wasatch fault zone, Utah: Journal Geophysical Research, v. 101, no. B3, p. 6233-6253.

Mikulich, M. and Smith, R.B., 1974. Seismic reflection and aeromagnetic surveys of the Great Salt Lake, Utah: Geological Society of America Bulletin, v. 85, p. 991-1002.

Miller, D.M., 1990. Mesozoic and Cenozoic tectonic evolution of the northeastern Great Basin: in Geology and Ore Deposits of the Northeastern Great Basin, Shaddrick, D.R., Kizis, J.R., and Hunsaker, E.L. III, (editors): Geological Society of Nevada Field Trip No. 5, p. 43-73.

Nelson, A.R., and Personius, S.F., 1993, Surficial geologic map of the Weber segment, Wasatch Fault zone, Weber and Davis Counties, Utah: U.S. Geological Survey Map I-2199, scale 1:50,000.

Oviatt, C.G., Currey, D.R., and Miller, D.M., 1990. Age and paleoclimatic significance of the Stansbury shoreline of Lake Bonneville, northeastern Great Basin: Quaternary Research, v. 33, p. 291-305.

Oviatt, C.G., Currey, D.R., and Sack, D., 1992. Radiocarbon chronology of Lake Bonneville, Eastern Great Basin, USA: Paleogeography, Paleoclimatology, Paleoecology, v. 99, p. 225-241.

Oviatt, C.G., Currey, D.R., and Sack, D., 1992. Radiocarbon chronology of Lake Bonneville, Eastern Great Basin, USA: Paleogeography, Paleoclimatology, Paleoecology, v. 99, p. 225-241.

Pechmann, J.C., Nash, W.P., Viveiros, J.J., and Smith, R.B., 1987. Slip rate and earthquake potential of the East Great Salt Lake fault, Utah: [abs.] E05, Transaction of the American Geophysical Union, v. 68, p. 1369.

Personius, S.F., and Scott, W.F., 1992. Surficial geologic map of the Salt Lake City segment and parts of adjacent segments of the Wasatch Fault Zone, Davis, Salt Lake, and Utah counties, Utah: U.S. Geological Survey Miscellaneous Investigations, Map I-2106, scale 1:50,000.

Sbar, M.L., Barazangi, M., Dorman, J., Scholz, C.H., and Smith, R.B., 1972, Tectonics of the Intermountain seismic belt, western United States: Microearthquake seismicity and composite fault plane solutions: Geological Society of America Bulletin, v. 83, p. 13-28.

Schwartz, D.P., and Coppersmith, K.J., 1984. Fault behavior and characteristics earthquakes--examples from the Wasatch and San Andreas fault zones: Journal of Geophysical Research, v. 89, no. B7, p. 5681-5698.

Smith, R.B., and Arabasz, W.J., 1991. Seismicity of the Intermountain seismic belt: in Slemmons, D.B., Engdahl, E.R., Zoback, M.D., and Blackwell, D.D., editors, Neotectonics of North America: Geological Society of America, Decade of North American Geology Map v. 1, p. 185-228.

Smith, R.B. and Sbar, M.L., 1974. Contemporary tectonics and seismicity of the western United States with emphasis on the Intermountain Seismic belt: Geological Society of America Bulletin, v. 85, p. 1205-1218.

Stewart, J.H., 1978, Basin-range structure in western North America, a review, in Smith, R.B., and Eaton, G.P., editors, Cenozoic tectonics and regional geophysics of the western Cordillera: Geological Society of America Memoir 152, p. 341-367.

Stewart, J.H., 1980, Geology of Nevada: Nevada Bureau of Mines and Geology Special Publication 4.

Stokes, W.L., 1977, Physiographic subdivision of Utah: Utah Geological and Mineral Survey Map 43, scale 1:2,400,000.

Swan, F.H., III, Schwartz, D.P., and Cluff, L.S., 1980. Recurrence of moderate to large magnitude earthquakes produced by surface faulting on the Wasatch fault zone, Utah: Bulletin Seismological Society of America, v. 70, p. 1431-1462.

U.S. Department of Agriculture, 1984, Aerial photographs 58014 Frames 3-10 and 3-11, dated April 10, 1985.

Utah Section of the Association of Engineering Geologists, 1986, Guidelines for preparing engineering geologic reports in

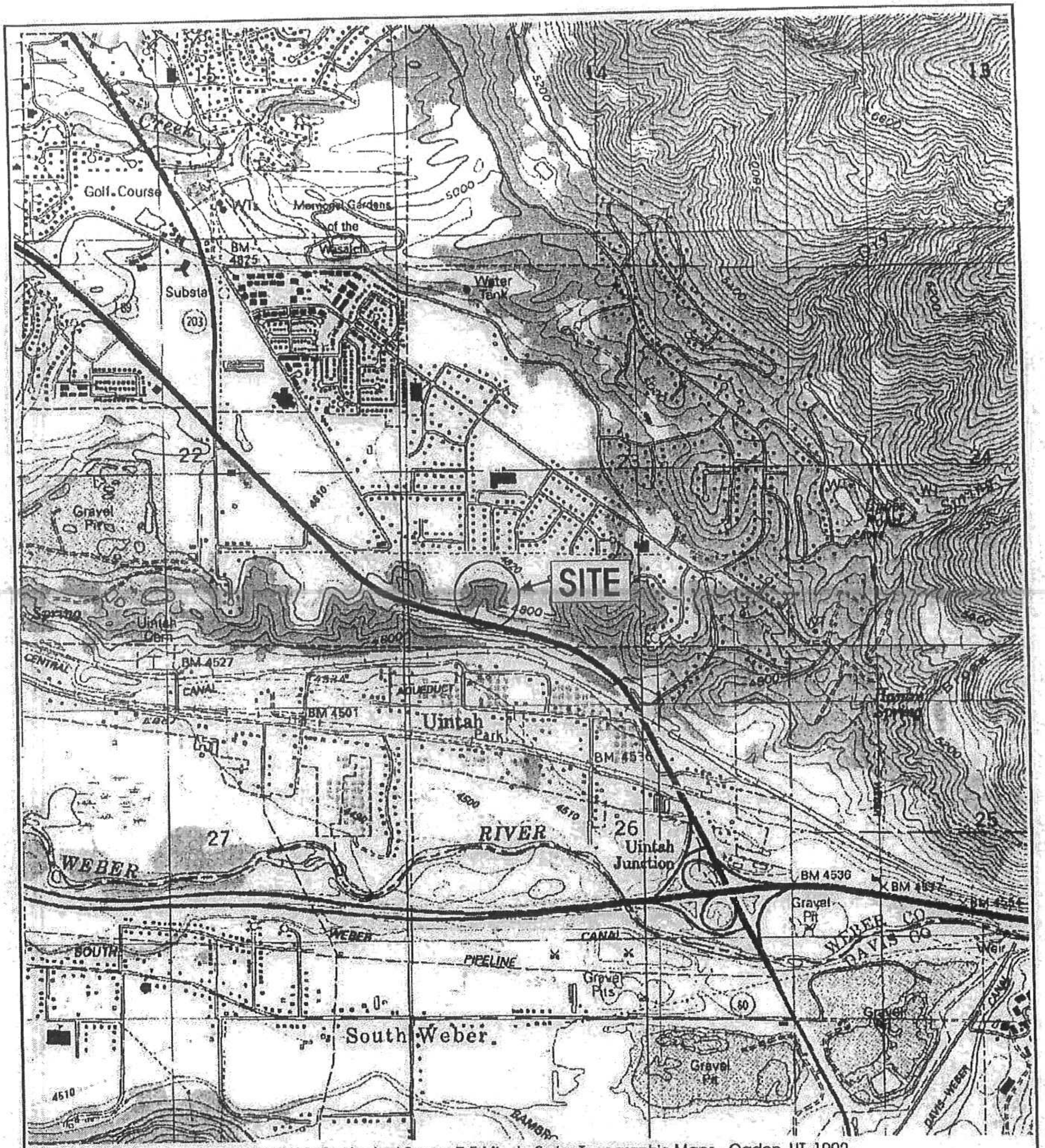
Utah Geologic Survey (UGS), Undated. Selected Critical Facilities and Geologic Hazards, Weber County, Utah.

Utah: Utah Geological Survey Misc. Pub. M, 2p.

Viveiros, J.J., 1986. Cenozoic tectonics of the Great Salt Lake from seismic reflection data: Salt Lake City, University of Utah, unpublished M.Sci. Thesis, 81 p.

Zoback, M.L. and Zoback, M.D., 1989. Tectonic stress field of the conterminous United States: Boulder, Colorado, Geological Society of America Memoir, v. 172, p. 523-539.

Zoback, M.L., 1989. State of stress and modern deformation of the northern Basin and Range province: Journal of Geophysical Research, v. 94, p. 7105-7128.



Source: U.S. Geological Survey 7.5 Minute Series Topographic Maps - Ogden, UT, 1992.

ETE

WESTERN
GEOLOGIC

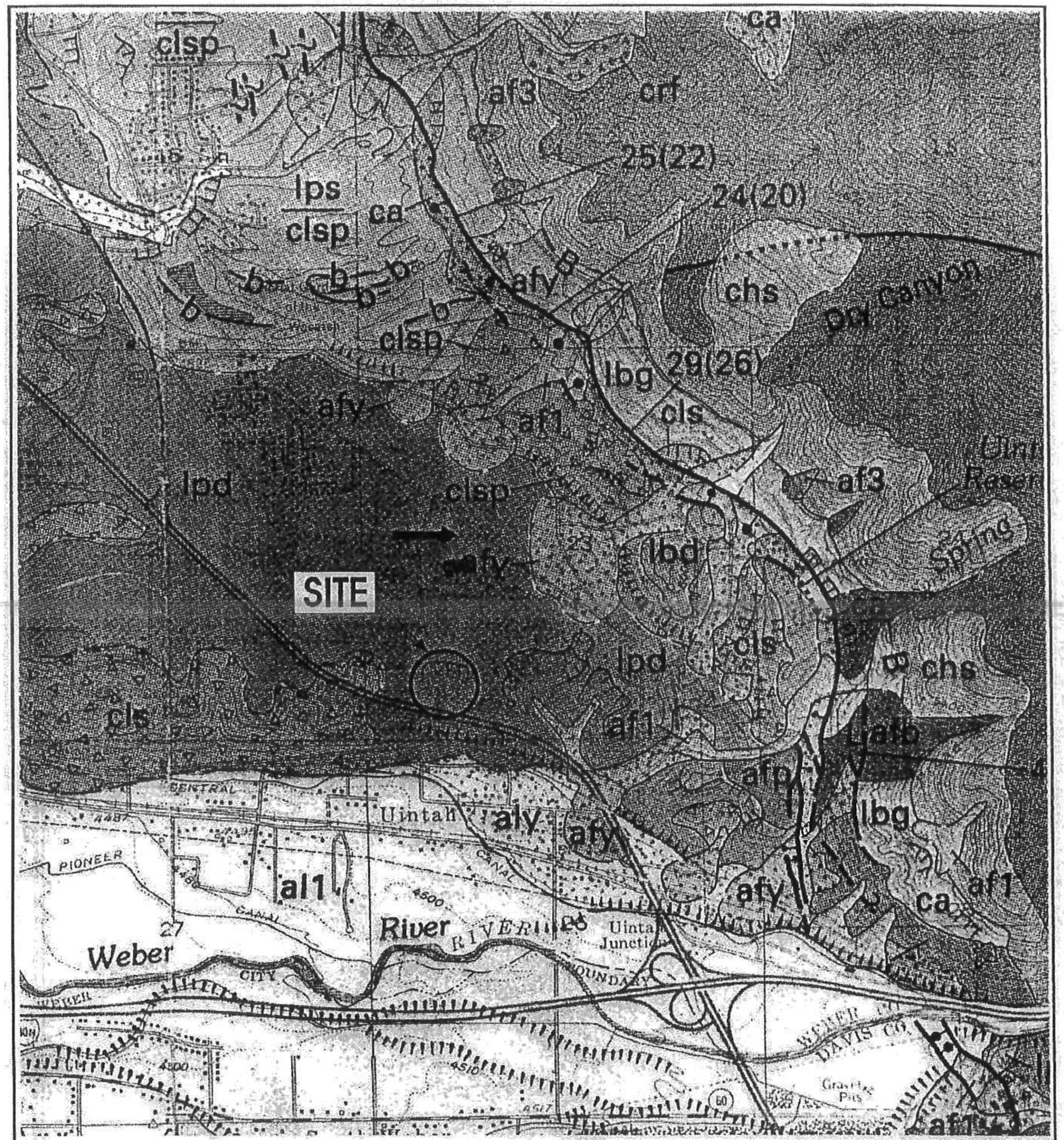
N
W E
S

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

LOCATION MAP

ENGINEERING GEOLOGY AND
GEOTECHNICAL ENGINEERING
KUNZLER SUBDIVISION
Approximately 6260 SOUTH 2125 EAST
OGDEN, UTAH

FIGURE 1



Source: Nelson, A.R., And Personius, S.F., 1993, Surficial geologic map of the Weber segment, Wasatch Fault Zone, Weber and Davis Counties, Utah: U.S. Geological Survey Map I-2199, scale 1:50,000.

ETE
WESTERN
GEOLOGIC

N
W E
S

Scale 1:24,000
see report text for
explanation
of geologic units

GEOLOGIC MAP

ENGINEERING GEOLOGY AND
GEOTECHNICAL ENGINEERING
KUNZLER SUBDIVISION
Approximately 6260 SOUTH 2125 EAST
OGDEN, UTAH

FIGURE 2

Source: Mapquest.com digital aerial photograph

ETE WESTERN GeoLogic

Landslide Headscarp
(solid line recent,
dashed line older appearing)

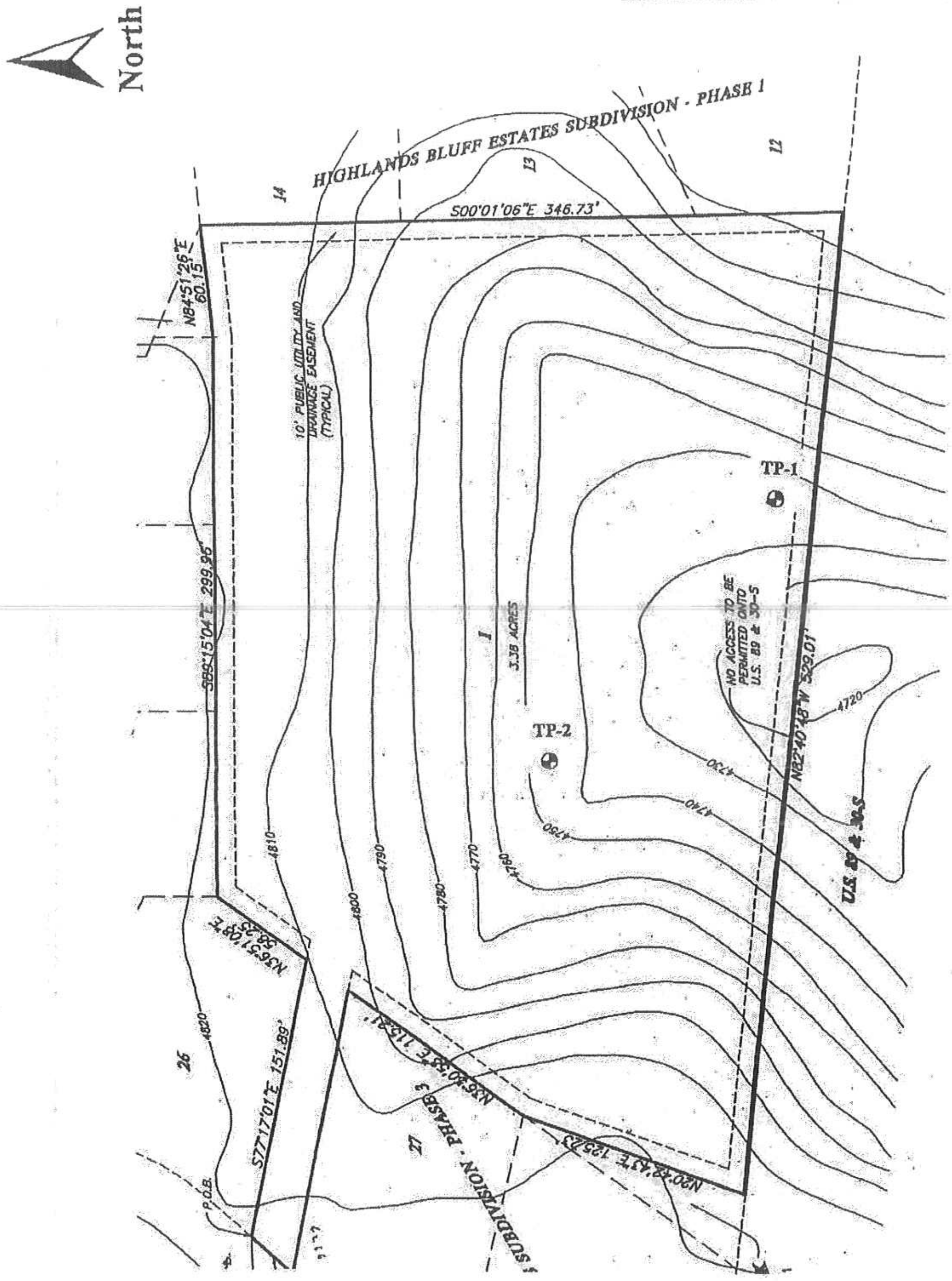
N
W E
S

AERIAL PHOTOGRAPH

ENGINEERING GEOLOGY AND
GEOTECHNICAL ENGINEERING
KUNZLER SUBDIVISION
Approximately 6260 SOUTH 2125 EAST
OGDEN, UTAH

FIGURE 3

EARTHTEC ENGINEERING



NOT TO SCALE

Base map provided by
Reeve & Associates, Inc.

SITE PLAN SHOWING LOCATION OF TEST PITS

ETE JOB NO. 03E-064

FIGURE 4

TEST PIT LOG

PIT NO.: TP-1

PROJECT: Kunzler Subdivision
CLIENT: Sean Kunzler
LOCATION: See Figure 4
OPERATOR: Kastle Rock Excavation
EQUIPMENT: Backhoe

PROJECT NO.: 03E-064
DATE: 1/31/03
ELEVATION: NM
LOGGED BY: Mark C. & Craig N.

Depth (Ft.)	Graphic Log	USCS	Description	Samples	TEST RESULTS							
					Dry Dens. pcf	Water Cont. %	PL	LL	Gravel %	Sand %	Fines %	Other Tests
	▨	Soil	Topsoil; Silty Sand - slightly moist, dark brown									
	▦		Silty Sand - medium dense, slightly moist, brown to light brown									
3	▦											
6	▦	SM										
9	▦											
12												
15												

Notes:

Tests Key:
A = Atterberg Limits
C = Consolidation
G = Gradation
DS = Direct Shear
SO = Solubility
UC = Unconf. Compress. Strength

PROJECT NO. 03E-064

EARTHTEC ENGINEERING, P.C.

FIGURE NO.: 5

TEST PIT LOG

PIT NO.: TP-2

PROJECT: Kunzler Subdivision
CLIENT: Sean Kunzler
LOCATION: See Figure 4
OPERATOR: Kastle Rock Excavation
EQUIPMENT: Backhoe

PROJECT NO.: 03E-064
DATE: 1/31/03
ELEVATION: NM
LOGGED BY: Mark C. & Craig N.

Depth (Ft.)	Graphic Log	USCS	Description	Samples	TEST RESULTS							
					Dry Dens. pcf	Water Cont. %	PL	LL	Gravel %	Sand %	Fines %	Other Tests
		Soil	Topsoil; Silty Sand - slightly moist, dark brown									
			Silty Sand with gravel - medium dense, slightly moist, brown to light brown									
3												
6		SM		☒	95.2	5.7			33	35	32	Collapse= 2.6%
9												
12												
15												

Notes:

Tests Key:
A = Atterberg Limits
C = Consolidation
G = Gradation
DS = Direct Shear
SO = Solubility
UC = Unconf. Compress. Strength

PROJECT NO. 03E-064

EARTHTEC ENGINEERING, P.C.

FIGURE NO.: 6

KEY TO SYMBOLS

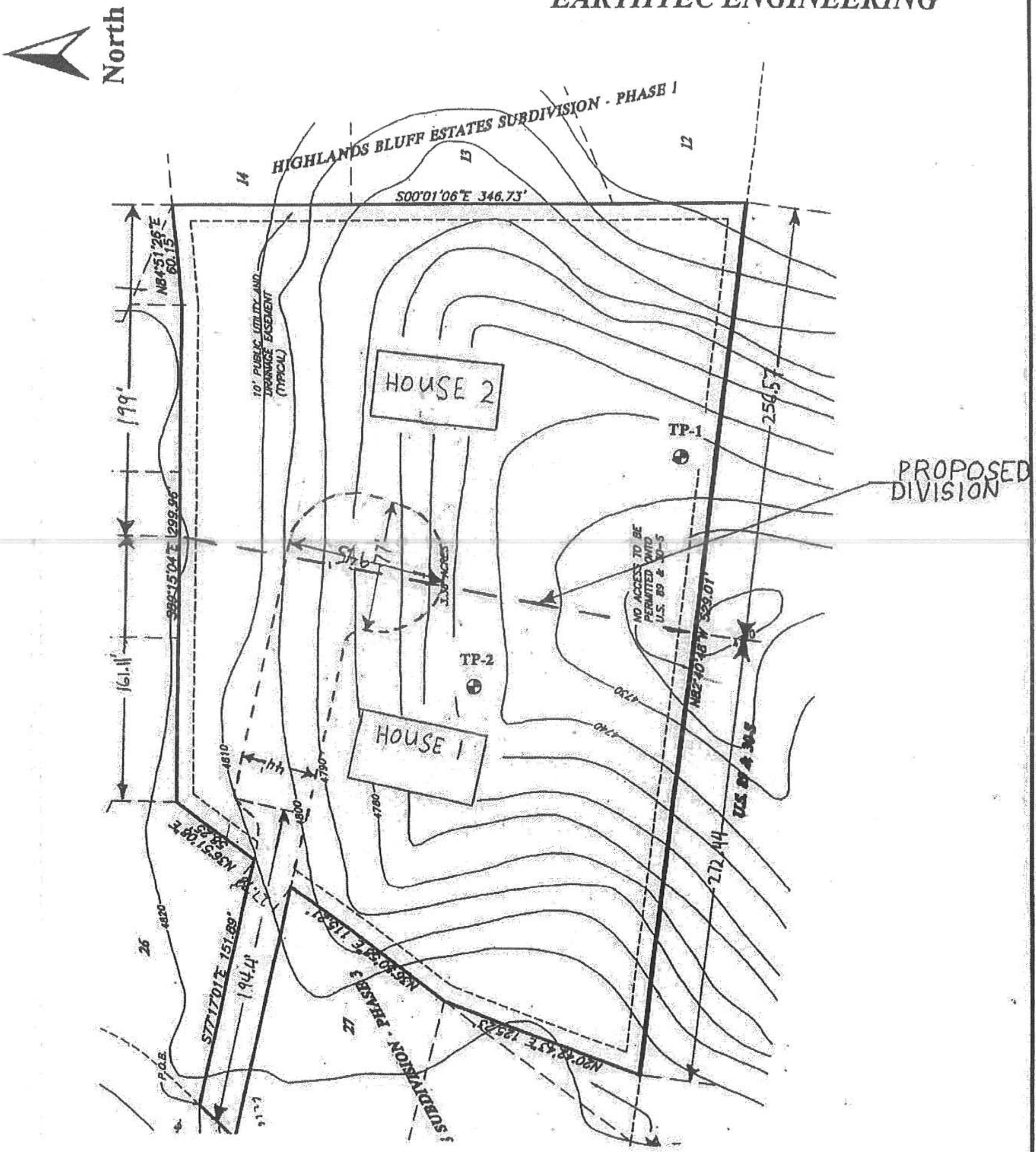
Symbol	Description
Strata symbols	
▒	Topsoil
▥	Silty sand
Soil Samplers	
☒	Block Sample

Notes:

1. Exploratory test pits were excavated on 1-31-03 with a backhoe.
2. Free water was not encountered at the time of excavation.
3. These logs are subject to the limitations, conclusions, and recommendations in this report.
4. Results of tests conducted on samples recovered are reported on the logs.

FIGURE NO.: 7

EARTHTEC ENGINEERING



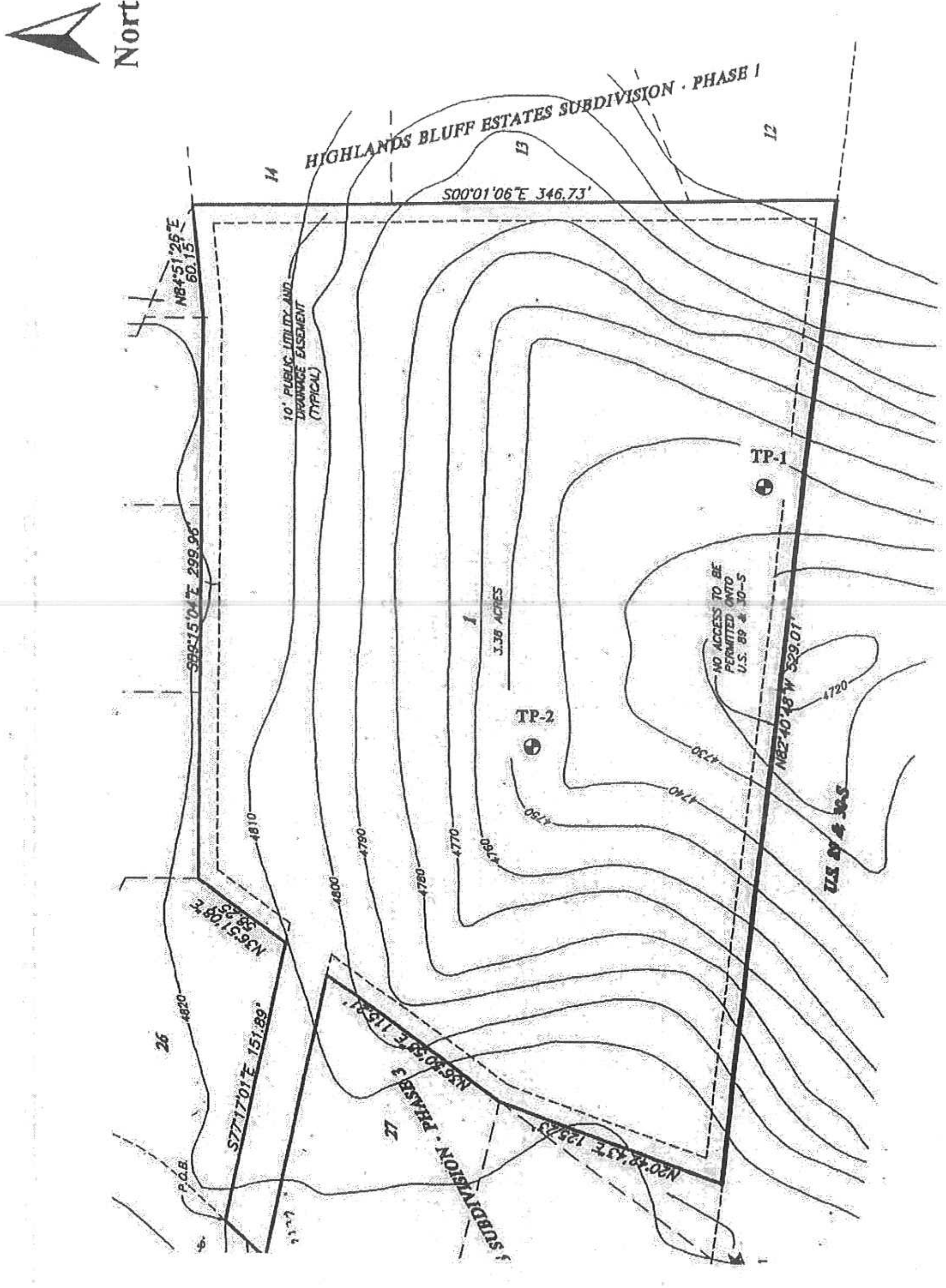
NOT TO SCALE

Base map provided by
Reeve & Associates, Inc.

SITE PLAN SHOWING LOCATION OF TEST PITS

ETE JOB NO. 03E-064 **FIGURE 4**

EARTHTEC ENGINEERING



NOT TO SCALE

Base map provided by
Reeve & Associates, Inc.

SITE PLAN SHOWING LOCATION OF TEST PITS

ETE JOB NO. 03E-064

FIGURE 4

Earthtec Testing & Engineering, P.C.

133 North 1330 West
Orem, Utah - 84057
Phone (801) 225-5711
Fax (801) 225-3363

1596 W. 2650 S. #108
Ogden, Utah - 84401
Phone (801) 399-9516
Fax (801) 399-9842

November 3, 2006

Donald Fulton

Subject: Slope Stability
Hidden Oaks Bluff Estates
Formerly Known as the Kunzler Subdivision
Weber County, UT
ETE Job No. 06-1216

Dear Mr. Fulton;

At your requested we have prepared this letter to present the results of a slope stability analysis conducted for the above referenced property. Based on conversations with you, we understand that it is desired to located a single family residence on the east facing slope of the bowl at the subject site. A previous study performed at the site by Earthtec and Western Geologic (dated February 4, 2003) indicated a risk of slope failure at the site. The letter presents our slope stability analysis of the east facing slope of the bowl at the above referenced site.

As part of our study, a test holes was drilled at the site to a depth of 75 feet. The approximate location of the test hole is indicated on Figure 1. Subsurface conditions encountered within our test hole generally consisted of about 6 feet of fill overlying medium dense poorly graded gravel with sand (GP) to a depth of 23 feet. Below these gravel soils we encountered medium dense silty sand (SM) to a depth of 56 feet, underlain by very stiff to hard silty with sand (ML) to a depth of 66 feet which overlies a very dense poorly graded sand with silt (SP-SM) which extends beyond the maximum depths explored (75 feet). Groundwater was encountered within our test hole at 18 feet below existing grades at the time of this investigation. Graphical representations of the soil conditions encountered in test hole are shown on the Test Hole Log, Figure 2. A legend of the symbols used on the test hole log is shown on Figure 3.

Laboratory testing consisted of moisture content and density determinations, grain size distribution analyses, and a direct shear test. The results of these tests are shown on Figures 2 through 4, attached.

The profile used in our analysis was based on a site plan prepared by Reeve & Associates. The soil strength was based on the above referenced direct shear test. The slope stability was analyzed using the XSTABLE computer program and the Modified Bishop's method of slices. Slopes with safety factors of 1.5 or greater for the static condition is typically considered stable.

Our analysis indicates a factor of safety of 1.1 (see Figure 5)and therefore poses a high risk of slope instability at this site.

If it is desired to proceed with development of this site we recommend measures be taken to increase the stability of the existing slope in the vicinity of the proposed house. There are several stabilization methods commonly used, such as driven piles, soil nails, and horizontal drains. If desired we can prepare recommendations for stabilization; however, we recommend contacting a contractor who specializes in geotechnical construction. A contractor who specializes in this area will have more experience in determining the most cost effective measures and can provide cost estimates for the stabilization measures.

We appreciate the opportunity of providing our services on this project. If we can answer questions or be of further service, please call.

Respectfully;
EARTHTEC ENGINEERING, P.C.

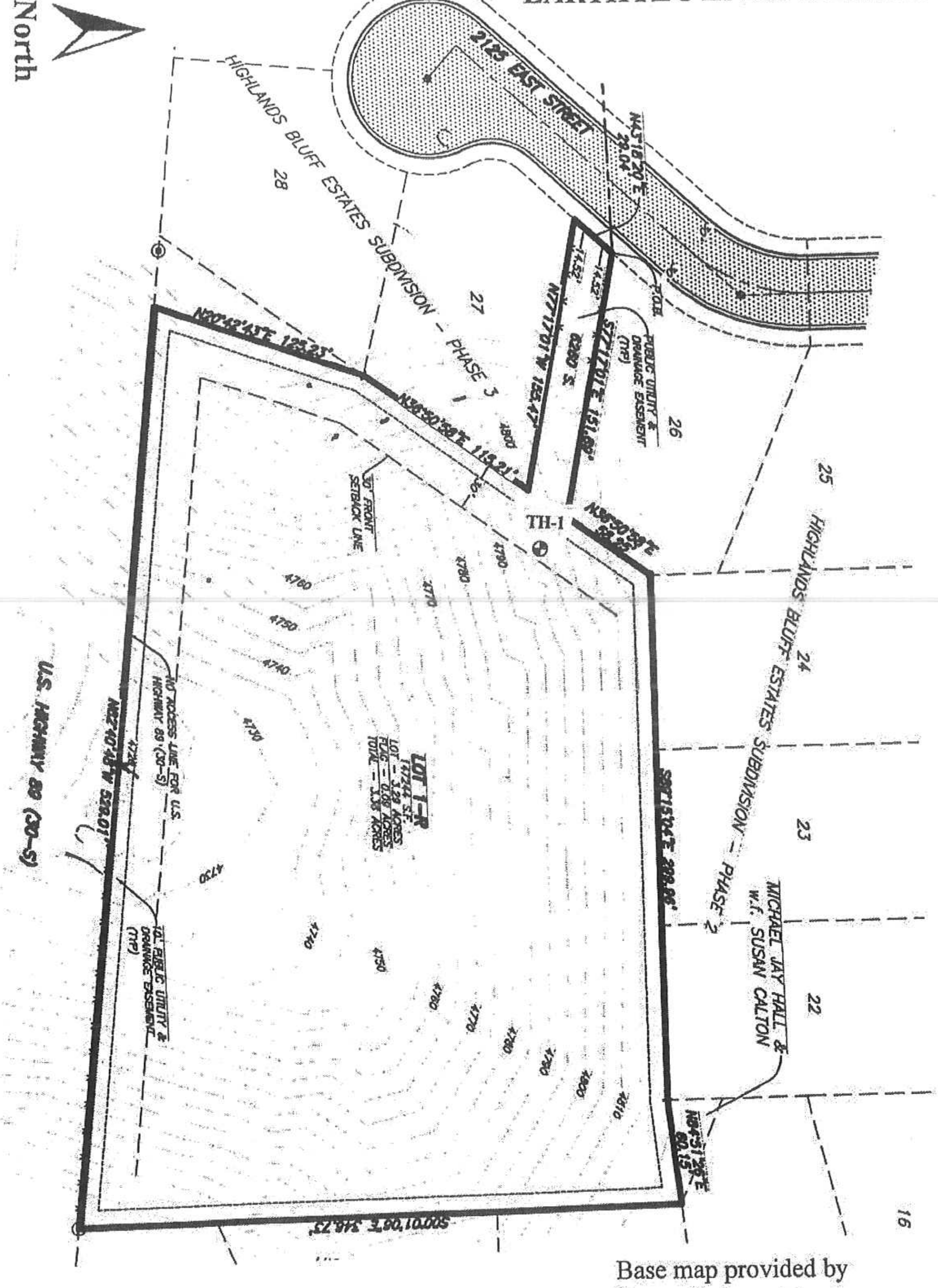
MARK I. CHRISTENSEN
No. 188039

Mark I. Christensen, P.E.
Project Engineer

2 copies sent

Attachments: Figure 1 - Site Plan Showing Location of Test Hole
Figure 2 - Test Hole Log
Figure 3 - Legend of Symbols Used on Test Hole Log
Figure 4 - Direct Shear Test Results
Figure 5 - Stability Analysis Results

EARTHTEC ENGINEERING



NOT TO SCALE

Base map provided by
Reeve & Associates

SITE PLAN SHOWING LOCATION OF TEST HOLE

ETE JOB NO. 06-1216

FIGURE 1

TEST HOLE LOG

NO.: TH-1

PROJECT: Hidden Oaks Bluff Estates
PROJECT NO.: 06-1216
CLIENT: Donald Fulton
DATE: 09/15/06
LOCATION: See Figure 2
ELEVATION: Not Measured
OPERATOR: Raycon Drilling
LOGGED BY: Mark Christensen
EQUIPMENT: Diedrich D-120
DEPTH TO WATER; INITIAL ▽ :
AT COMPLETION ▼ : 18 ft.

Depth (Ft.) 0	Graphic Log	USCS	Description	Samples	TEST RESULTS								
					Blows per foot	Dry Dens. (pcf)	Water Cont. (%)	PI	LL	Gravel (%)	Sand (%)	Fines (%)	Other Tests
			Fill; Silty Gravel with sand - moist, brown										
3													
				◣	27								
6													
			Poorly Graded Gravel with sand - medium dense, moist, brown										
9													
				◣	19								
12													
15		GP											
				◣	23								
18			▼										
			- wet below 18 feet										
21				◣	13								
24			Silty Sand - medium dense, wet, brown										
				◣	11								
27													
30													
				⧓	54								
33													
36				⧓	45								
39		SM											
			- silt and clay lenses below 39 feet	⧓	39								
42													
45													
				⧓	70								
48													
51				⧓	40		30			0.0	73.8	26.2	
54													
				⧓	53								
57			Silt with sand - very stiff to hard, wet, brown										
60		ML		⧓	86		23			0.0	26.8	73.2	DS
63													
66				▬	50/5"								
			Poorly Graded Sand with silt - very dense, wet, brown										
69													
		SP-SM			50/2"		15			0.0	90.8	9.2	
72													
75													
				▬	50/3"								
78													

Notes:

Tests Key
CBR = California Bearing Ratio
C = Consolidation
R = Resistivity
DS = Direct Shear
SS = Soluble Sulfates
UC = Unconfined Compressive Strength

PROJECT NO.: 06-1216

Earthtec
Testing and Engineering, P.C.

FIGURE NO.: 2

LOG OF TESTHOLE 06-1216.GPJ EARTHTEC.GDT 11/3/06

LEGEND

PROJECT: Hidden Oaks Bluff Estates
CLIENT: Donald Fulton

DATE:
LOGGED BY:

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR SOIL DIVISIONS			USCS SYMBOL		TYPICAL SOIL DESCRIPTIONS
COARSE GRAINED SOILS (More than 50% retaining on No. 200 Sieve)	GRAVELS (More than 50% of coarse fraction retained on No. 4 Sieve)	CLEAN GRAVELS (Less than 5% fines)	○	GW	Well Graded Gravel, May Contain Sand, Very Little Fines
			○	GP	Poorly Graded Gravel, May Contain Sand, Very Little Fines
		GRAVELS WITH FINES (More than 12% fines)	○	GM	Silty Gravel, May Contain Sand
			▨	GC	Clayey Gravel, May Contain Sand
	SANDS (50% or more of coarse fraction passes No. 4 Sieve)	CLEAN SANDS (Less than 5% fines)	░	SW	Well Graded Sand, May Contain Gravel, Very Little Fines
			░	SP	Poorly Graded Sand, May Contain Gravel, Very Little Fines
		SANDS WITH FINES (More than 12% fines)	▥	SM	Silty Sand, May Contain Gravel
			▨	SC	Clayey Sand, May Contain Gravel
FINE GRAINED SOILS (More than 50% passing No. 200 Sieve)	SILTS AND CLAYS (Liquid Limit less than 50)		▨	CL	Lean Clay, Inorganic, May Contain Gravel and/or Sand
			▥	ML	Silt, Inorganic, May Contain Gravel and/or Sand
			☰	OL	Organic Silt or Clay, May Contain Gravel and/or Sand
	SILTS AND CLAYS (Liquid Limit Greater than 50)		▨	CH	Fat Clay, Inorganic, May Contain Gravel and/or Sand
			▥	MH	Elastic Silt, Inorganic, May Contain Gravel and/or Sand
			≋	OH	Organic Clay or Silt, May Contain Gravel and/or Sand
HIGHLY ORGANIC SOILS			ψ	PT	Peat, Primarily Organic Matter

SAMPLER DESCRIPTIONS

- ◩ SPLIT SPOON SAMPLER (1 3/8 inch inside diameter)
- ⧗ MODIFIED CALIFORNIA SAMPLER (2½ inch outside diameter)
- ▥ SHELBY TUBE (3 inch outside diameter)
- ◨ BLOCK SAMPLE
- ☒ BAG/BULK SAMPLE

WATER SYMBOLS

- ▽ Water level encountered during field exploration
- ▼ Water level encountered at completion of field exploration

NOTES:
1. The logs are subject to the limitations, conclusions, and recommendations in this report.
2. Results of tests conducted on samples recovered are reported on the logs and any applicable graphs.
3. Strata lines on the logs represent approximate boundaries only. Actual transitions may be gradual.
4. In general, USCS symbols shown on the logs are based on visual methods only: actual designations (based on laboratory tests) may vary.

PROJECT NO.: 06-1216

Earthtec
Testing and Engineering, PC

FIGURE NO.: 3

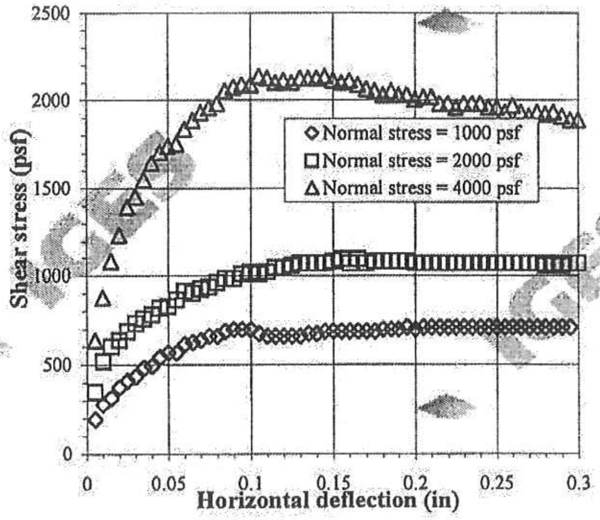
LEGEND 06-1216.GPJ EARTHTEC.GDT 11/2/06

EARTHTEC ENGINEERING

Test type: Inundated
Horizontal deformation (in.): 0.3
Shear rate (in./min): 0.0028

	Sample 1		Sample 2		Sample 3	
Normal stress (psf)	1000		2000		4000	
Peak shear stress (psf)	708		1092		2136	
Horizontal deformation at peak(in)	0.20		0.16		0.11	
	Initial	Final	Initial	Final	Initial	Final
Sample height (in)	1.0000	0.9630	1.0000	0.9528	1.0000	0.9348
Sample diameter (in)	2.416	2.416	2.416	2.416	2.416	2.416
Wt. rings + wet soil (g)	190.50	193.39	192.36	196.21	190.83	191.94
Wt. rings (g)	44.56	44.56	46.54	46.54	45.63	45.63
Wet soil + tare (g)	701.30	165.20	701.30	165.28	701.30	160.38
Dry soil + tare (g)	591.80	134.16	591.80	133.78	591.80	131.60
Tare (g)	152.95	20.97	152.95	22.27	152.95	20.52
Water content (%)	25.0	27.4	25.0	28.2	25.0	25.9
Dry unit weight (pcf)	97.1	100.8	97.0	101.8	96.6	103.3

ϕ (deg)	26	Average of 3 samples	Initial	Final
c (psf)	186	Water content (%)	25.0	27.2
		Dry unit weight (pcf)	96.9	102.0



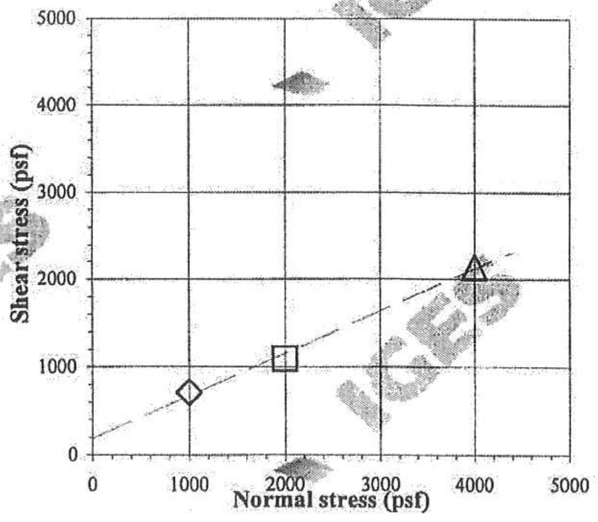


Chart and Graph by IGES

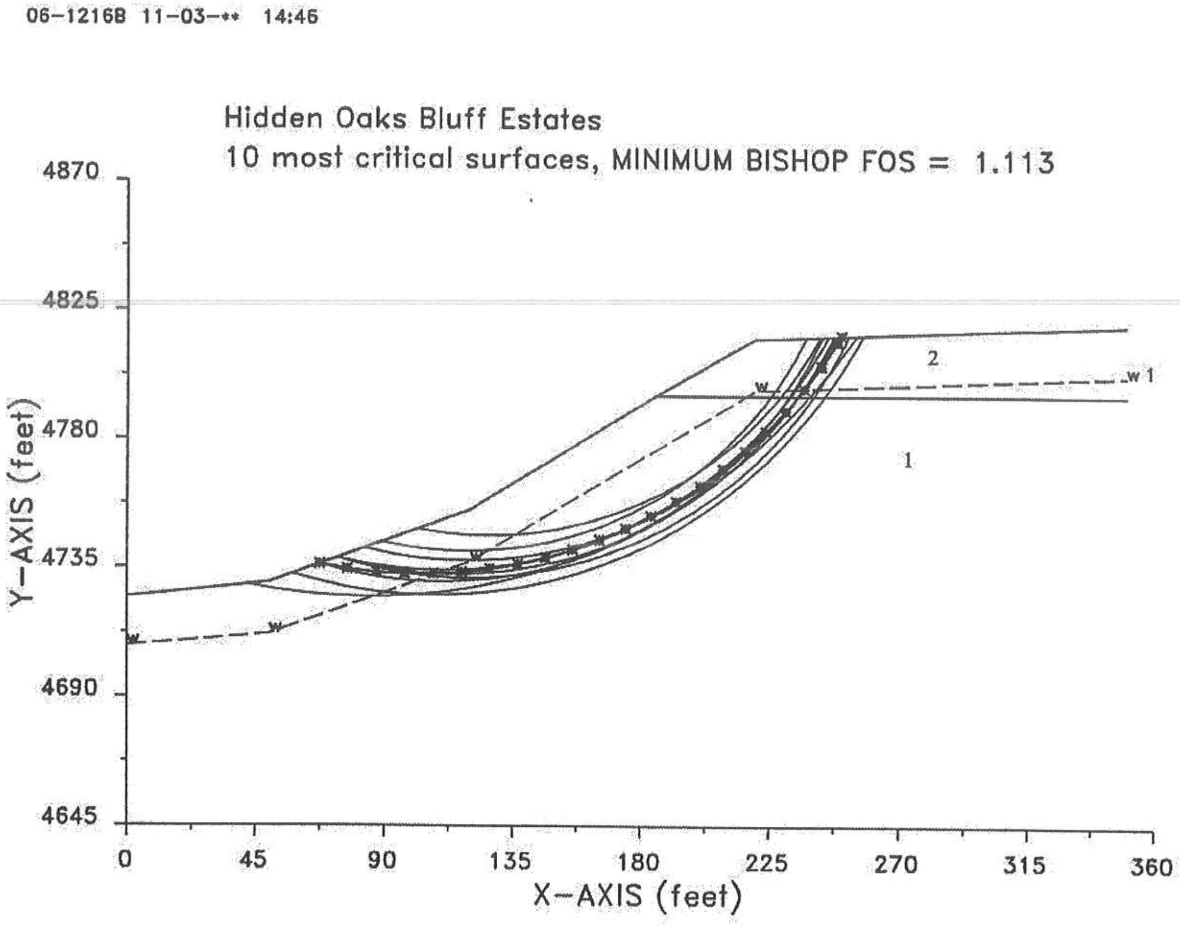
REMOLDED DIRECT SHEAR
(CONSOLIDATED DRAINED)
TH-1 @ 59 ft

ETE JOB NO. 06-1216

FIGURE 4

EARTHTEC ENGINEERING

Soil Layer	*Soil Type*	*Moist Unit Wt. (pcf)*	*Sat. Unit Wt. (pcf)*	*Cohesion (psf)*	*Friction Angle (degrees)*
1	*Sand*	*120*	*130*	*185*	*26*
2	*Gravel*	*125*	*130*	*0*	*35*



GLOBAL SLOPE STABILITY

ETE JOB NO. 06-1216

FIGURE 5