Applied Geotechnical Engineering Consultants, Inc.

GEOLOGIC AND GEOTECHNICAL REPORT

EAGLEPOINTE DEVELOPMENT

SECTIONS 12 AND 13, TOWNSHIP 1N, RANGE 1W

NORTH SALT LAKE, UTAH

PREPARED FOR:

SKY PROPERTIES
585 WEST 500 SOUTH, SUITE 110
BOUNTIFUL, UTAH 84010

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PROJECT NO. 1020320

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EXECUTIVE SUMMARY

Based on the study conducted, our professional opinions are:

1. The site for the proposed Eaglepointe Development is suitable for the proposed residential development.

2. The proposed residences may be supported on spread footings.

3. Rockfall and avalanche are not considered significant geologic hazards for the proposed development.

4. Debris flow is a very low potential hazard for the drainage area which extends through the central portion of the property. Debris flow may be a concern if a significant fire were to occur in the drainage above the site. There is no significant debris flow hazard for the area below the south drainage.

5. Surface fault rupture is a potential hazard for the northwest corner of the site. This hazard has been investigated by others and recommendations to mitigate surface fault rupture hazard were given, (AGRA, September 10, 1999).

   Evidence from site reconnaissance, subsurface investigation and aerial photograph reviews indicate that the Meridian Peak Fault does not pose a hazard to the proposed development.

6. Fill was encountered in 10 of 41 explorations. The fill thickness varied from 1 to over 15 feet. The fill in 6 of the test pits was 6 feet thick, or less.

   The fill encountered generally is not well compacted. Deeper fills were encountered where significant grading has been accomplished. The suitability of fill encountered within building pads should be evaluated and appropriate measures taken to provide suitable support. This should be accomplished by the individual builders.

7. Some of the clay and bedrock encountered at the site exhibits moisture sensitive (expansive) characteristics. When moisture sensitive soil and/or bedrock is encountered, it is important to maintain little or no moisture change in the soil or bedrock below proposed structures.

8. Several active springs and seeps were observed. Subsurface water was also encountered in a few explorations. Good surface and subsurface drainage are important to provide desired performance of the homes.

Executive Summary Continued

9. Ground and bedrock slopes should remain stable. Positive measures should be taken to maintain the stability of the slopes on site.

10. Geotechnical information related to foundations, slopes, subgrade preparation, materials, compaction and pavement design is included in the report.

SCOPE

This report presents the results of a geologic and geotechnical study for the proposed Eaglepointe Development in North Salt Lake, Utah. The site encompasses an area of approximately 279 acres within Sections 12 and 13, Township 1 North, Range 1 West, Salt Lake Base and Meridian. The approximate location of the proposed development is shown on Figure 1.

The study was conducted to evaluate the geologic conditions and potential geologic hazards at and near the site as they relate to the proposed development. The study included a review of geologic literature, an evaluation of aerial photographs, field mapping, subsurface exploration and engineering analysis.

The geology of the site was mapped by geologists from AGEC and compared with previous mapping in the area. Geologic maps and cross-sections of the project area were developed. Our interpretation of the site geologic and hydrogeologic conditions was used to evaluate the potential effect that development of the site would have on adjacent properties and to identify potential geologic hazards that may affect the proposed development.

The study was performed in general accordance with our proposal dated May 7, 2002. Geologic hazards were evaluated in general accordance with guidelines from the Association of Engineering Geologists (Utah Section of the Association of Engineering Geologists, 1986).

A field exploration program was conducted to obtain information on the subsurface conditions. Samples obtained from the field exploration were tested in the laboratory to determine physical and engineering characteristics of the on-site soil and bedrock. Information obtained from the field and laboratory was used to define conditions at the site for our engineering analysis and to develop recommendations for the proposed site development, foundations and pavement.

This report has been prepared to summarize the data obtained during the study and to present our conclusions and recommendations based on the proposed construction and the subsurface conditions encountered. Design parameters and a discussion of geotechnical engineering considerations related to construction are included in the report.

PREVIOUS STUDIES IN THE AREA

AGEC conducted geotechnical investigation and surface fault rupture hazard studies for the Scenic Hills Subdivision Phases 4 and 5, and presented our findings and recommendations in reports dated April 17, 2001 and December 20, 2001, respectively, under AGEC Project No. 1010116. AGEC conducted a hydrogeologic review for the Scenic Hills Subdivision, Phases 4 and 5 and presented our findings in a letter dated January 23, 2002 under the same project number. Scenic Hills Phases 4 and 5 are located along the north side of the proposed Eaglepointe Development.

A geotechnical investigation was performed by AGEC for a water reservoir (Alger Reservoir) located in the north central portion of the proposed Eaglepointe Development. The geotechnical investigation was conducted for the City of North Salt Lake. Our findings and recommendations were submitted in a report dated October 21, 1999 under AGEC Project No. 1990556.

AGEC conducted a geologic investigation for Phase 6 of the proposed Eaglepointe Development which is located along the northeast portion of the proposed development. The geologic investigation was conducted for SKY Properties and our findings and recommendations were submitted in a report dated April 4, 2002 under AGEC Project No. 1010070A.

SITE CONDITIONS

The site consists primarily of undeveloped hillside and areas where sand and gravel have been mined. Initial phases of the Eaglepointe Development (generally the northwest and north portions of the site) have been developed and some houses are currently under construction. Houses which have been constructed or are under construction consist of one to three-story, wood frame structures, some with full depth basements. Asphalt paved roads extend through the northern portion of the proposed development.

The ground surface in the area generally slopes down to the northwest. The site ranges in elevation from approximately 4,500 feet in the northwest corner to 5,600 feet in the southeast corner of the proposed development. The general topography of the site is presented on Figure 4.

Vegetation at the site consists of weeds, sagebrush and patches of trees. Most of the trees are on north aspects of slopes and drainages in areas which have not been disturbed by the past gravel mining operation and recent development.

Undeveloped land extends along the north side of the site. There is residential development northeast of the proposed development. The Eaglewood Golf Course and undeveloped hillside extend along the east side of the proposed development. There is undeveloped land west of the central portion of the site. Reclaimed gravel pit areas extend along the southwest and northwest portions of the property. There are undeveloped hillside and sand and gravel mining facilities south of the site.

GEOLOGIC AND SEISMOTECTONIC SETTING

A. Regional Geology

The site is located on the eastern edge of the Basin and Range Province. The province is made up of north/south elongated mountain blocks and valleys. North Salt Lake is situated on the eastern edge of the Salt Lake Valley. The Salt Lake Valley is one of the valleys in the province with the Oquirrh and Wasatch Mountains partially bounding the valley on the west and east, respectively.

The Salt Lake Valley was once occupied by a large lake known as Lake Bonneville during the Wisconsin Glacial period of the Pleistocene age. The present-day Great Salt Lake is a remnant of ancient Lake Bonneville. Stillstands of Lake Bonneville formed benches along the Wasatch Front. The highest level of Lake Bonneville is marked by a bench, the Bonneville Shoreline, at approximate elevation 5,200 feet. The lake remained at this high level from approximately 17 to 15 thousand years before present (bp), until it dropped approximately 350 feet during a catastrophic flood known as the Bonneville Flood (Currey and Oviatt, 1985; Jarrett and Malde, 1987). Two lower stillstands of Lake Bonneville are the Provo and Gilbert levels, which formed at approximate elevations 4,850 and 4,250 feet, respectively.

The site ranges in elevation from approximately 4,500 feet in the northwest corner to 5,600 feet in the southeast corner. Three unnamed, intermediate Bonneville shorelines are mapped in the project area from approximate elevations 4,960 to 5,140 feet (Figure 2). The highest, most prominent Bonneville shoreline is mapped near elevation 5,200 feet in the project area. The Provo shoreline is at approximate elevation 4,820 feet across the site. The Bonneville shorelines mapped by Personius and Scott (1992) can be identified on the 1937, 1952, 2000 and 2001 aerial photographs reviewed for the project.

B. Tectonic Setting

The site is located on the east side of the Salt Lake Valley along the Wasatch Front, which is a prominent mountain front escarpment extending approximately 240 miles from near Malad, Idaho to the vicinity of Fayette, Utah. The prominent west facing steep escarpment of the Wasatch Mountain Front is the result of repeated normal fault displacements which have taken place over the last several million years. The system of normal faults which makes up this escarpment is known collectively as the Wasatch Fault. Relatively recent fault movements are evidenced by offsets in Lake Bonneville sediments and more recent alluvial and colluvial deposits.

The Wasatch Fault is considered to be made up of several segments, each segment acting relatively independently (Machette et al., 1992). The site is located along the northern end of the Salt Lake City segment of the Wasatch Fault, which is a segment approximately 23 miles in length that extends from North Salt Lake to Draper, Utah.

The Wasatch Fault is seismically active. The average recurrence interval of surface faulting on the Salt Lake City segment in the past 6,000 years is 1,350 years plus or minus 200 years (Black, et al., 1996).

The northern portion of the Salt Lake City segment of the Wasatch Fault is known as the Warm Springs Fault. The fault extends along the western flank of the Salt Lake salient, however, the north and south extent of the fault are uncertain (Personius and Scott, 1992). The most recent mapping of this fault was performed by Personius and Scott (1992). According to their work, the fault extends northward south of the project area and then northeastward through the northeast corner of the project area before dying out approximately 1,200 feet north of the site along the northern flank of the Salt Lake Salient. Figure 2 is based on mapping by Personius and Scott (1992) and shows the approximate location of the Warm Springs Fault, which is mapped as a single trace through the northwest corner of the site.

The Warm Springs Fault is mapped as extending through the northwestern corner of the site as shown on Figures 2 through 5. The fault location shown on Figures 4 and 5 was interpreted from the 1937 aerial photographs, previous mapping and site reconnaissance.

The 1937 aerial photographs reviewed for the project do not predate gravel mining in the area. The surface trace of the fault can be identified on the photographs. Gravel mining operations in the area of the Orchard Hills Subdivision and along the fault scarp west of the site obscured the fault scarp in places by 1937. No other fault scarps were identified near the site based on a review of the photographs.

Several small faults with offsets between 4 inches to greater than 3.5 feet were observed in a utility trench during a surface fault rupture hazard study by representatives from AGRA (1999). The majority of these faults were considered to be inactive for several reasons including the age of the faulted deposits (approximately 40 million years old), location of the faults to the east of the main trace of the Wasatch Fault and the nature of the faulting direction. Many of the faults were antithetic to the main fault trace which is uncommon in footwall blocks. However, AGRA felt that a few of the faults may be active or may be reactivated during an event on the Warm Springs Fault because they are parallel or subparallel to the main trace, and one of the faults contained soft fault gouge (AGRA, 1999). These faults are shown on Figures 4 and 5 in the northwest portion of the site, east side of the trace of the Warm Springs Fault.

A Tertiary-aged bedrock fault (Meridian Peak Fault)was mapped by Van Horn (1981) and Van Horn and Crittenden (1987) to extend through the eastern edge of the property. This fault juxtaposes a younger, Tertiary-aged conglomerate with older, Tertiary-aged volcanics and conglomerate. Its sense of motion is down to the east, opposite that of the Wasatch Fault. It is not mapped on Quaternary surficial maps (e.g. Personius and Scott, 1992) and is not considered active.

C. Structure

Van Horn (1981) mapped the bedrock geology of the project area at a 1:24,000 scale. The bedrock in this area consists of Tertiary-aged volcanic, tuffaceous and sedimentary rock. Other than in areas where gravel mining has removed the unconsolidated overburden, most of the bedrock is overlain by lacustrine and alluvial deposits. The bedrock geology of the site based on mapping by Van Horn (1981) is presented on Figure 3.

Surficial and bedrock geologic maps of the project area were developed based on mapping by Van Horn (1981, 1987), Personius and Scott (1992), reports by Terracon (1998) and AGRA (November 1997, November 1998, February 1999), our field mapping and exploration, interpretation of aerial and site photographs (see Appendix A). The surficial geologic map is presented on Figure 4 and the bedrock geologic map of the project area is presented on Figure 5. Cross-sections A-A' through E-E' portraying interpreted bedrock geology are shown on Figures 6 through 10. The exposure of bedrock and areas where the attitude of the bedrock could be measured are very limited. The cross-sections represent a generalized model of bedrock structure at the site. The cross-sections presented were selected to model the worst case scenario for groundwater flow toward the northwest corner of the site. Other models are possible which would result in less available groundwater for the area near the northwest corner of the site.

Bedrock units generally strike north/northwest in the project area. Two folds are mapped by Van Horn (1981); a syncline and an adjacent anticline. The folds plunge to the northwest. Geologic mapping by AGEC generally found geologic conditions to be similar to those described in the literature.

D. Stratigraphy

The stratigraphy of the bedrock units in the project area consists of Tertiary sedimentary rocks overlain by Tertiary tuffaceous deposits and volcanic breccia, in part overlain by a younger Tertiary-aged conglomerate. Quaternary deposits consist primarily of granular, lacustrine deposits related to Lake Bonneville. Personius and Scott (1992) map post-Bonneville alluvial fans in the southeastern portion of the site. The bedrock and overlying lacustrine and alluvial deposits modified from previous mapping by Van Horn (1981) and Personius and Scott (1992) and used in our investigation are described (generally in order of most recent to oldest) below.

1. Hillslope colluvium (Holocene to upper Pleistocene) — (hc) Gravel, cobbles and boulders, usually clast-supported, in a matrix of sand and silt, poorly sorted.

2. Fan alluvium (middle Holocene to uppermost Pleistocene) — (af) Clast-supported gravel and cobbles, locally boulders, in a matrix of sand and silty sand, poorly sorted.

3. Lake Bonneville deposits, undifferentiated (Quaternary) — (lbs, lbg) These surficial deposits cover much of the undisturbed portion of the project area. They consist predominantly of interbedded silt, sand, gravel and cobbles; silt and sand are light brown, gravel and cobbles are light to medium gray.

4. Pre- Bonneville lacustrine and alluvial deposits, undifferentiated (Quaternary)— (la/Qu) In areas west of the Warm Springs Fault where much of the younger soil has been mined and removed, lacustrine and alluvial deposits are exposed. These deposits consist predominantly of sand, silt and clay.

5. Conglomerate No. 2 (Tertiary) — (Tc2) This unit is not well-exposed in the project area. It is mapped adjacent to older rocks due to the Meridian Peak Fault mapped by Van Horn and Crittenden, (1987) which is inferred to be late Tertiary in age. Conglomerate No. 2 consists of pale-brown red and light gray conglomerate and sandstone and with some well-cemented layers. The clasts in this unit are fractured.

6. Volcanic breccia (Tertiary) — (Tv) Outcrops of this unit were observed northwest of the site in the area east of Springhill Drive where past gravel mining operations have exposed bedrock. The unit is also exposed in drainages at the north boundary of the site. The andesitic volcanic breccia is dark-gray to purplish-gray and slightly to moderately weathered.

7. Tuffaceous deposits (Tertiary) — (Tt) The light gray to light brown deposits of claystone, siltstone and sandstone containing volcanic ash are almost exclusively exposed in cut or extensively mined areas at the site. The degree of weathering of this unit varies from slightly weathered to highly weathered. The finely bedded claystone appears to weather to high plastic clay. Two red siltstone and sandstone layers were also observed within the tuffaceous deposits. They are exposed across much of the mined area and contain some claystone and clayey gravel. They are predominantly massive with very little bedding, and in some places, the claystone has weathered to medium to high plastic clay.

8. Conglomerate No. 1 (Tertiary) — (Tc1) Conglomerate and sandstone bedrock is exposed in the cliffs southwest of the site and in cuts and outcrops in the southern portion of the site. This is the oldest of the bedrock units exposed in the project area. The unit is pale brown to medium gray and well-cemented. Clasts are medium to dark gray limestone and light gray to tan quartzite.

E. Springs

Several active springs and seeps were observed during the site reconnaissance. The approximate locations of springs identified during our site reconnaissance are shown on Figures 4 and 5. Several areas which appear to have had water flow in the past were identified during the site visits and the locations of these areas are labeled as springs on Figures 4 and 5. The occurrence of the springs does not appear to be related to faulting but to areas where the ground surface intercepts the contact between Lake Bonneville sediments and low permeable bedrock or the contact within the tuffaceous deposits where higher permeable layers containing sands and gravels are underlain by lower permeable bedrock. Previous mining in the area generally removed the gravel down to the bedrock in the northwest portion of the property. Since the groundwater is typically perched on the bedrock, springs have been created in the mined areas.

GEOLOGIC HAZARDS

Areas of potential geologic hazards were mapped on a regional level by Davis County in a series of maps including Potential Surface-Fault Rupture Sensitive Area Overlay Zone, Debris-Flow Hazard Special Study Zone Map, Landslide Hazard Map and Slope-Failure Inventory Map by Mike Lowe (1989). These maps were developed to assist city and county planners in determining areas which may need more detailed study of potential geologic hazards as these areas are considered for development. Based on the maps; a more detailed study on portions of the proposed development is suggested for surface fault rupture, landslide and debris flow. Rockfall and avalanche are not considered potential hazards for the proposed development.

A. Surface Fault Rupture Hazard

1. Warm Springs Fault

In 1999, AGRA observed geologic conditions in a utility trench investigation within Eaglepointe Drive (AGRA, 1999). The trench extended roughly north/south from approximately 200 feet southeast of the mapped trace of the Warm Springs Fault to approximately 100 feet outside of the Davis County Potential Surface-Fault Rupture Sensitive Area Overlay Zone. Several normal faults with displacements ranging from less than four inches to greater than 3.5 feet were observed in the roughly 40 million year old bedrock deposits (Van Horn, 1981). The younger lacustrine sediments had previously been removed by gravel mining, so the approximate age of the displacement event(s) could not be determined. AGRA's report listed evidence that supported both active and inactive faulting. The report concluded "that it would be prudent to avoid construction over the larger offset faults that strike nearly parallel to the most recent fault trace...in the unlikely case that they are active or are reactivated during a future earthquake event". Faults with small offsets and trends striking at a high angle to the Warm Springs Fault were not considered to be active by AGRA.

The Wasatch Fault (Warm Springs Fault in this area) extends through the northwest corner of the site. We understand that the development has complied with the AGRA recommendations in this area.

2. Meridian Peak Fault

While not within the Davis County potential surface fault rupture sensitive overlay zone (Lowe, 1989), the Meridian Peak Fault (Van Horn and Crittenden, 1987) crosses the east side of the property (Van Horn, 1981). There is no fault scarp or evidence of lineaments on the property associated with this fault based on a review of aerial photographs and site reconnaissance. We

anticipate that the Meridian Fault was mapped by Van Horn based on structural and stratigraphic evidence, likely north and south of the site. The fault has been queried in the North Salt Lake area. The variations in stratigraphy and structure of the bedrock in the North Salt Lake area could be mapped without a fault. The contact between the younger overlying Tertiary conglomerate is unconformable to the older, underlying Tertiary volcanics (Van Horn, 1981). The later Van Horn (1982) surficial geology map does not map the Meridian Peak Fault extending south of the property and shows it as a queried fault through the northern portion of the property. Van Horn (1982) indicates that faults are "shown only where surficial deposits appear to have been involved in the faulting." Personius and Scott (1992) show no mapped fault extending through the property in the area of the Meridian Peak Fault.

The mapped location of the fault was determined by overlay onto the site plans for the project as accurately as practical. An exploratory trench was excavated June 26 and 27, 2002 in a general east/west direction across the mapped location of the fault shown on Figures 4 and 5. The location of the Meridian Peak Fault is questionable and may not have been mapped at its actual location. The trench was excavated to a length of approximately 275 feet and a maximum depth of approximately 12 feet. A level line and stationing were established on the south face of the trench and the trench was logged at a scale of approximately 1 inch equals 5 feet. The log of the exploratory trench is presented on Figures 11 through 13 with a description of soil units and symbols on Figure 14.

The soil encountered in the exploratory trench consists of alluvial deposits overlying Lake Bonneville gravels. The alluvial deposits are silty to sandy lean clay with reworked gravel from the underlying Lake Bonneville sediments. The Lake Bonneville gravels are horizontally bedded and have calcareous deposition on the underside of the clasts.

No evidence of sediment deformation or offsets due to faulting was observed to the depth excavated. Lake Bonneville sediment, which is older than Holocene age, was encountered the full length of the trench. Thus, there is no evidence of active faulting extending through the area trenched. This result is consistent with previous work performed by AGEC (2001) for this same fault in the Scenic Hills Subdivision to the north in which no evidence of faulted Bonneville sediments were encountered.

Three conclusions can be reached concerning the Meridian Peak Fault based on our study. First, the fault offsets bedrock and not Quaternary sediments through the property. Second, the fault offsets bedrock and Quaternary sediments but not Lake Bonneville based on the lack of scarps or lineaments at the ground surface. Third, there is no Meridian Peak Fault through the site. Based on evidence from site reconnaissance, subsurface investigation and review of aerial photographs and geologic literature, it is our professional opinion that the Meridian Peak Fault (whether it exists or not) does not pose a hazard to the proposed development since there is no evidence of Holocene fault activity in this area.

3. Other Areas of Faulting

No other areas of potential surface fault rupture hazard were identified for areas proposed for building construction based on our review of aerial photographs, geologic literature and site reconnaissance.

B. Landslide Hazard

The northwestern portion of the site is in the potential landslide hazard area mapped by Lowe (1989). The potential landslide hazard area also incorporates the natural drainage that extends northwest through the central portion of the site. The extreme southeastern corner of the property is included in the mapped potential landslide

hazard area (Lowe, 1989). The area designated include slopes that are generally steeper than 3:1 (H:V)

Based on a study of aerial photographs which predate most mining at the site, there is no evidence of landslide in areas which have not had gravel mining or in areas that have not been disturbed. Slope failure has been reported in poorly compacted fill during site grading. The area of failure was removed. There are small slumps which have formed in cuts into weathered tuffaceous bedrock in the south central portion of the property. The stability of these slopes was evaluated and recommendations for slopes are included in the Slope Stability section of this report.

C. Debris Flow Hazard

1. South Alluvial Fan

The general area of the south alluvial fan mapped on Figure 2 as "af2" (and Figure 4 "af")in the southeastern portion of the site is located in the potential debris flow hazard area mapped by Lowe (1989). The remainder of the site is not in areas mapped by Lowe (1989) as having debris flow hazard. There is a poorly defined alluvial fan north of the fan mapped by Lowe (1989).

The alluvial fan mapped by Lowe (1989) and the north drainage fan can be identified on aerial photographs reviewed for the project. Both fans are mapped by Personius and Scott (1992) as older fan deposits of middle Holocene to upper most Pleistocene age (> 5000 years old). The fans show no evidence of significant recent deposition based on field observations and a review of aerial photos dated 1937, 1952, 2000 and 2001.

The south fan has a small source area and would not be expected to produce significant debris. It is our professional opinion that typical surface erosion protection methods would sufficiently handle sediment runoff for this drainage. In our professional opinion, debris flow is not a significant hazard for the south alluvial fan area.

2. North Alluvial Fan

The north fan (see Figure 2 "af2" and Figure 4 "af") is at the mouth of a more extensive drainage. The fan has been incised and the drainage continues through the site to the northwest. Most deposition for this drainage appears to have occurred approximately 1 mile to the northwest near the Wasatch Fault scarp. The northwest alluvial fan deposits have been removed by gravel mining operation.

The drainage is approximately 6,000 feet long southeast of the upper fan and rises approximately 800 feet in elevation. The gradient of the channel from the head to the mouth of the drainage is approximately 7.6 degrees. A reconnaissance of the north drainage indicates that the channel is in a relatively stable condition. Most of the channel is vegetated with grass and trees. There are no significant signs of erosion in the channel or along the slopes above the channel. There are very few cobbles and boulders in the channel suggesting the slopes are fairly permeable and do not concentrate significant water flow in the channel.

The average channel gradient is slightly flatter than the low end of what is considered deposition angles (8 to 12 degrees) for channeled debris flows (Hungr et al., 1984). This would indicate that deposition rather than erosion should occur for most of the channel in the north drainage.

Very hot fires can temporarily reduce the permeability properties of surface soil resulting in reduced infiltration and increased surface runoff during a storm. Thus, a fire which burns a large area of the drainage could result in potential sediment production from the drainage.

For the north fan area the debris flow hazard is expected to be very low. The very low hazard would only apply if a significant fire were to occur in the drainage above the site.

SUBSURFACE FIELD STUDY

The field study was conducted in June through August 2002. Forty test pits were excavated throughout the site. The test pits were excavated at the approximate locations indicated on Figure 15. The test pits were excavated with a rubber-tired backhoe. The test pits were logged and soil samples obtained by a geologist from AGEC. The test pits were backfilled without significant compaction. The backfill in the test pits should be properly compacted where it will support proposed buildings, floor slabs or pavement.

One boring was drilled on August 28 and 29, 2002. The boring was drilled in the northeast portion of the site at the approximate location indicated on Figure 15. The boring was drilled with 8-inch diameter hollow stem auger powered by an all-terrain drill rig. The boring was logged and soil samples obtained by a geologist from AGEC. The boring was drilled at this location to help define the bedrock geology.

Samples of material exposed in hillside cuts were obtained by hand-sampling techniques. Locations of the area where samples were obtained are shown on Figure 15 and on cut profile included in Appendix A.

The cut profile, included in Appendix A, was developed from a photograph obtained from North Salt Lake City's review consultant and our observation and sampling of the materials on the slope.

Logs of the subsurface conditions encountered in the test pits and borings are graphically shown on Figures 16 through 19 with legend and notes on Figure 20.

SUBSURFACE CONDITIONS

The subsurface conditions encountered at the site generally consist of topsoil overlying clay, silt, sand and bedrock. Up to approximately 1 foot of topsoil was encountered at the site. Up to approximately 9 feet of fill was encountered in Test Pits TP-4, 8, 11, 12, 15, 17, 19, 26 and 35. Fill extends the full depth investigated, approximately 15 and 6 feet below the adjacent ground surface in Test Pits TP-13 and TP-26, respectively.

The natural soil consists of clay, silt, sand and gravel. Clay, silt and sand was generally encountered in the central portion of the proposed development. Fat clay was encountered in test pits along the south central portion of the proposed development. Gravel was generally encountered along the northeast and southwest portions of the site.

Bedrock was generally encountered along the west side of the proposed development. Another area of relatively shallow bedrock was encountered along the east central portion of the site. Some bedrock was encountered in other areas of the site.

A description of the subsurface soil and bedrock along with their index properties and consolidation/expansion characteristics follows:

Fill - The fill consists of lean clay with small to moderate amounts of sand and gravel and occasionally clayey gravel. Silty sand with gravel was encountered in Test Pit TP-

13. Pieces of asphalt, wood and debris were encountered in Test Pit TP-12. The fill is slightly moist, mottled and brownish orange, brown and red in color.

Laboratory tests conducted on samples of the fill indicate that it has moisture contents ranging from 7 to 9 percent. Results of gradation tests conducted on relatively small samples of the fill are presented on Figures 35 and 36.

Specific information of the fill encountered is presented below:

TP-4

There was approximately 7½ feet of fill in this test pit. The fill appears to have been placed as grading fill. A topsoil layer with large tree roots was observed below the fill. The roots appear to be from scrub oak. It also appears as though the trees had been stripped off of the surface. Fill was composed of brownish-gray silt with gravel and cobbles. It was loose to slightly stiff.

TP-8

There was approximately 3 feet of fill in this test pit. The excavation was performed at the foot of the slope. It appeared to be fill placed for grading. Fill was composed of orange-brown clayey gravel. It was soft and loose.

TP-11

There was approximately 2 feet of fill in this test pit. The excavation was performed at the back of the lot. It appeared that the fill was placed for grading. Fill was composed of brownish-gray clay with gravel. It was loose to slightly stiff.

TP-12

There was approximately 3 feet of fill in this test pit. The excavation was performed at the toe of the slope. There was significant garbage and debris found in the excavation.

TP-13

Approximately 15 feet of fill was observed in this test pit. The excavation was performed in the middle rear of the lot. This location was approximately even with the back of the houses on either side of the lot. At the time of the field work, there were several piles of fill which had been dumped at the rear of the lot. The material was loose through the entire depth of the excavation. The test pit collapsed to a depth of 6 feet prior to backfilling. It appears that the fill was placed to fill the drainage. There may be up to 20 feet of fill at this location. The fill was composed fo clayey gravel to clayey sand.

TP-15

There was approximately 1 foot of fill in this test pit. The excavation was performed at the rear of the lot. The fill was probably placed for grading purposes. The fill was composed of clay with gravel and was soft and loose.

TP-17

There was approximately 1 ½ feet of fill in this test pit. The excavation was performed at the toe of the slope near the rear of the lot. It appeared that the fill was placed for grading purposes. The fill was a soft, red clay with gravel and was placed over topsoil.

TP-19

There was approximately 6 feet of fill in this test pit. The excavation was performed in the area of the large cut slope in the west middle of the property. There were multiple piles of loosely dumped fill throughout the area. The fill was a dense, gray-brown clayey gravel.

TP-26

There was approximately 6 feet of fill in this test pit. The excavation was performed near the toe of the large cut slope in the west middle of the property. The test pit continually caved in restricting the excavation from going deeper. The fill was mostly composed of a poorly graded gravel with clay and sand. It was very loose.

TP-35

There was approximately 9 feet of fill in this test pit. The fill appeared to be loosely placed material related to previous gravel mining operations. The fill was composed of a slightly dense clayey gravel.

Based on observations of the fill during excavation of test pits for the field study portion of our investigation, the fill is poorly to moderately compacted.

Topsoil - The topsoil consists of lean clay and silt and contains small amounts of sand and occasional gravel. The topsoil is slightly moist, brown to dark brown in color and contains roots.

Lean Clay - The clay contains small to moderate amounts of sand and occasional silt layers and gravel. It is medium stiff to very stiff, slightly moist to wet and brown, gray and red in color with some iron oxide staining.

Laboratory tests conducted on samples of the clay indicate that it has natural moisture contents ranging from 4 to 43 percent and natural dry densities ranging from 75 to 122 pounds per cubic foot (pcf).

Consolidation tests conducted on samples of the lean clay indicate that it will compress a small to moderate amount with the addition of light to moderate loads. The results of consolidation tests conducted on samples of the clay are presented on Figures 23, 26, 27 and 32.

Fat Clay - Some lean clay layers were encountered in the fat clay. The clay is stiff to very stiff, slightly moist to moist and red and gray in color with white calcareous deposits.

Laboratory tests conducted on samples of the fat clay indicate that it has natural moisture contents ranging from 16 to 33 percent and natural dry densities ranging from 84 to 108 pcf.

Consolidation tests conducted on samples of the fat clay indicate that it will compress a small to moderate amount with the addition of light to moderate loads. The sample obtained from Test Pit TP-39 at a depth of approximately 2 feet was found to exhibit moisture sensitive (expansive) characteristics. The sample swelled approximately 1.1 percent when wetted at a constant pressure of 1,000 pounds per square foot (psf). A swell pressure of approximately 3,000 psf was measured. Results of consolidation tests conducted on samples of fat clay are presented on Figures 30 and 31.

Interlayered Silt and Lean Clay - The interlayered soil contains small amounts of sand. It is medium dense/medium stiff to very stiff, slightly moist and brown to gray in color.

Clayey Sand - The clayey sand contains occasional gravel. It is medium dense, slightly moist to moist and brown, orangish brown and light gray in color. The clayey sand is calcareous.

Laboratory tests conducted on samples of the clayey sand indicate that it has natural moisture contents ranging from 7 to 8 percent and a natural dry density of 91 pcf. Results of a gradation test conducted on a sample of the clayey sand are presented on Figure 37.

Silty Sand - The silty sand contains small to moderate amounts of silt and occasional gravel and cobbles. It is medium dense, slightly moist and brown to orangish brown in color.

Laboratory tests conducted on a sample of silty sand indicate that it has a natural moisture content of 6 percent and a natural dry density of 94 pcf.

Poorly Graded Sand - The sand contains some silt, gravel, cobbles and boulders up to approximately 2 feet in size. It is medium dense, slightly moist and light brown to brown in color with occasional iron oxide staining. Slightly cemented sand was encountered in Test Pits TP-20 and TP-25.

Laboratory tests conducted on a sample of the sand indicate it has a natural moisture content of 2 percent. Results of a gradation test conducted on a relatively small sample of the sand are presented on Figure 35.

Poorly Graded Gravel - The gravel contains small to moderate amounts of silt and sand, cobbles and occasional boulders up to approximately 3 feet in size. It is medium dense, slightly moist and brown to brownish red in color with occasional slightly cemented gravel layers.

Laboratory tests conducted on samples of the gravel indicate that it has natural moisture contents ranging from 1 to 2 percent. Results of gradation tests conducted on samples of the gravel are presented on Figures 34 and 36.

Tuffaceous Bedrock - The bedrock consists primarily of claystone and siltstone, some sandstone and minor conglomerate. It is slightly to highly weathered, contains small to moderate amounts of sand and has medium to high plasticity. The bedrock is firm to very hard, slightly moist to wet and primarily gray to white in color with occasional red areas.

Laboratory tests conducted on samples of the bedrock indicate natural moisture contents ranging from 15 to 30 percent and natural dry densities ranging from 83 to 111 pcf.

Consolidation tests conducted on samples of the bedrock indicate that it exhibits moisture sensitive (expansive) characteristics and that it will compress a small amount with the addition of light to moderate loads. The samples were observed to swell up to approximately 1.9 percent when wetted at a constant pressure of 1,000 psf. A maximum swell pressure of up to approximately 4,000 psf was measured. Results of the consolidation tests are presented on Figures 21, 22, 24, 25, 28 and 29.

Conglomerate and Sandstone Bedrock - The bedrock is poorly to well cemented, hard to very hard, slightly moist to wet, brown/reddish brown and gray in color. The bedrock contains clasts of limestone and quartzite. Some of the clasts are fractured.

Results of the laboratory tests are summarized on Table I and are included on the logs of the exploratory test pits and boring, Figures 16 through 19. Legend and notes of the test pits and boring are presented on Figure 20. Results of laboratory tests conducted on samples obtained from the hand excavations are presented on Table I.

Efforts to determine a relationship between the on-site soil and the bedrock units and surficial deposits is difficult at best. Much of the soil at the site is composed of Lake Bonneville sediment and younger alluvial soil. These sediments unconformably overlie Tertiary bedrock at the site and most did not originate from this bedrock. The Bonneville sediments and younger alluvium generally originate from offsite sources. For example, Lake Bonneville gravel clasts are generally quite different from clasts derived from the conglomerates (Tc1 and Tc2) within the area. Thus, for much of the site there is no correlation between the underlying bedrock and the soil.

The high plastic clay encountered in Test Pits TP-37 and TP-39 likely represents residual soil derived from the underlying tuffaceous bedrock. There appears to be some thin residual soil overlying the tuffaceous bedrock in areas where the Bonneville sediments have been removed due to previous mining activity although the thickness of residual soil appears to be relatively thin.

The characteristics of the on-site soil and bedrock have been used to develop and present recommendations (as contained in this report) for design and construction at the site.

SUBSURFACE WATER

In most cases free water was not encountered in the test pits at the time of excavation to the maximum depth investigated. Free water was observed in five test pits at depths ranging from approximately 3 to 10 feet below the adjacent ground surface at the time of excavation. Free water was encountered in Boring B-1 at a depth of approximately 35 feet at the time of drilling. Free water was encountered at depths of approximately 11, 2, 5½ and 7½ feet below the adjacent ground surface in Test Pits TP-2, 8, 17 and 40, respectively, when checked 127 to 131 days after excavation. No water was encountered in Test Pit TP-5 when checked 130 days after excavation.

Slotted PVC pipe was installed in Test Pits TP-2, 5, 8, 17 and 40 to facilitate future measurement of the water level. Fluctuations in the water level will occur over time and can be measured within the PVC pipe.

SLOPE STABILITY

Portions of the Eaglepointe Development have been rough graded prior to our study. Grading has resulted in slopes of 2:1 (horizontal to vertical) and flatter with a height up to 175 feet.

The materials exposed in the cut slopes consists of bedrock, sand, gravel and clay.

Water was observed seeping out of some of the cut slopes where the ground surface intercepts permeable materials overlying less permeable materials as shown on Figure 4. The water seepage is the result of perched water.

There is a cut and fill slope up to 350 feet in height near the lower elevation area of the development. The slope is as steep as 2:1 with an overall slope of 2.1:1 which includes the benches incorporated in the slope. We understand that during mining operations, seepage water was frequently encountered. We also understand that as the gravel mining operations proceeded and as the area was reclaimed, areas of water seepage were provided with a drainage system to facilitate collection and discharge of seepage water.

A. Slope Performance

1. <u>On Site</u>

   At the time of our field investigation, there were areas on the 2:1 cut slopes where the surface soils had slumped down slope (see Appendix A). The slumps are shallow, relatively small and extend up to 8 feet below the ground

surface and up through the "clay" portion of the slope. There is a significant amount of water at each slump.

The gravel slopes appear to be performing well except for some areas of surface erosion.

2. Neighboring Site

In preparation for this evaluation, AGEC reviewed a study conducted on the landslide within the Orchard Hill (Spring Hill) development (Terracon, November 30, 1998, Project No. 61985059). The Springhill landslide is located downslope and to the north of the Eaglepointe Development. Within this study, Terracon indicated the following:

- "The slide plane is primarily in the weaker zones of the tuffaceous bedrock unit and in weathered volcanic breccia along the northern flank of the slide. There are layers within the tuffaceous bedrock and weathered volcanic breccia that weathers to weak, highly plastic clay."

- "We believe the landsliding at Springhill Subdivision was ... triggered by the unusual heavy precipitation and attendant increases in groundwater levels."

- "Considerable land development is occurring on the hillside east of the property. Several subdivisions and the Eaglewood Golf course are all up gradient from Springhill Subdivision. Future increased contribution of water from these developments or other unknown up gradient sources could be a factor in landsliding unless drainage systems are effective in intercepting seepage before it enters the landslide area and/or drainage installation in the landslide are effective in permanently maintaining a lowered ground water level."

- "There is no apparent evidence of older, pre-existing landslides in this area."

The following engineering related information was obtained from the Terracon report:

- A sample of "Lean Tuffaceous Clay" was tested in the laboratory to determine strength characteristics. The sample was pre-sheared in order to obtain a "fully softened or residual" strength. The test results indicate approximately 350 pounds per square foot cohesion (c) with a friction angle (ϕ) of 15 degrees (The reported strength value is c = 0 psf and ϕ = 10°).

  Terracon also back calculated the soil strength from the landslide profile. The back calculation indicated a friction angle varying from 12 to 17 degrees with no cohesion. Their study was based on a "total stress" analysis (no excess pore pressure).

After our review of the Spring Hill slide, the following items are presented.

- The movement at Spring Hill does not appear to have occurred until after the sand and gravel was removed from the area of the development, (the area was a gravel pit).

  Removal of the sand and gravel resulted in overall flattening of the slope, which results in a more stable slope. The removal of the sand and gravel could also result in a reduction in slope stability by allowing the underlying materials (soil and bedrock) to swell. The swelling process could occur due to a decrease in overburden stress and also an increase in pore water pressure. The swelling can result in a reduction

in shear strength, in some cases reducing the shear strength to a residual value.

Downslope creep strains could also contribute to a reduction in shear strength.

Using the back calculated strengths ($\phi = 12°$ to $17°$, $c = o$) used by Terracon, the stability of the slope is calculated to be below or close to 1.0 when we evaluate the slope with an estimate of the original ground surface (prior to erosion and gravel mining, condition well before development). This indicates that the ground may have slid before the gravel was removed unless the current shear strengths are lower than they were in the past and/or if the pore pressure impact is greater today than in the past.

- The movement at Spring Hill seems to correspond with wet years. The higher amounts of water could result in an increase in pore water pressure. The increase in pore water pressure could result in a decrease in effective stress and therefore a reduction in the available shear strength supporting the slope.

- Springs in the area have been continuous, seasonal and in some areas artesian. The continuous springs are at elevations 4,530 to 4,550. The seasonal springs are at elevations 4,570 to 4,600. Artesian conditions were encountered at elevations 4,275 and 4,579. Terracon felt that some of the water that they encountered was perched or confined. These conditions were reported at elevations of 4,554 and 4,579.

Based on this information, we believe that the following combination of conditions resulted in the movement experienced.

- Shear strength reduction of the tuffaceous bedrock. We anticipate that the shear strength had been reduced to values somewhere between the fully softened and the reduced strengths.
- Reduction in the overburden stress due to removal of the overlying gravel during the gravel pit operation.
- An increase in pore water pressure.

Evidence of these conditions include the following:

- The tuffaceous bedrock has been observed to weaken.
- There were no apparent landslides until after the gravel had been mined from the area.
- The tuffaceous deposit includes relatively impervious and pervious materials. This type of deposit could provide the structure for a zone where the water could be pressurized (pore water pressure).
- The geology of the site appears to indicate that infiltration water from a large area upslope of the site appears to drain to the Spring Hill area.
- The toe of the slide includes the Warm Springs Fault. The bulk of the slide mass is upslope of the Warm Springs Fault. The fault could be restricting water flow out of the slope resulting in the possibility for increased pore pressure.

It is our professional opinion that the instability at Orchard Hills (Spring Hill) is a result of the items presented. We also believe that the area may have remained stable if one or more of the listed conditions did not occur.

B. Profiles

In order to evaluate the stability of the existing slopes at the site, the area was reviewed and three profiles were selected for analysis. Each of these slope profiles were measured in the field using a Global Positioning System. The locations of these profiles are shown on Figure 15.

Each profile is sloped at 2:1 (horizontal to vertical) with heights 150 and 175 feet.

The profiles were found to consist of the following:

Profile 1

Height: 150 feet

Materials: The slope consists of gravel separated by a 20 foot clay/silt layer. The clay/silt layer was encountered at approximately 80 vertical feet above the bottom of the slope.

Profile 2

Height: 175 feet

Materials: The slope consists of gravel separated by a 60 foot clay/silt layer. The clay/silt layer was encountered at 45 feet above the bottom of the slope.

Profile 3

Height: 150 feet

Materials: The upper 50 feet consists of gravel with tuffaceous bedrock below.

The water level used in the stability analysis was assumed, based on our observation, to be perched above low permeable materials or only in the upper few feet of soil. No water table was assumed within the gravel and sand slopes.

C. Materials

In order to evaluate the stability of the slopes, samples of the materials exposed in the cut slopes were obtained. Samples were obtained of the unweathered materials, of the material involved in current slope movements and materials that appear to have weathered. The approximate sampling locations are shown on Figure 15 and the profile in Appendix A and are designated as "HB"'s.

Samples of the materials exposed in the cut slopes were tested in the laboratory to determine strength parameters for the stability analysis. The samples were carved out of the hillside in blocks. The samples tested were carved from the blocks obtained in the field. Disturbed samples were obtained on the materials that had slumped. Disturbed samples were remolded to the density and moisture desired.

The testing included classification, direct shear and triaxial compression.

A table summarizing the index properties of the samples tested is shown below:

Index Properties on Strength Tested Samples

Sample I.D.	Moisture Content (%)	Dry Density (pcf)	Grain Size Distribution (percent)			Atterberg Limits		Sample Condition	Test Conducted
			+4 Sieve	4 to 200 Sieve	-200 Sieve	Liquid Limit (%)	Plasticity Index (%)		
HB-1"B"	14.5	97	0	27	73	31	14	Weathered	DS
HB-2"A"	26	94	0	22	78	31	8	Unweathered	DS
HB-2"B"	26	93	0	20	80	31	16	Weathered	DS
HB-3"A"	31	88	0	14	86	38	21	Unweathered	DS
HB-3"B"	27	94			81	38	23	Weathered	DS
HB-3"C"	33	87			80	43	28	Weathered	DS
HB-4			79	17	4			Disturbed	
HB-5"A"	3.9	121.6	23	11	66	38	20	Unweathered	DS
HB-5"B"	5.4	119.9	9	37	54	45	31	Unweathered	DS
HB-5"C"	30.3	85.3	0	26	74	68	53	Remolded	Triax
HB-5"D"	30.3	85.1	5	29	66	46	33	Remolded	Triax
HB-5"E"					85	25	9	Remolded	DS
HB-6"	5.1		69	11	20	54	42	Disturbed	
HB-7"B"	5.8		43	26	31	28	17	Disturbed	
TP-8 @ 10'	30	92	0	17	83	99	80	Remolded	DS

DS - Represents direct shear testing

Triax - Represents triaxial compression tests

A summary of the strength tests are presented on Figures 33 through 44. Listed below is a summary of the soil strengths obtained.

Material Strengths

Sample	Direct Shear Peak		Direct Shear Residual		Triaxial Compression				Description
	ϕ (°)	c (psf)	ϕ (°)	c (psf)	ϕ (°)	c (psf)	ϕ' (°)	c' (psf)	
HB-1 "B"	28	230	17	460					Lean Clay w/Sand
HB-2 "B"	28	640	25	170					Silty Clay w/Sand
HB-2 "B"	30	250	29	0					Lean Clay w/Sand
HB-3 "A"	34	520	27	225					Lean Clay
HB-3 "B"	23	1200	23	300					Lean Clay w/Sand
HB-3 "C"	28	0	25	0					Lean Clay w/Sand
HB-5 "A"	37	1300	41	50					Lean Clay w/Gravel
HB-5 "B"	33	1300	36	425					Sandy Lean Clay
HB-5 "C"					14	320	20	330	Fat Clay w/Sand
HB-5 "D"					15	60	23	141	Sandy Lean Clay
HB-5 "E"	20	300	20	150					Lean Clay w/Sand
TP-8 @10'	24	1950	26	0					Tuffaceous Bedrock

The strength of the gravel was assumed based on the condition of the gravel observed, the penetration resistance in Boring B-1 and based on the angle of repose measured of loose gravel in the laboratory ($\phi = 32°$). A realistic value of 36.9° has been used in our analysis for undisturbed sand and gravel. It is our professional opinion that the friction angle of the gravel is greater than this in most areas.

Based on the strength characteristics measured, the following soil parameters have been used for our analysis.

1. Unweathered - Intact Strengths

   a. Gravel: $\phi \geq 36.9°$ $c = 0.$

   b. Soil: $\phi = 28°$ $c = 600$ psf.

   The soil strength used is close to if not less than the weakest strength obtained of unweathered material and is from 64 to 82 percent of the average values.

2. Weathered - Intact and Fully Softened Strengths

   a. Gravel: $\phi \geq 36.9°$ $c = 0.$

   b. Soil: $\phi = 20°$ $c = 300$ psf.

   The selection of samples to test for weathered conditions was specifically conducted to obtain the weakest material we could find. With the similarity of our "Weathered - Intact" and "Fully Softened" strengths, we have used the same values for both.

   This soil strength is close to the weakest strength obtained of weathered material and is from 68 to 71 percent of the average values.

3. Residual

   a. Gravel: $\phi = 32°$ $c = 0.$

   b. Soil: $\phi = 15°$ $c = 200$ psf.

   This soil strength is generally less than the residual strength obtained of remolded samples and is 66 percent of the average values. It appears to be an average value of empirical correlations.

D. Water Conditions

One of our conclusions on the Orchard Hills stability includes the presence of water in the soil below the site and the likelihood of pore pressure build up. These two issues for the Orchard Hills and the Eaglepointe development are addressed below:

1. Hydrology

In order to evaluate the impact on subsurface water from the proposed Eaglepointe development, we requested the services of David Hansen, PhD of Hansen, Allen and Luce to evaluate the potential for increase or decrease of subsurface water with the development of Eaglepointe as proposed.

A critical issue in this evaluation includes the amount of water which would infiltrate the subsurface soils and migrate to an area where it would be detrimental to the stability of a slope.

We understand that the proposed development will only have culinary water that is metered. With this in mind, the amount of water usage is significantly reduced from that if the area were provided with irrigation water.

Dr. Hansen's evaluation indicates that the overall impact of the development will be a reduction in the amount of water that would infiltrate the subsurface. This benefit is derived by the collection and removal of surface water on roadways. Dr. Hansen's study is included in Appendix B.

2. Pore Pressure

We anticipate that the increase in pore pressure at the Orchard Hills development is largely a result of the Warm Springs Fault offsetting the pervious zones of subsurface materials. Terracon recommended installing subsurface drains to collect and remove as much water as possible to maintain stability of the slope. Providing drains would potentially mitigate pore pressure increase.

For the Eaglepointe development, we recommend that water seepage observed be collected and discharged without restriction in flow. We understand that this process has been followed throughout the reclamation of the gravel pit

area. We have seen evidence where water from springs has been collected and discharged with a drainage system. With these drainage systems in place and functioning along with no fault offset in the areas of concern, the likelihood of pore pressure buildup has been minimized.

3. Water Level for Analysis

For our stability analysis, we have assumed that there is not a groundwater condition in the Eaglepointe development. We have assumed that the water level is perched on top of relatively impervious layers and that it will flow to areas where these impervious layers meet the ground surface. We have assumed that these seeps will be collected and discharged to minimize the possibility of flow restriction. We have also evaluated the condition of very wet conditions near the ground surface during times of heavy rain and/or snowmelt.

E. Seismic Factor

In order to evaluate the potential for the hillside to move under a seismic event, the procedure recommended by Stewart, Blake and Hollingsworth, (2002) was followed.

In this procedure, the acceleration with a 10 percent probability of exceedance in 50 years is used in conjunction with 1) the magnitude of earthquake; 2) the distance between the site and the fault and 3) assumes a 50 percent likelihood of movement between 5 and 15 centimeters. The acceleration for the site with a 10 percent exceedance probability in 50 years is given as 0.285g (Frankel, et.al., 1996). Based on the referenced document, the acceleration is multiplied by 0.595 for displacements of 5 cm or less and 0.443 for displacements of up to 15 cm. The two resultant factors are 0.17g and 0.13g. Using an average of the two, a factor of 0.15g has been used for our analysis.

F. Analysis

Based on the variability of the subsurface materials, our analysis included evaluation of each of the three profiles along with simplified profiles of all gravel (friction only) and of soil like (friction and cohesion) materials. The stability analyses were conducted using infinite slope, sliding wedge and circular failure using the modified Bishop method.

The Utah Geologic Survey indicates that the factor of safety for slopes within a residential development should be at least 1.5 under static conditions and 1.0 under pseudo-static conditions.

1. Gravel Slopes

   A gravel slope was evaluated using infinite slope type analysis. The safety factor for a 2:1 slope without water in the slope is 1.5. Under a seismic event using a horizontal coefficient of 0.15g the factor of safety is 1.16. Both of these values indicated that dry, gravel slopes meet or exceed the requirements.

2. Soil Slopes

   Two to one slopes consisting entirely of "clay soil" up to 175 feet high were evaluated. Using the unweathered clay soil strengths, a safety factor of 1.5 is obtained without water in the slope. The same profile under a pseudo-static analysis is calculated to have a safety factor of 1.1. Both of these values comply with the recommendations of the Utah Geologic Survey.

   If all of the clay materials in the slope were to weaken to a "weathered" strength, the safety factor of the slope would drop to 1. Under a seismic event, the safety factor would be less than 1.

These values are consistent with the conditions observed on the site. Areas where the materials have remained unweathered and/or dry exhibit stable conditions. Areas where the materials have weathered and are wet have exhibited shallow slippages (extending to depths of 8 to 10 feet).

3. Bedrock Slopes

In areas where the bedrock is cemented, the stability meets the stability criteria if the slopes are at 2:1 due to the anticipation that the strengths would be greater than the "unweathered soil" strengths. There are areas of the site where cut slopes are steeper than 2:1, and the materials are cemented bedrock.

The stability analyses indicate that the 2:1 slopes meet stability criteria of the UGS when:

- The material strengths are at least as strong as the "unweathered soil" strengths indicated above.
- The water level remains only perched on relatively impervious layers.
- Pore pressures are not elevated.

We anticipate that the slopes will perform in the following fashion without any corrective action.

- Soil slumping will continue to occur in the wet soil areas.
- The gravel slopes will remain stable.
- The area of significant seepage will continue to erode and weather. Areas with clay or tuffaceous bedrock and seepage will weather and slump.

It is our professional opinion that the slopes will maintain their stability if the recommendations contained in this report are followed.

PROPOSED CONSTRUCTION

We understand that the proposed development is planned to consist of residential subdivisions containing one to three-story, wood frame structures with the possibility of full depth basements. We have assumed building loads consisting of maximum wall loads of 3 kips per lineal foot and maximum column loads of 20 kips based on typical residential construction in the area.

We understand that a school, church and other non-residential structures may be constructed at the site. Additional study should be performed for these facilities to provide specific design recommendations.

Roads are planned to be constructed throughout the proposed development. The following traffic conditions were used for the pavement analysis.

Traffic	Condition #1	Condition #2
Vehicles per day	1,000	2,000
Trucks per day	5	20

The traffic conditions indicated above are assumed to be one-way traffic with an average gross weight of 20 kips for trucks and buses.

If the proposed construction, assumed building loads or anticipated traffic conditions are significantly different from those described above, we should be notified to re-evaluate our recommendations.

RECOMMENDATIONS

Based on the subsurface conditions encountered, laboratory test results, our understanding of the proposed construction and our experience in the area, the following recommendations are given:

A. Site Grading

1. Excavation

We anticipate that excavation in the soil and upper portion of the tuffaceous bedrock can be accomplished with typical excavation equipment. Excavation equipment refusal was encountered in some of the test pits at depths ranging from approximately 4½ to 13 feet below the adjacent ground surface in bedrock. Heavy-duty excavation equipment will be needed for excavation into the conglomerate and deep and cemented layers in the tuffaceous bedrock. Difficult excavation may be encountered in confined excavations which extend into the dense gravel, bedrock or encounter boulders.

2. Existing Fill

Up to approximately 9 feet of fill overlying natural soil was encountered in Test Pits TP-4, 8, 11, 12, 15, 17, 19, 26 and 35. Fill extends the full depth investigated, approximately 15 and 6 feet below the adjacent ground surface, in Test Pits TP-13 and TP-26, respectively. The fill encountered in test pits consists primarily of sand with small to moderate amounts of silt and clay and occasional gravel. The fill encountered generally is not well compacted.

There are likely other areas of fill on the site. It will be important for individual builders to evaluate the fill on a lot by lot basis to determine if there is fill in the area of proposed construction and if it is suitable to support the construction.

3. Pavement Subgrade Preparation

Prior to placing site grading fill or base course; organic material, topsoil, unsuitable fill and other deleterious material should be removed. The subgrade should be scarified to a depth of approximately 10 inches, the moisture adjusted to within 2 percent of the optimum and compacted to at least 90 percent of the maximum dry density as determined by ASTM D-1557. The subgrade should then be proof-rolled to identify soft areas. Soft areas should be removed and replaced with properly compacted granular fill.

Clay and silt encountered at proposed subgrade will result in construction difficulties for rubber tired construction equipment when the soil is very moist to wet. The subgrade should not be scarified and proof-rolled in areas of very moist to wet clay, but cut to undisturbed natural soil below the topsoil and a sufficient thickness of granular fill placed to provide support fo construction equipment. Generally, 1 to 2 feet of granular fill containing less than 15 percent passing the No. 200 sieve will provide limited support for moderately loaded rubber-tired construction equipment above very moist to wet clay and silt soil. Care will be required to minimize disturbance of the natural soil during construction.

Subgrade areas which are disturbed or rutted should be removed and replaced with properly compacted granular fill.

4. Materials

Listed below are materials recommended for structural fill.

Fill to Support	Recommendations
Footings	Non-expansive granular soil Passing No. 200 Sieve < 35% Liquid Limit < 30% Maximum size 4 inches
Floor Slab (Upper 4 inches)	Sand and/or Gravel Passing No. 200 Sieve < 5% Maximum size 2 inches
Slab Support	Non-expansive granular soil Passing No. 200 Sieve < 50% Liquid Limit < 30% Maximum size 6 inches

Materials placed as fill to support foundations should be non-expansive granular soil. The on-site sand and gravel exclusive of topsoil, organics, debris and over-size particles meeting the criteria above is suitable for use as structural fill at the site. The tuffaceous bedrock, natural clay and silt and some of the existing fill at the site are not suitable for use as fill below structures but may be used as site grading fill and utility trench backfill, if the expansive soil and bedrock, organics, debris and other deleterious materials are removed, or they may be used in landscaping areas.

The existing fill, bedrock and natural soil may require moisture conditioning (wetting or drying) prior to use as fill. Drying of these materials may not be practical during cold or wet times of the year.

5. Compaction

Compaction of materials placed at the site should equal or exceed the minimum densities as indicated below when compared to the maximum dry density as determined by ASTM D-1557.

Fill To Support	Compaction
Foundations	≥ 95%
Concrete Slabs and Pavement	≥ 90%
Landscaping	≥ 85%
Retaining Wall Backfill	85 - 90%

To facilitate the compaction process, fill should be compacted at a moisture content within 2 percent of the optimum moisture content.

Base course should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D-1557.

Fill and pavement materials placed for the project should be frequently tested for compaction.

6. Drainage

The ground surface surrounding the proposed residences should be sloped away from the residences in all directions. Roof down spouts and drains should discharge beyond the limits of backfill.

The collection and diversion of drainage away from the pavement surface is important to the satisfactory performance of the pavement section. Proper drainage should be provided.

B. Slope Stability

The condition and stability of existing and proposed slopes in the Eaglepointe Development are dependent upon the height, slope angle, water conditions and material characteristics.

Based on the stability analysis, the following three options are given.

Option #1

The clay slopes could be flattened in order to minimize the potential for slope instability. If we are to assume that the soil weathering could extend approximately 5 feet into the slope, the slope would need to be flattened to approximately 2.5:1 (H:V) for a safety factor of 1.3. If the entire slope were to weather, the slope would need to be flattened to a 2.8:1 (H:V) for a safety factor of 1.3 and 3.3:1 (H:V) for a safety factor of 1.5. The angle for the slope varies depending on the height and thickness of the "clay" in the slope.

Option #2

Consideration could also be given to flattening just the clay portions of the slope. The sand and gravel portion of the slopes could still be sloped at 2:1. The clay portion of the slope would need to be flattened to between 2.5 to 3.3 (horizontal to vertical). The angle for the clay portion of the slope depends on the thickness of clay.

Option #3

The clay on the slope should be protected from accelerated weathering. This can be accomplished by covering the slope with topsoil and vegetating the slopes. With this option, new construction should be set back from the toe and top of the slope as indicated in this report.

General conditions to maintain stability of the slopes include the following:

1. The slopes need to be protected from water. Seeps on the slope should be controlled by collecting the water from behind the surface of the slope and discharging the water away from the slope. The seepage condition should be evaluated on an individual basis and the method used to collect and discharge the water should be determined. We anticipate that the specific control would include, but will not be limited to the following:

   a. Excavate into the hillside in order to establish a location where gravel and a pipe could be placed to collect the water. This could also be accomplished by installing horizontal drains into the hillside and installing pipes for collection and discharge of the water.

   b. There may be a possibility of drilling vertical holes through thin layers of the fine grained materials down to gravel below. Access may be a challenge for this approach. The drilled holes would be filled with gravel to provide a vertical drain for seepage water to flow down to the lower gravel.

   c. Areas of previous slumps should be modified by removing the disturbed soil and disturbed bedrock. The removal should include the soils that have been pushed back into place. The excavation should extend down to undisturbed material. Once the disturbed soil and disturbed bedrock have been removed, a drain, consisting of gravel and a pipe, should be installed to collect the water as far back into the slope as possible.

      The area of excavation should be replaced with granular, nonplastic soil that can be placed and properly compacted to the requirements contained in this report.

2. It is important to provide means for access to the hillside during the lifetime of the development. There may be areas in the future which may erode and/or slough and will need to be repaired.

3. The Uniform and International Building Codes indicate that homes should be set back from the top of the slopes steeper than 3:1 a distance equal to ⅓ the height of the slope but need not exceed 40 feet. Homes should be set back from the toe of slopes steeper than 3:1 slopes a distance equal to ½ the height of the slope but need not exceed 15 feet. We recommend that this criteria be met for slopes of granular soil and bedrock.

We recommend that the top set back for slopes with 2:1 slopes with clay be established by extending an imaginary plane from the toe of the clay up to the ground surface. The intersection of the plane with the footings will provide the appropriate setback. The slope of the imaginary plane is as indicated below. The value listed is the "B" of "B":1 slope.

Top Slope Setback Criteria

Slope Height (ft)	Percent Clay (in slope, clay and gravel above)					
	0	20	40	60	80	100
35	*2.35	*2.35	*2.35	*2.35	*2.35	*2.35
50	*2.35	*2.35	2.5	2.65	2.8	2.9
75	*2.35	2.5	2.8	3.0	3.15	3.25
100	*2.35	2.6	2.95	3.2	3.35	3.45
125	*2.3	2.65	3.05	3.3	3.45	3.55
150	*2.3	2.7	3.1	3.35	3.5	3.6
175	*2.2	2.75	3.15	3.4	3.55	3.65

Note - Slope height for this criteria is from the toe of the clay up to the top of the 2:1 slope.

- Slope is from the toe of the clay.

* UBC/IBC criteria.

The actual set-back should be established once the final grading plans are available with accurate contours.

We recommend that the toe set back from 2:1 slopes with clay exceed the UBC/IBC criteria to allow for maintenance access. The set back should also include an area for accumulation of slumped material. The set back would depend on the amount of clay exposed on the 2:1 slope. Listed below are our recommendations for building setbacks.

Bottom Slope Setback Criteria

Slope Height (ft)	Percent Clay					
	0	20	40	60	80	100
35	*	*	*	*	*	*
50	*	*	*	*	*	*
75	*	*	*	*	*	
100	*	*	*	*	*	35
125	*	*	*	*	25	50
150	*	*	*	20	40	60
175	*	*	*	30	55	75

* UBC/IBC Criteria

The actual setback can be refined once the final grading plans are available.

These criteria will change if the overall slope is modified.

4. All fill placed should be compacted to the compaction indicated in the Site Grading section of this report.

5. The impact on stability of any proposed slope modifications should be evaluated and should meet the minimum stability requirements before proceeding. This is particularly important for fill placed at the top of slopes and cuts at the toe of slopes.

6. Consideration should be given to providing a means to collect surface runoff and any subsurface seepage that accumulates at the bottom of the slopes. This collection and disposal of the water would help reduce the very moist and relatively unstable conditions currently being experienced at the toe of many of the slopes.

C. Foundations

1. Foundation Support

a. All Areas

With the proposed construction and the subsurface conditions encountered, proposed residences may be supported on spread footings bearing on the undisturbed natural soil, bedrock or on compacted structural fill extending down to the undisturbed natural soil or bedrock. For foundations bearing on bedrock and partially on soil, at least 2 feet of structural fill should be provided between the foundation and bedrock. Structural fill, where required, should extend out away from the edge of the footings a distance at least equal to the depth of fill beneath footings.

Topsoil, organics, unsuitable fill, debris and other deleterious material should be removed from below foundation areas.

b. Moisture Sensitive Areas

Based on the subsurface conditions encountered at the site and the results of laboratory tests conducted, structures supported with spread footings bearing on the low to moderately expansive soil or bedrock may experience up to 1 inch of heave (uplift) if the bearing material becomes wet.

To reduce the potential for significant differential movement of foundations above areas of potentially moisture sensitive soil and bedrock, consideration may be given to removal and replacement of some of the moisture sensitive material or extending the foundation support below the moisture sensitive material with deep foundation systems.

2. Bearing Pressure

Spread footings bearing on the undisturbed natural soil may be designed using an allowable net bearing pressure of 1,500 psf. Footings bearing on at least 2 feet of properly compacted structural fill or on at least 2 feet of the natural gravel may be designed using an allowable net bearing pressure of 3,000 psf. Footings bearing on the undisturbed bedrock may be designed using an allowable net bearing pressure of 3,500 psf.

Higher bearing pressures may be possible depending on the type and thickness of structural fill, the underlying bearing material.

Footings should have a minimum width of 1½ feet and a minimum depth of embedment of 10 inches.

3. Temporary Loading Conditions

The allowable bearing pressure may be increased by one-half for temporary loading conditions such as wind or seismic loads.

4. Settlement

We estimate that total and differential settlement will be less than 1 inch and ¾ inch, respectively, for footings bearing on the undisturbed natural soil. Total and differential settlement is estimated to be less than ½ inch for footings bearing on at least 2 feet of structural fill, undisturbed natural gravel or bedrock.

We estimate that total and differential vertical movement (settlement or heave) due to the load of the structure and subsurface wetting will be less than 1 inch for footings designed as indicated above.

Disturbance of the soil or bedrock below foundations can result in greater differential vertical movement. Care should be taken not to disturb the bearing materials so that settlement can be maintained within tolerable limits.

5. Drainage/Moisture Control

It is important to maintain little or no moisture change in the moisture sensitive soil below and surrounding proposed foundation areas during the life of the homes. Care should be taken to prevent significant changes in moisture during excavation and construction of the foundation. Foundation backfill should consist of low permeable material and should be placed and compacted with a moisture content within 2 percent of the optimum moisture content. The exterior grade should promote drainage of runoff water from the areas surrounding the structure. Roof down spouts and drains should discharge well beyond the limits of backfill. Irrigation in areas surrounding the structure should be minimized.

6. Frost Depth

Exterior footings and footings beneath unheated areas should be placed at least 30 inches below grade for frost protection.

7. Foundation Base

The base of all excavations should be cleared of loose or deleterious material prior to structural fill or concrete placement.

8. Construction Observation

A representative of the geotechnical engineer should observe footing excavations prior to structural fill or concrete placement. This is particularly important for structures constructed in areas of potentially moisture sensitive soil and/or bedrock (see Figure 49).

D. Concrete Slab-on-Grade

1. Slab Support

Concrete slabs may be supported on the undisturbed natural soil, bedrock or on compacted structural fill extending down to the undisturbed natural soil or bedrock.

Topsoil, organics, unsuitable fill and other deleterious materials should be removed from below proposed floor slab areas.

2. Underslab Sand and/or Gravel

A 4-inch layer of free draining sand and/or gravel (less than 5 percent passing the No. 200 sieve) should be placed below the concrete slabs for ease of construction and to promote even curing of the slab concrete.

3. Moisture Sensitivity

Some of the clay and bedrock encountered at the site are sensitive to changes in moisture and may expand when wetted. Lightly loaded portions of a structure (like floor slabs and exterior concrete flatwork) will be most susceptible to movement if the support clay and/or bedrock expands.

Up to approximately 2 inches of heave (uplift) may be experienced for floor slabs and concrete flatwork if the moisture sensitive soil becomes wet.

Ideally, all moisture sensitive soil and/or bedrock would be removed from below proposed building areas. Alternatively, removing a portion of the moisture sensitive soil or bedrock will reduce the potential for floor slab distress if the underlying moisture sensitive soil or bedrock becomes wet. If the moisture sensitive material is left in place, it is important to maintain little or no moisture change in the moisture sensitive soils below and surrounding proposed foundation areas during the life of the building. Care should be taken to prevent significant changes in moisture during excavation and construction of the foundation. Foundation backfill should consist of low permeable material and should be placed and compacted at a moisture content slightly above the optimum moisture content. The exterior grade should promote drainage of runoff water from the areas surrounding the structure. Roof down spouts and drains should discharge well beyond the limits of backfill. Irrigation in areas surrounding the structure should be minimized.

E. Lateral Earth Pressures

1. Lateral Resistance for Footings

Lateral resistance for spread footings placed on the natural soil or on compacted structural fill is controlled by sliding resistance between the footing and the foundation soils. Friction values of 0.3 and 0.45 may be used in

design for ultimate lateral resistance for cohesive and granular soil, respectively.

2. Subgrade Walls and Retaining Structures

The following equivalent fluid weights are given for the design of subgrade walls and retaining structures. The active condition is where the wall moves away from the soil. The passive condition is where the wall moves into the soil and the at-rest condition is where the wall does not move. The values listed below assume a horizontal surface adjacent the top and bottom of the wall.

Soil Type	Active	At-Rest	Passive
Clay & Silt	50 pcf	65 pcf	250 pcf
Sand & Gravel	40 pcf	55 pcf	300 pcf

3. Seismic Conditions

Under seismic conditions, the equivalent fluid weight should be increased by 42 pcf for active and at-rest conditions and decreased by 42 pcf for the passive condition. This assumes a short period spectral response acceleration of 1.797g which represents a 10 percent probability of exceedance in a 50 year period (IBC, 2000).

4. Safety Factors

The values recommended above for active and passive conditions assume mobilization of the soil to achieve the soil strength for the indicated lateral load. Conventional safety factors used for structural analysis for such items as overturning and sliding resistance should be used in design.

F. Seismicity and Liquefaction

1. Seismicity

Listed below is a summary of the site parameters for the International Building Code 2000.

a.	Site Class	D
b.	Short Period Spectral Response Acceleration, S_S	1.797g
c.	One Second Period Spectral Response Acceleration, S_1	0.761g

2. Liquefaction

The Davis County liquefaction hazard map (Anderson and others, 1994) indicates that the site is located in an area mapped as having a "very low" liquefaction potential. Research indicates that the soil type most susceptible to liquefaction during a large magnitude earthquake is loose, clean sand. In order for liquefaction to occur, the soil must be saturated. The liquefaction potential for soil tends to decrease with an increase in fines content and density.

Based on the subsurface conditions encountered to the depth investigated and our understanding of geologic conditions in the area, it is our professional opinion that liquefaction is not a concern for the proposed development.

G. Subsurface Drains

As described in the geology section of the report, many active springs and seeps were observed. Subsurface water is anticipated in excavations that extend below grade in areas where seeps have been seen in the past.

Building floors which extend below grade adjacent to springs and seeps and where subsurface water is encountered above or within 3 feet of the floor level should be protected with a perimeter underdrain. Basement excavations which extend down into conditions where water may perch above a low permeable layer of soil or bedrock should be evaluated for a subdrain on an individual basis. The perimeter underdrain should consist of at least the following items.

1. The underdrain should consist of a perforated pipe installed in a free-draining gravel filled trench around the perimeter of the below grade floor portion of the building.

2. The flow line of the pipe should be placed at least 18 inches below the finished floor level and should slope to a sump or outlet where water can be removed by pumping or by gravity flow.

3. If placing the gravel and drain pipe requires excavation below the bearing level of the footing, the excavation for the drain pipe and gravel should have a slope no steeper than 1:1 (horizontal to vertical) away from the edge of the footing to avoid disturbing the soil below the footing.

4. A filter fabric should be used to help reduce the migration of fine-grained materials.

5. The subgrade floor slab should have at least 6 inches of free draining gravel placed below it and the under slab gravel should connect to the perimeter drain.

6. Consideration should be given to installing cleanouts to allow access into the perimeter drain, should cleaning of the pipe be required in the future.

Consideration may be given to collecting and controlling the flow of seepage water that could be a nuisance to the proposed development. The method of collection and discharge should be evaluated on an individual basis.

H. Water Soluble Sulfates

One sample of the clayey fill and three samples of the natural soil were tested in the laboratory for water soluble sulfate content. Test results indicate less than 0.1 percent water soluble sulfates in the samples tested. Based on the test results, our experience in the area and published literature, the natural soil possesses negligible sulfate attack potential on concrete. No special cement type is required for concrete placed in contact with the natural soil. Other conditions may dictate the type of cement to be used in concrete for the project.

I. Pavement

Based on the subsoil conditions encountered, laboratory test results and the assumed traffic conditions presented in the Proposed Construction section of the report the following pavement support recommendations are given.

1. Subgrade Support

   The near surface soil consists of clay, silt and sand. California Bearing Ratio (CBR) values of 3 and 15 percent were used in the analysis which assumes a clay and granular subgrade, respectively.

2. Pavement Thickness

Based on the subsoil conditions, assumed traffic, a design life of 20 years for flexible pavement and 30 years for rigid pavement and methods presented by the Utah Department of Transportation, the following pavement sections are calculated.

PAVEMENT THICKNESSES

		Rigid Pavement	Flexible Pavement		
Traffic Condition	Subgrade	Portland Cement Concrete Thickness	Asphaltic Concrete Thickness	Base Course Thickness	Granular Borrow Thickness
#1	Clay	5"	—	—	—
		—	3"	8"	—
	Granular	5"	—	—	—
		—	3"	6"	—
#2	Clay	5"	—	—	—
		—	3"	14"	—
		—	3"	6"	9"
	Granular	5"	—	—	—
		—	3"	6"	—

Note: Traffic conditions are described in the Proposed Construction section of the report. Other pavement sections could be provided using different combinations of pavement materials and traffic loads.

If construction occurs when the subgrade is very moist to wet and consists of clay, silt or claystone, it will likely be necessary to place granular fill below the construction traffic areas to facilitate construction of the pavement section. This is discussed in the Pavement Subgrade Preparation section of the report.

The base course thickness may be reduced to 6 inches in areas where significantly less traffic is anticipated such as cul-de-sacs.

3. Pavement Materials and Construction

a. Flexible Pavement (Asphaltic Concrete)

The pavement materials should meet the Utah Department of Transportation specifications for gradation and quality. Other materials may be considered for use in the pavement section. The use of other materials may result in the need for different pavement material thicknesses.

b. Rigid Pavement (Portland Cement Concrete)

The rigid pavement thickness assumes that the pavement will have aggregate interlock joints and that a concrete shoulder or curb will be provided.

Pavement materials should meet the Utah Department of Transportation Specifications. The pavement thickness indicated above assumes that the concrete will have a 28 day compressive strength of 4,000 pounds per square inch. Concrete should be air entrained with approximately 6 percent air. Maximum allowable slump will depend on the method of placement but should not exceed 4 inches.

Joints for concrete pavement should be laid out in a square or rectangular pattern. Joint spacings should not exceed 30 times the thickness of the slab. The joint spacings indicate should accommodate the contraction of the concrete and under these conditions steel reinforcing will not be required. The depth of joints should be approximately one-quarter of the slab thickness.

LIMITATIONS

The analysis and report findings are based on review of geologic literature, review of aerial photographs, site reconnaissance and subsurface investigation. Geologic related conclusions are based on currently accepted geologic interpretation of this information. Geologic maps and diagrams presented in this report depict our interpretation of the subsurface geologic conditions based on available information and surface exposures on and near the site. No attempt has been made to predict earthquake ground motions or determine potential magnitude for earthquakes associated with faults in the project area, to estimate recurrence intervals or to address the potential for surface fault rupture along faults discussed in this report. The conditions presented herein should be re-evaluated if construction activity indicates significantly differing conditions or if land use or lot placement plans for the proposed development are modified or extended outside the explored area.

This report has been prepared in accordance with generally accepted soil and foundation engineering practices in the area for the use of the client for design purposes. The geotechnical conclusions and recommendations included in the report are based on the information obtained from the test pits excavated, boring drilled, the data obtained from laboratory testing and our experience in the area. Variations in the subsurface conditions may not become evident until additional exploration or excavation is conducted. If the subsurface conditions or groundwater level is found to be significantly different from what is described above, we should be notified to re-evaluate our recommendations.

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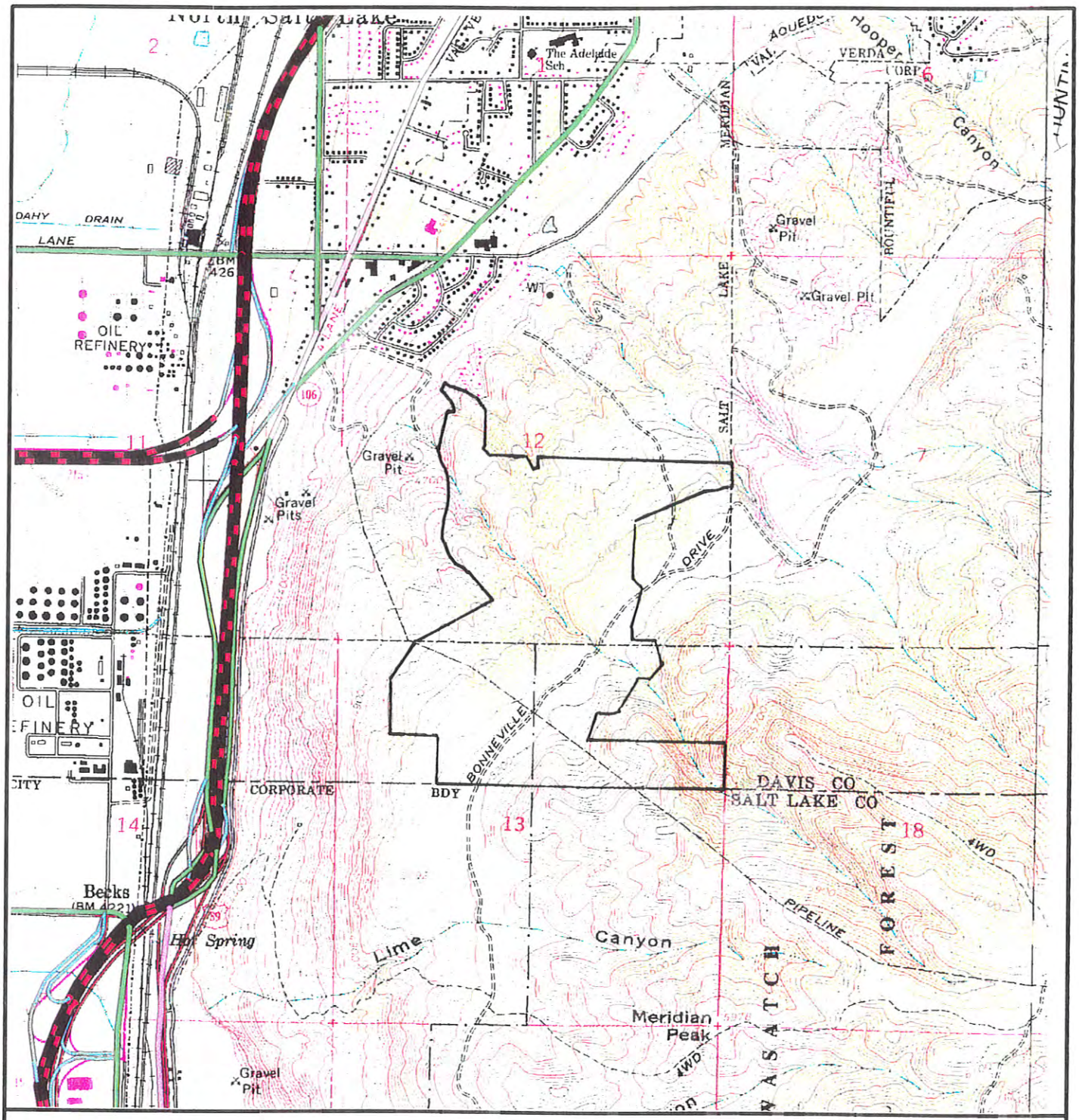


Section 12 & 13, T1N, R1W

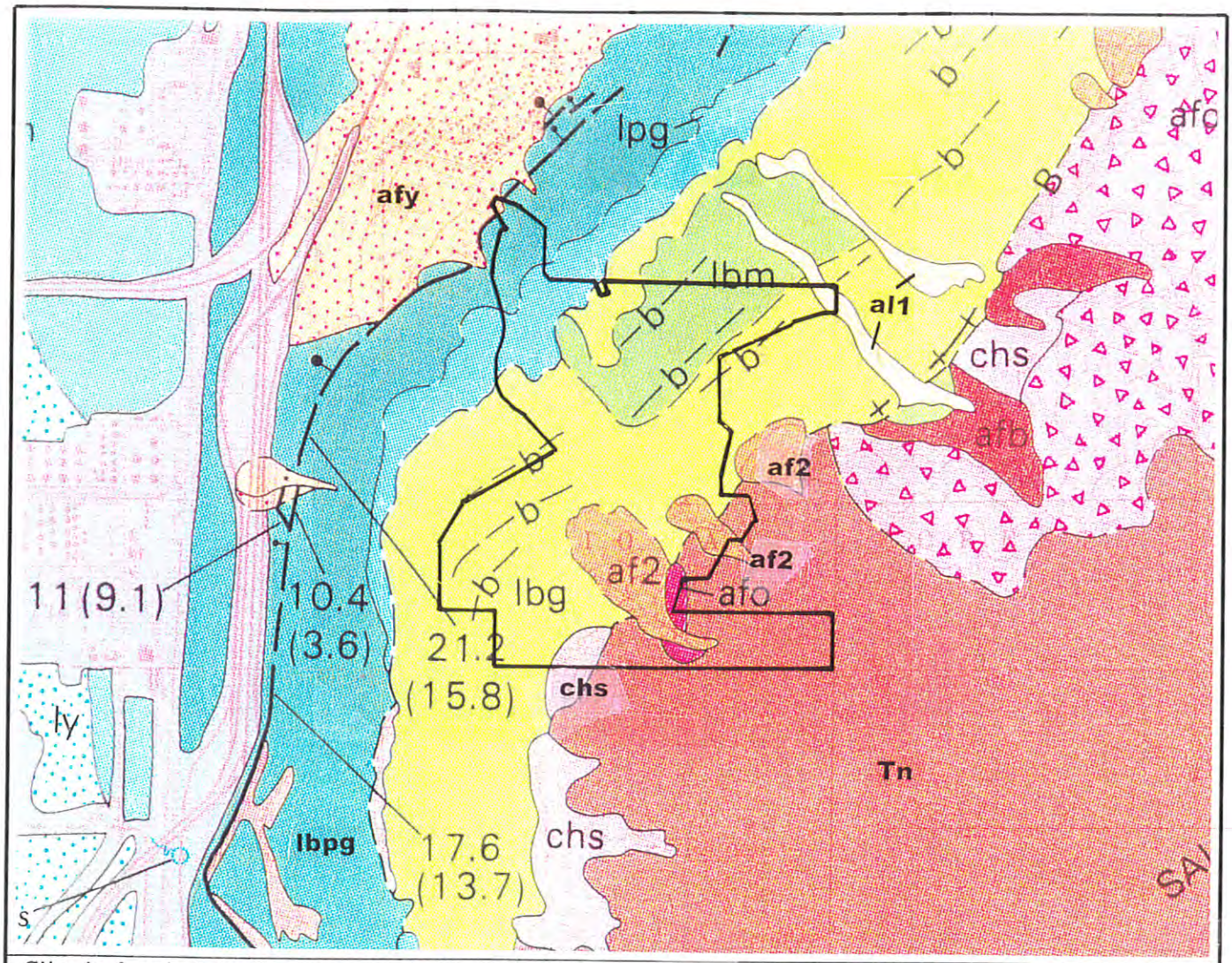
From USGS Salt Lake City North Quadrangle (1975) and GDT Dynamap (January 2000)

N

Approximate Scale
1 inch = 2,000 feet

PROPOSED EAGLEPOINTE DEVELOPMENT
NORTH SALT LAKE, UTAH

1020320	AGEC	Approximate Site Location	Figure 1



Site is located in Sections 12 & 13, T1N, R1W

modified from Personius & Scott (1992)

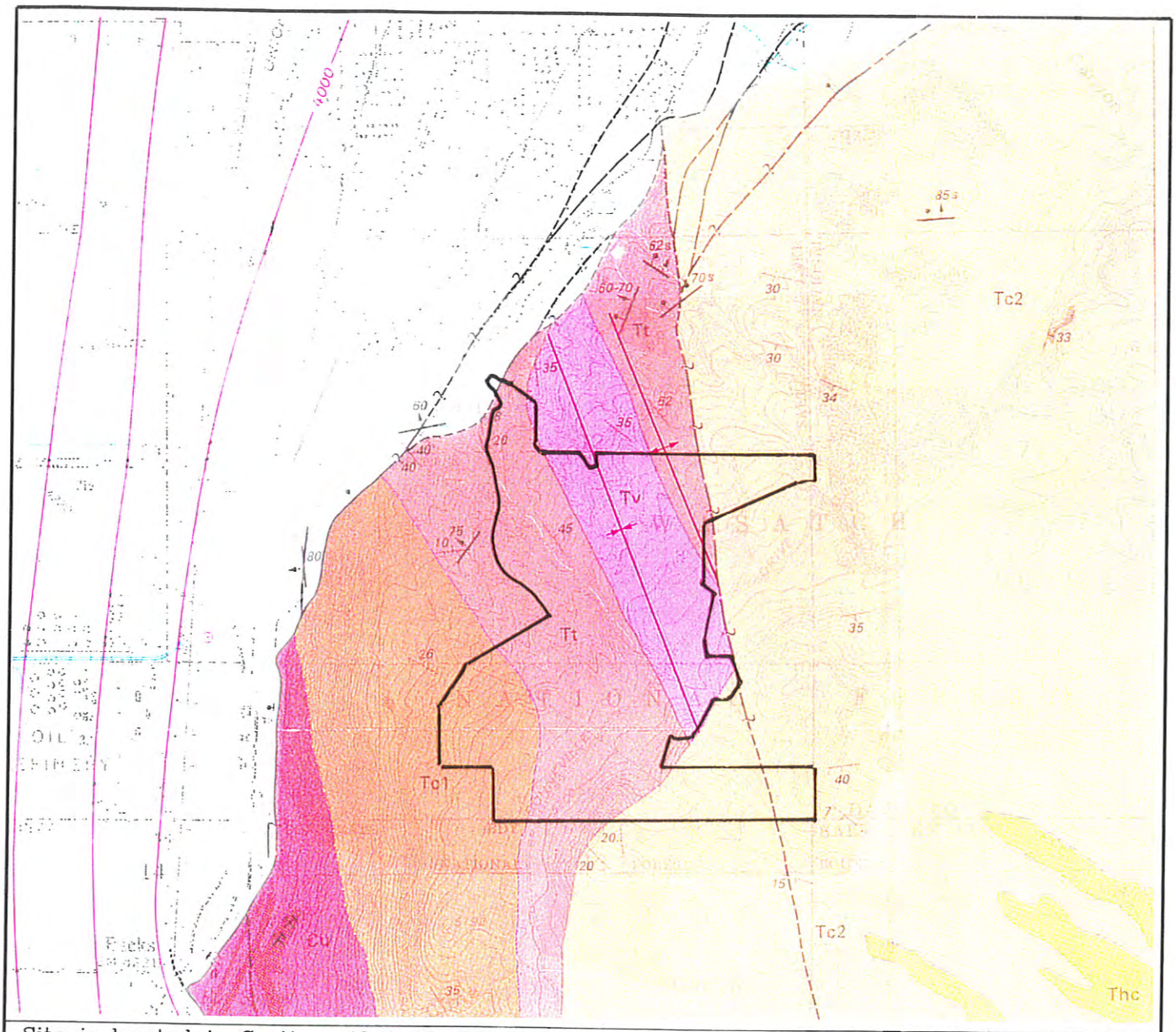
N

Approximate Scale
1" = 2,000'

PROPOSED EAGLEPOINTE DEVELOPMENT
NORTH SALT LAKE, UTAH

al1	Stream alluvium (upper Holocene)
afy	Younger fan alluvium, undivided (Holocene to uppermost Pleistocene)
af2	Fan alluvium 2 (middle Holocene to uppermost Pleistocene)
chs	Hillslope Colluvium (Holocene to upper Pleistocene)
lpg	Lacustrine sand and gravel related to regressive phase (uppermost Pleistocene)
lbpg	Lacustrine sand and gravel, undivided (upper Pleistocene)
lbm	Lacustrine clay and silt related to transgressive phase (upper Pleistocene)
lbg	Lacustrine sand and gravel related to transgressive phase (upper Pleistocene)
afo	Older fan alluvium, undivided (middle Pleistocene)
Tn	Tertiary sedimentary and volcanic rocks (Neogene)
———	Contact – dashed where approximate
—•— — —	Normal fault – bar and ball on downdropped side, dashed where approximate
— B —	Bonneville Shoreline
— b —	Other shorelines of transgressive phase

1020320	AGEC	Surficial Geology of Project Area	Figure 2



Site is located in Sections 12 & 13, T1N, R1W

modified from Van Horn (1981)

PROPOSED EAGLEPOINTE DEVELOPMENT
NORTH SALT LAKE, UTAH

Tc2	Conglomerate No. 2 (Tertiary)
Tv	Volcanic Breccia (Tertiary)
Tt	Tuffaceous Deposits (Tertiary)
Tc1	Conglomerate No. 1 (Tertiary)

N

Approximate Scale
1" = 2,000'

Contact – long dashed where approximately located, short dashed where inferred, queried where probable

Fault – arrow indicates dip, long dashed where approximately located, short dashed where inferred, queried where probable. Bar and ball on downthrown side

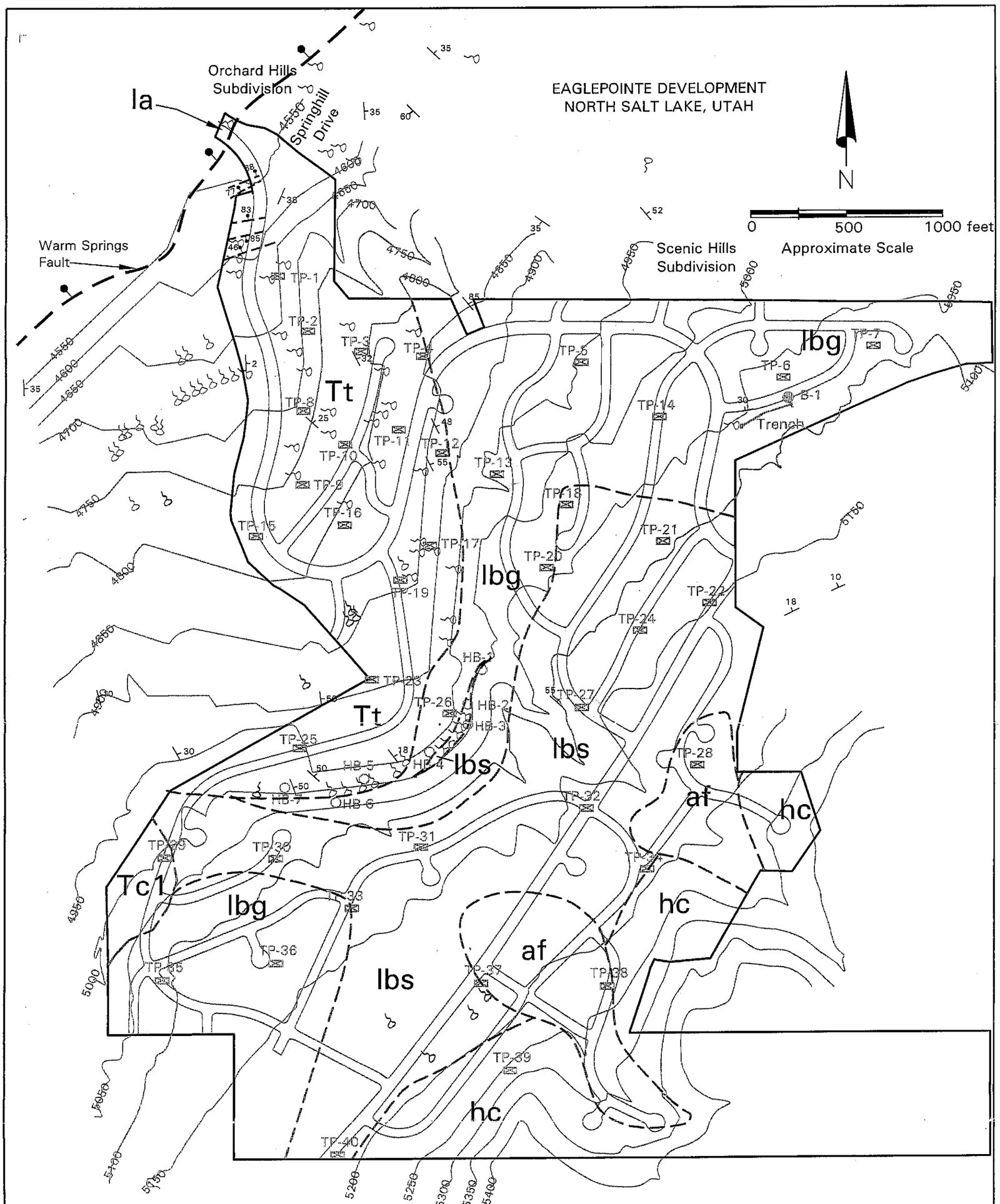
Anticline

Syncline

30 Strike and dip of beds.

30 Strike and dip of fault.

1020320	AGEC	Bedrock Geology of Project Area	Figure 3

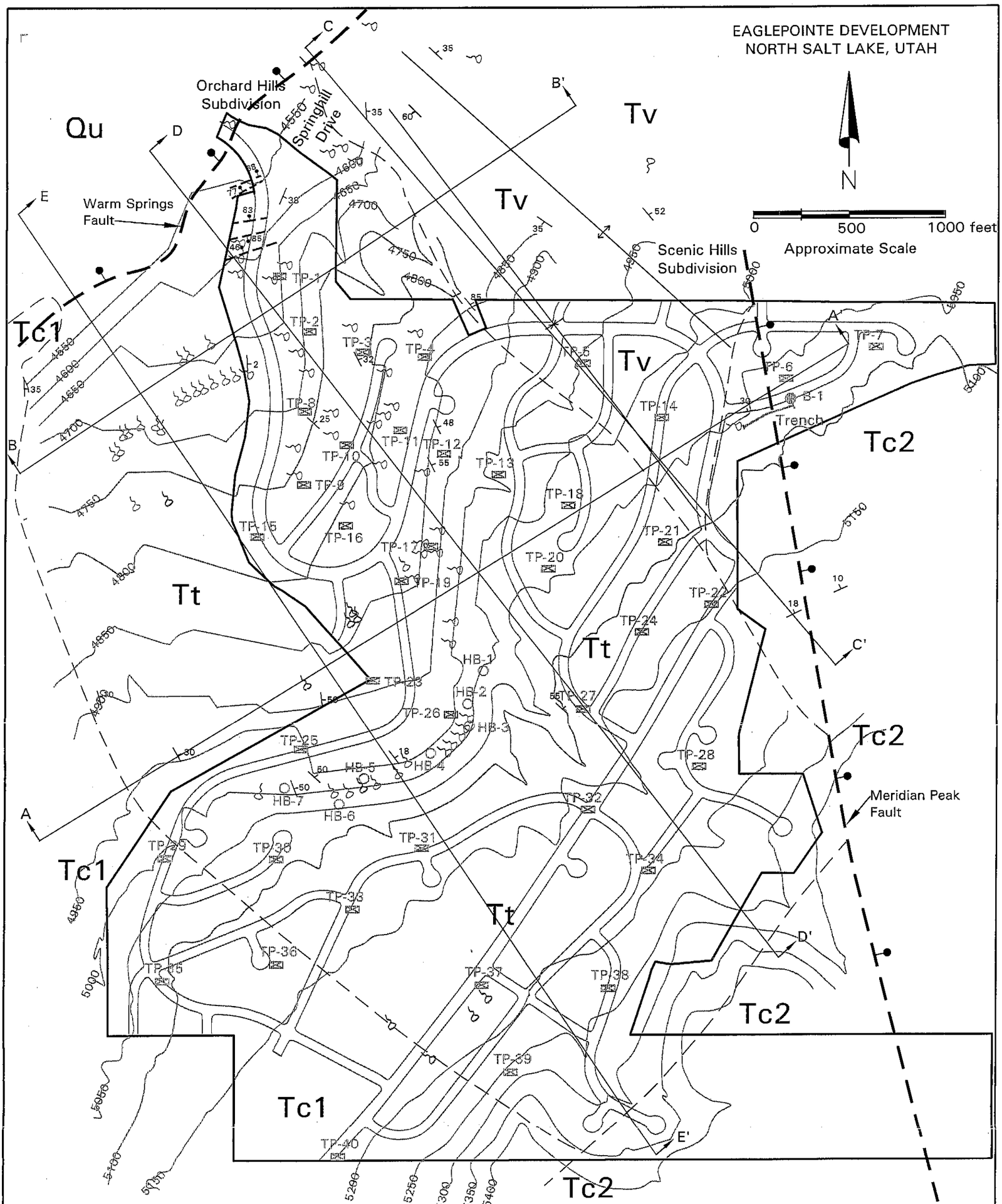


Legend:

- Trench
- Test Pit Location
- Boring Location
- Hand Excavation Location
- Seeps

af	-	Older alluvial fan (middle Holocene to uppermost Pleistocene)
hc	-	Hillslope Colluvium (Holocene to upper Pleistocene)
lbs/lbg	-	Lake Bonneville transgressive phase deposits (uppermost Pleistocene)
la	-	Undifferentiated lacustrine and alluvial deposits (Pre-Lake Bonneville)
Tt	-	Tuffaceous deposits (Tertiary)
Tc1	-	Conglomerate No. 1 (Tertiary)
---		Contact, approximately located
		Fault, approximately located, bar and ball on down thrown side Number next to fault indictes measured dip of fault
10		Strike and dip of beds

1020320	AGEC	Surficial Geologic Map	Figure 4



Legend:

- Trench
- Test Pit Location
- Boring Location
- Hand Excavation Location
- Seeps

Qu	-	Quaternary undifferentiated sediments
Tc2	-	Conglomerate No. 2 (Tertiary)
Tv	-	Volcanic breccia (Tertiary)
Tt	-	Tuffaceous deposits (Tertiary)
Tc1	-	Conglomerate No. 1 (Tertiary)

Contact, approximately located

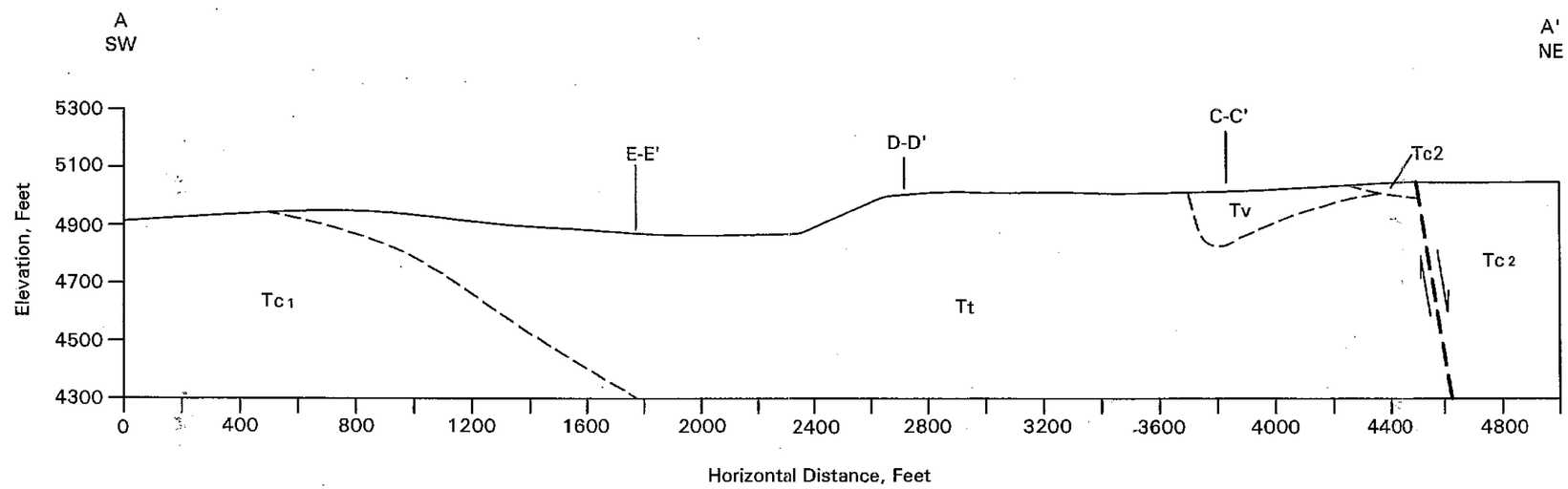
Fault, approximately located, bar and ball on down thrown side
Number next to fault indicates measured dip of fault

Syncline

Anticline

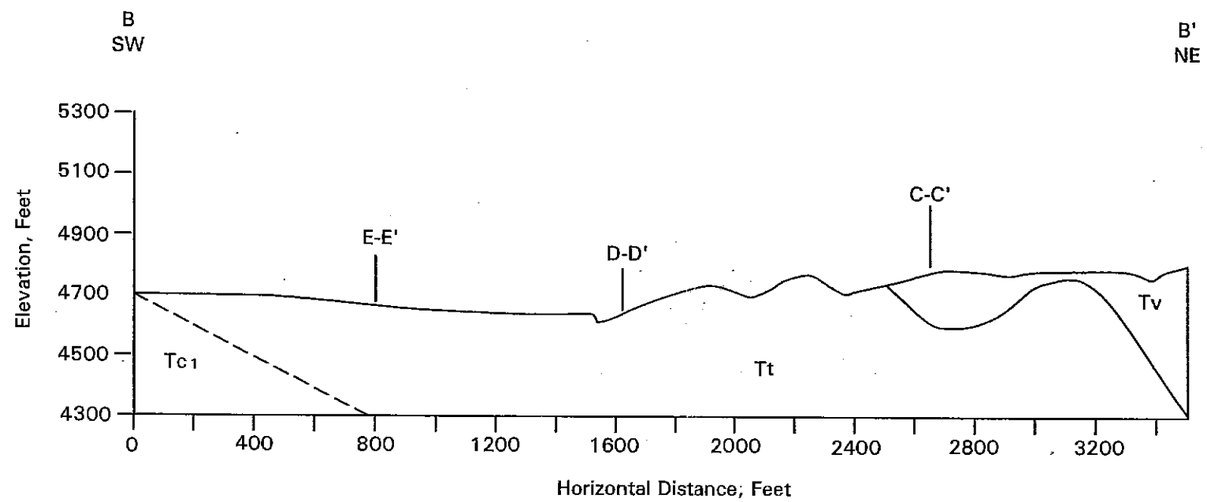
10 Strike and dip of beds

1020320	AGEC	Bedrock Geologic Map	Figure 5



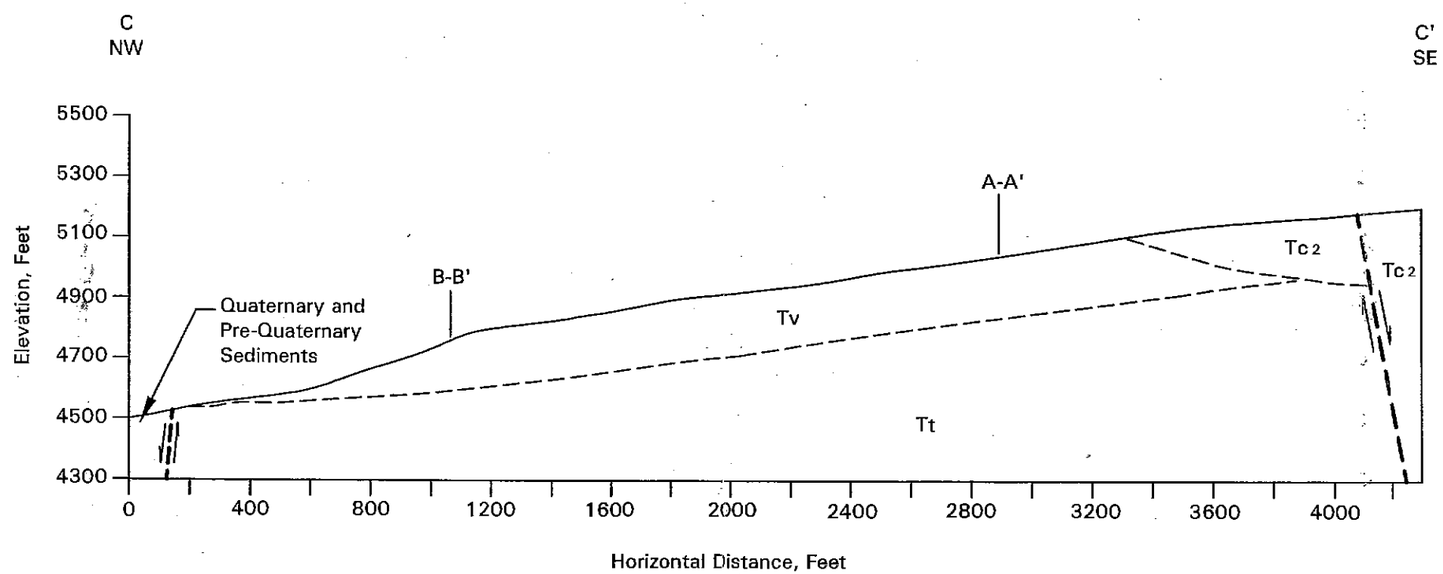
See Figure 5 for Location of Profile and Explanation of Symbols

1020320	AGEC	Cross Section A-A'	Figure 6



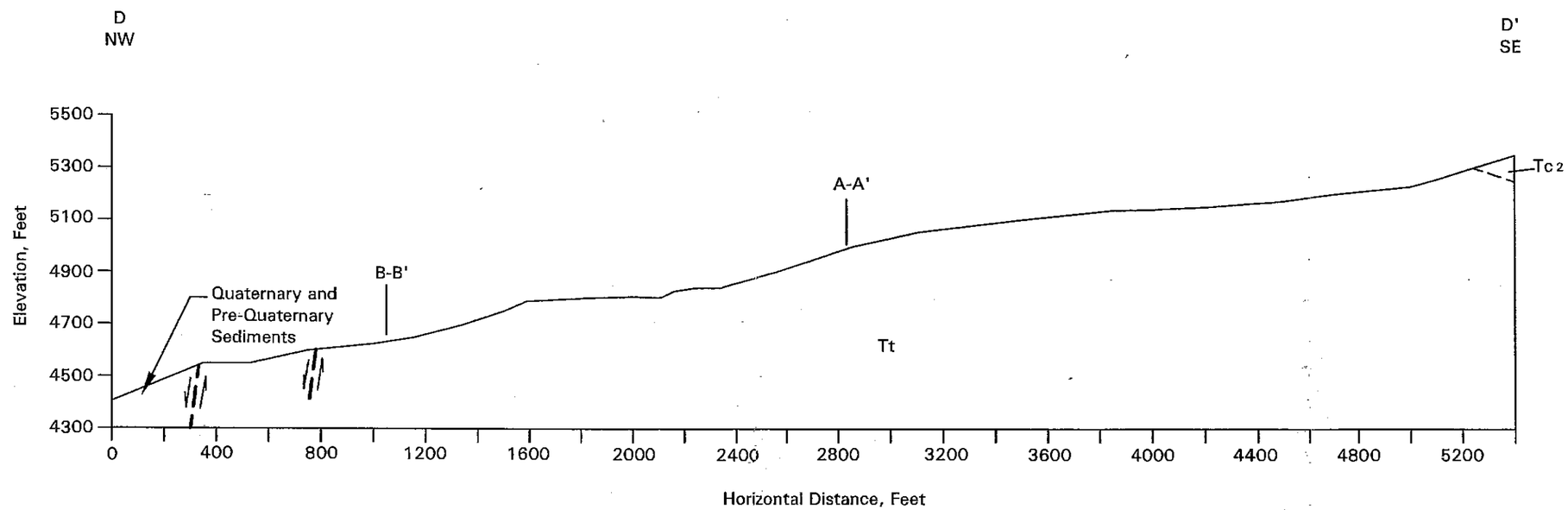
See Figure 5 for Location of Profile and Explanation of Symbols

1020320	AGEC	Cross Section B-B'	Figure 7



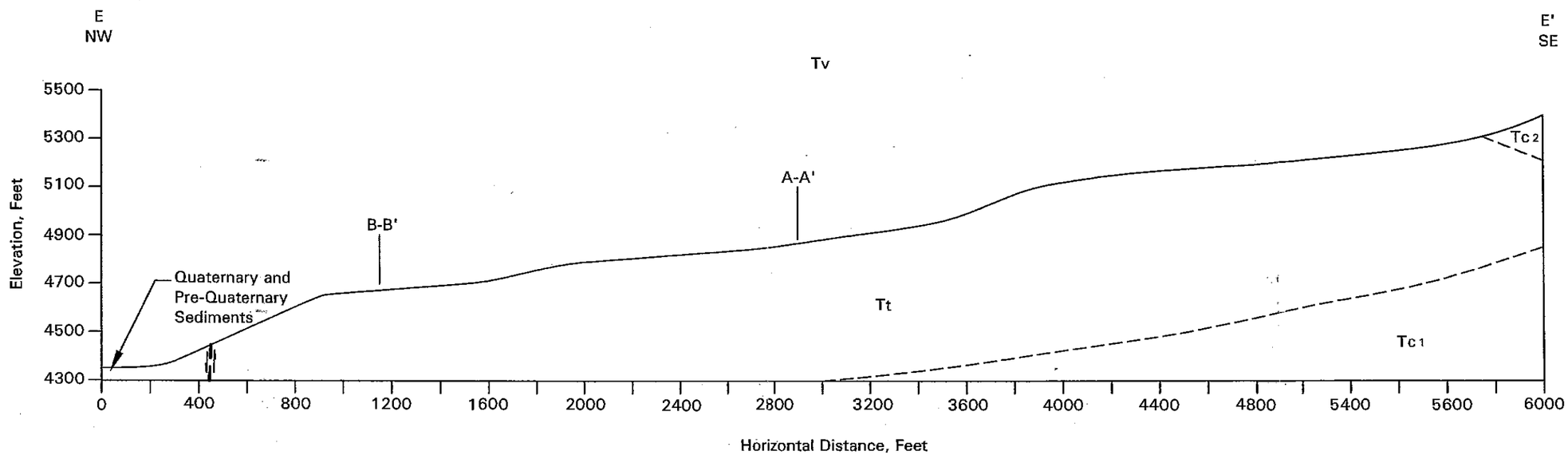
See Figure 5 for Location of Profile and Explanation of Symbols

1020320	AGES	Cross Section C-C'	Figure 8



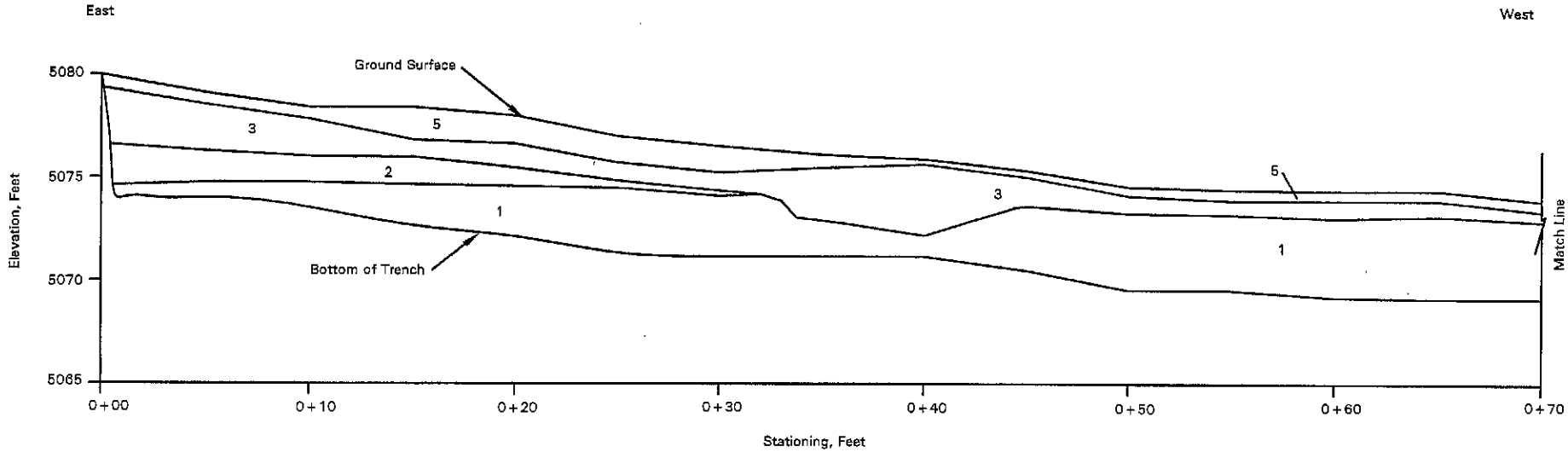
See Figure 5 for Location of Profile and Explanation of Symbols

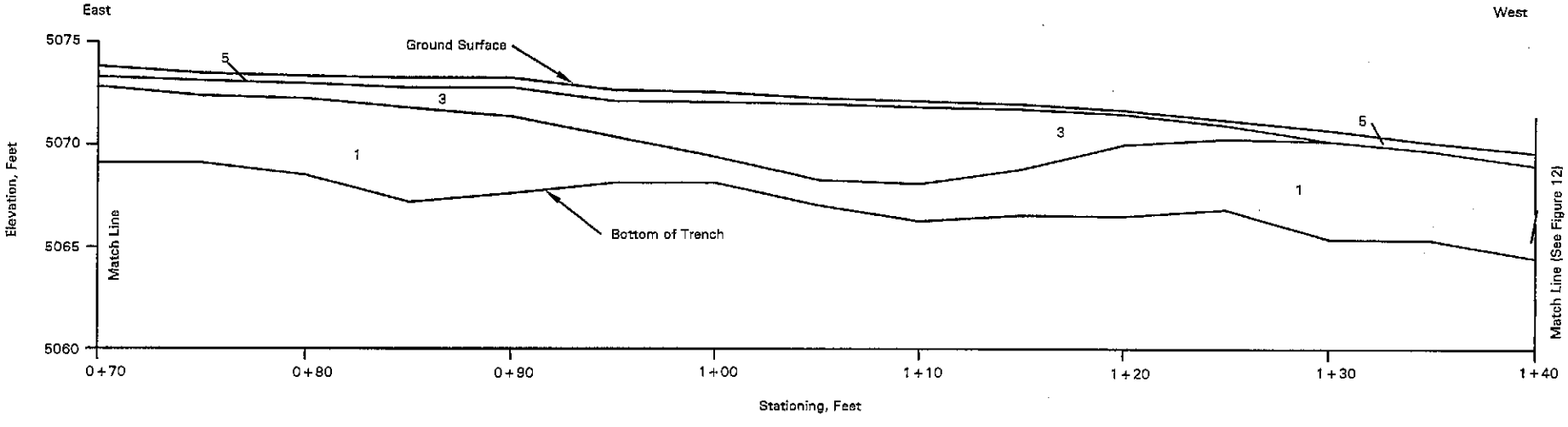
1020320	AGEC	Cross Section D-D'	Figure 9



See Figure 5 for Location of Profile and Explanation of Symbols

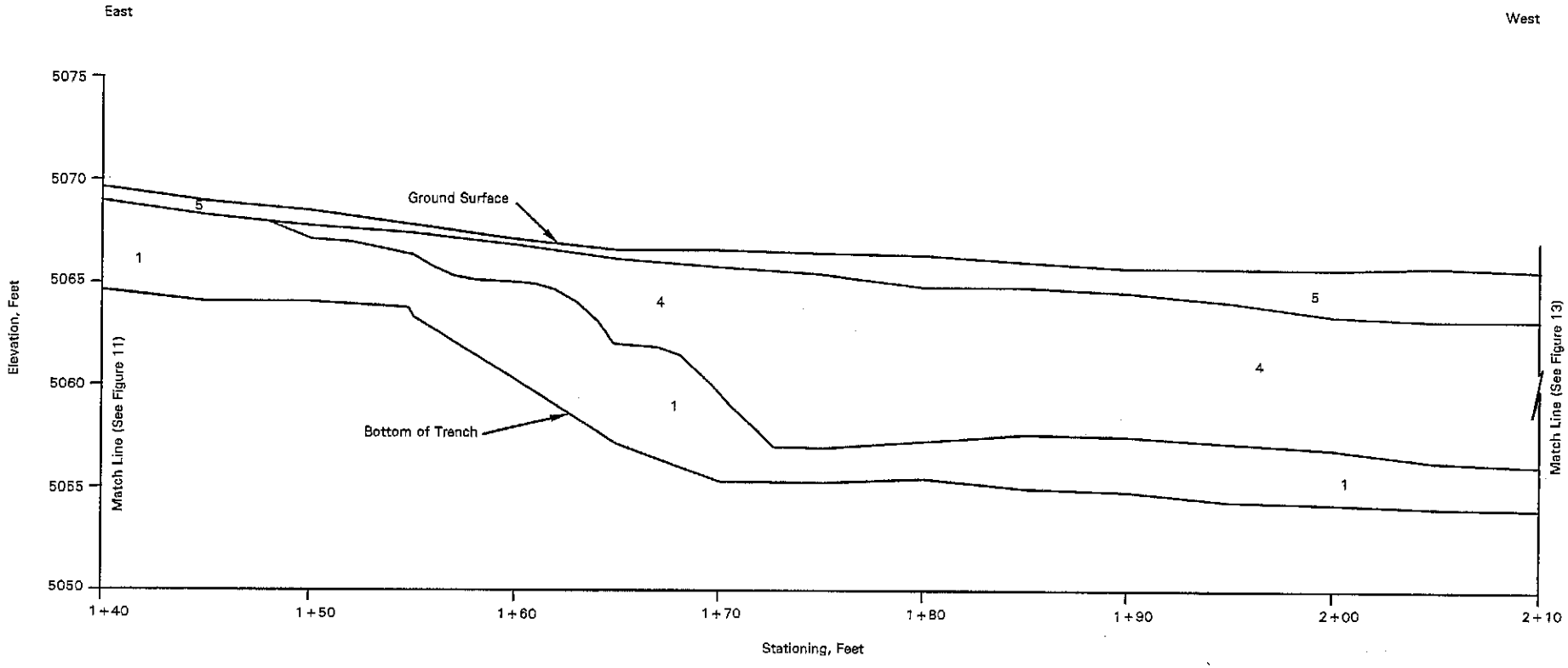
1020320	AGES	Cross Section E-E'	Figure 10





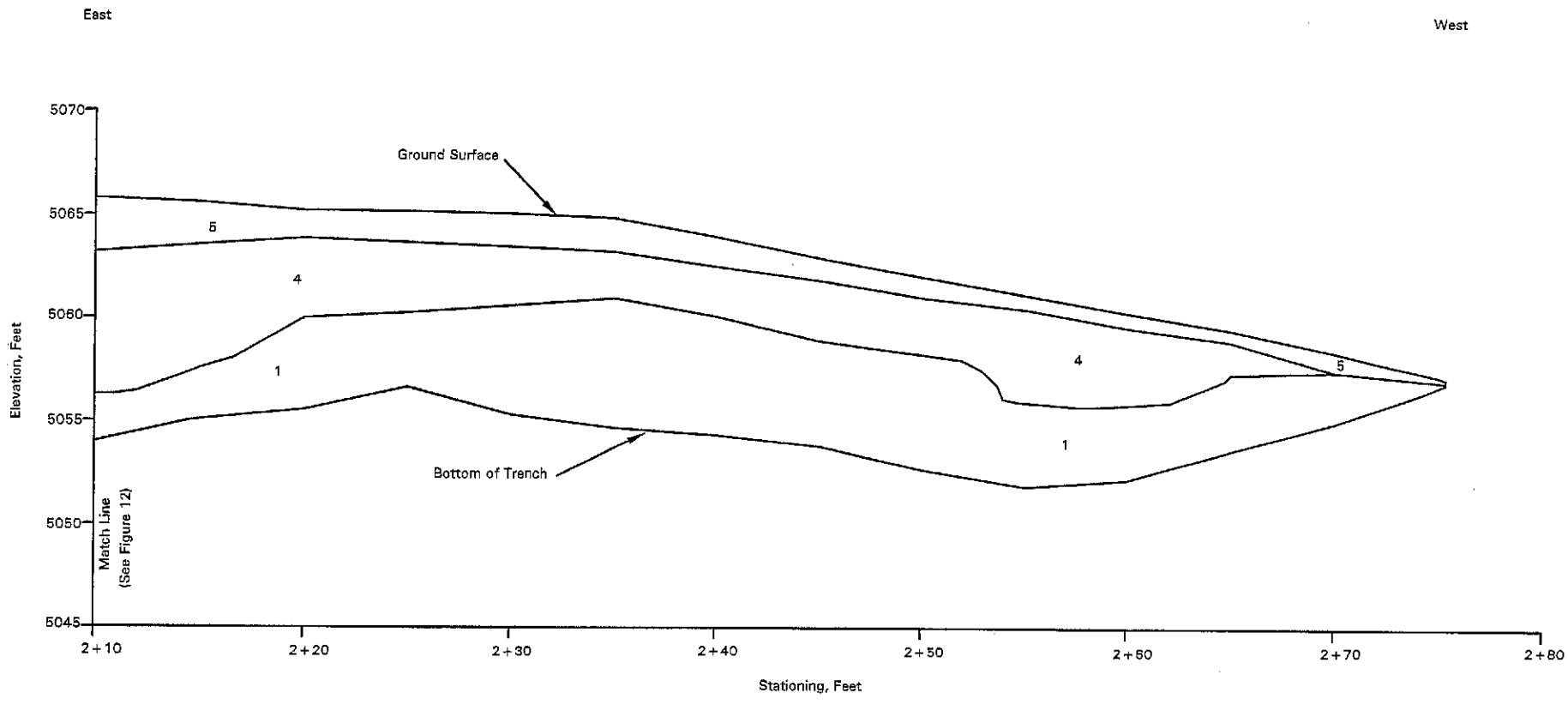
See Figure 14 for Description of Soil Units and Symbols

1020320	AGES	Log of Trench	Figure 11



See Figure 14 for Description of Soil Units and Symbols

1020320	AGRA	Log of Trench	Figure 12



See Figure 14 for Description of Soil Units and Symbols

1020320	AGRA	Log of Trench	Figure 13

DESCRIPTION OF SOIL UNITS AND SYMBOLS

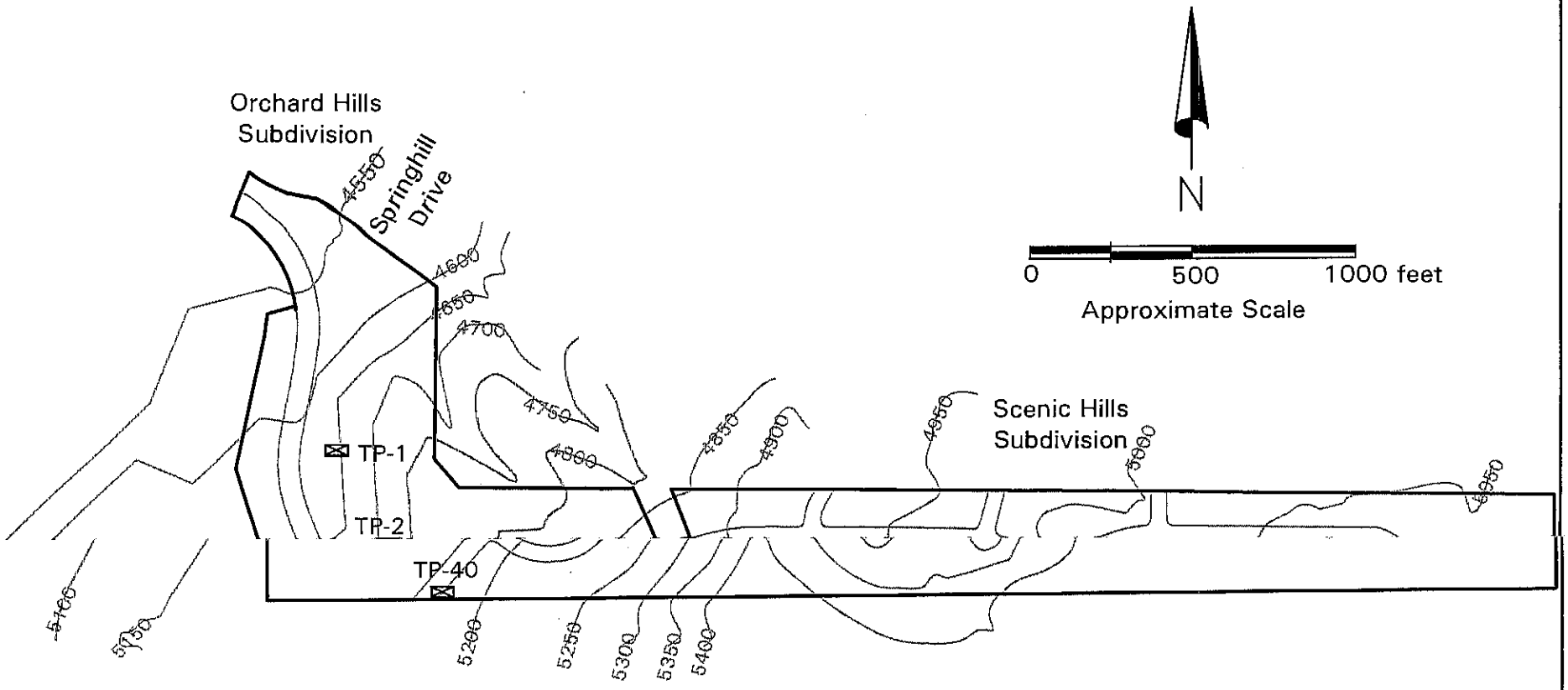
1 Lake Bonneville transgressive phase gravel and sand, poorly graded, slightly moist-moist, brown, clasts are well-rounded, moderately well-sorted, calcareous deposition on bottoms of clasts.

2 Alluvial sand and gravel, poorly graded, slightly moist, brown, well-rounded, poorly sorted, interbedded lenses of sand with gravel.

3 Alluvial silty clay to lean clay, dry, light brown, some roots, contains small pockets of gravel, slightly cemented, contains reworked clasts of units 1 and 5.

4 Alluvial sandy lean clay with gravel and cobbles, dry, brown, partially cemented, especially near the boundary with unit 1, contains reworked clasts of units 1 and 5, erosionally cuts unit 1.

5 Topsoil; lean clay, dry, dark brown, roots, especially in top 4-6 inches, occasional gravel.

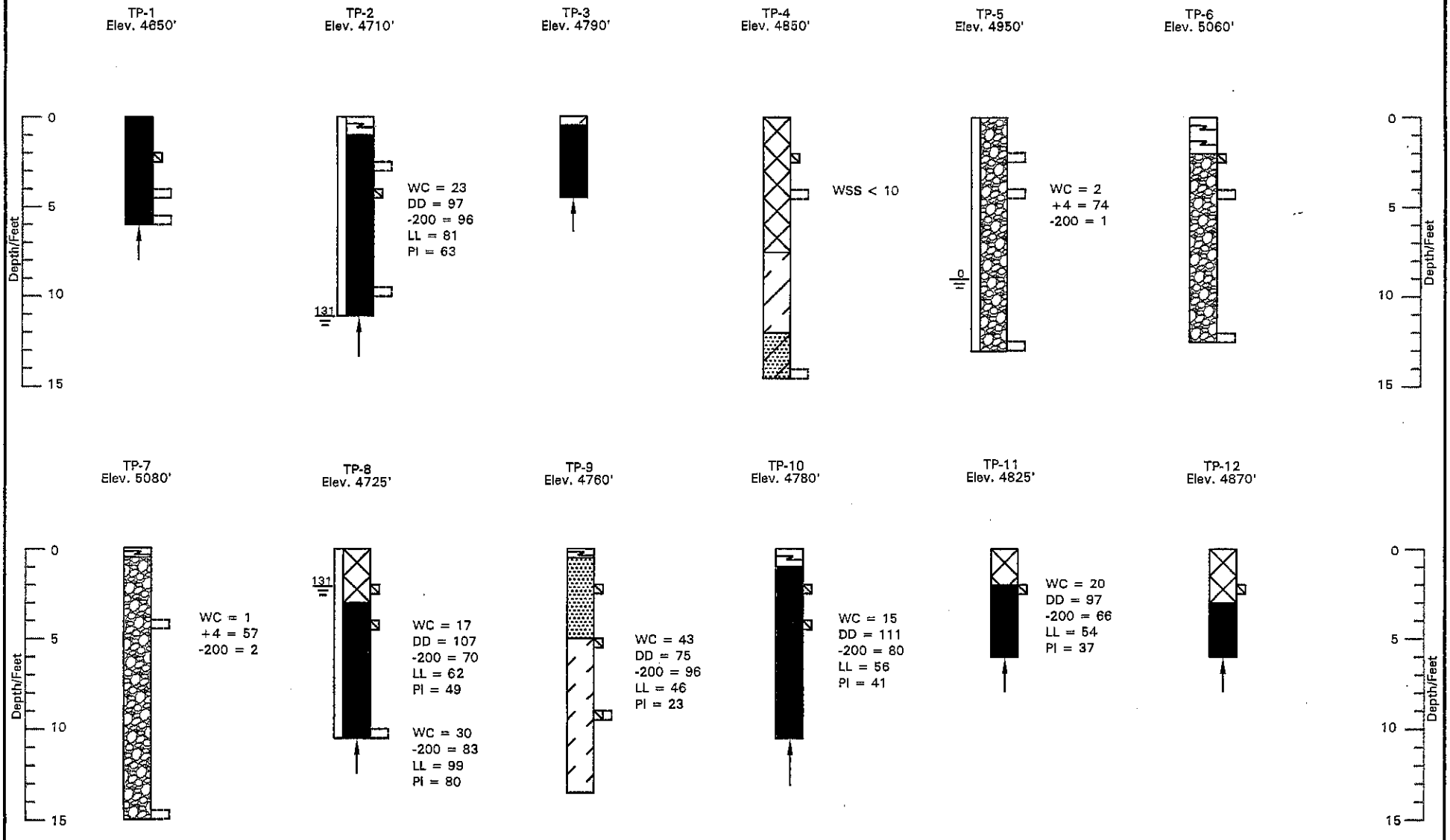


Legend:

- Trench
- Test Pit Location
- Boring Location
- Hand Excavation Location

EAGLEPOINTE DEVELOPMENT
NORTH SALT LAKE, UTAH

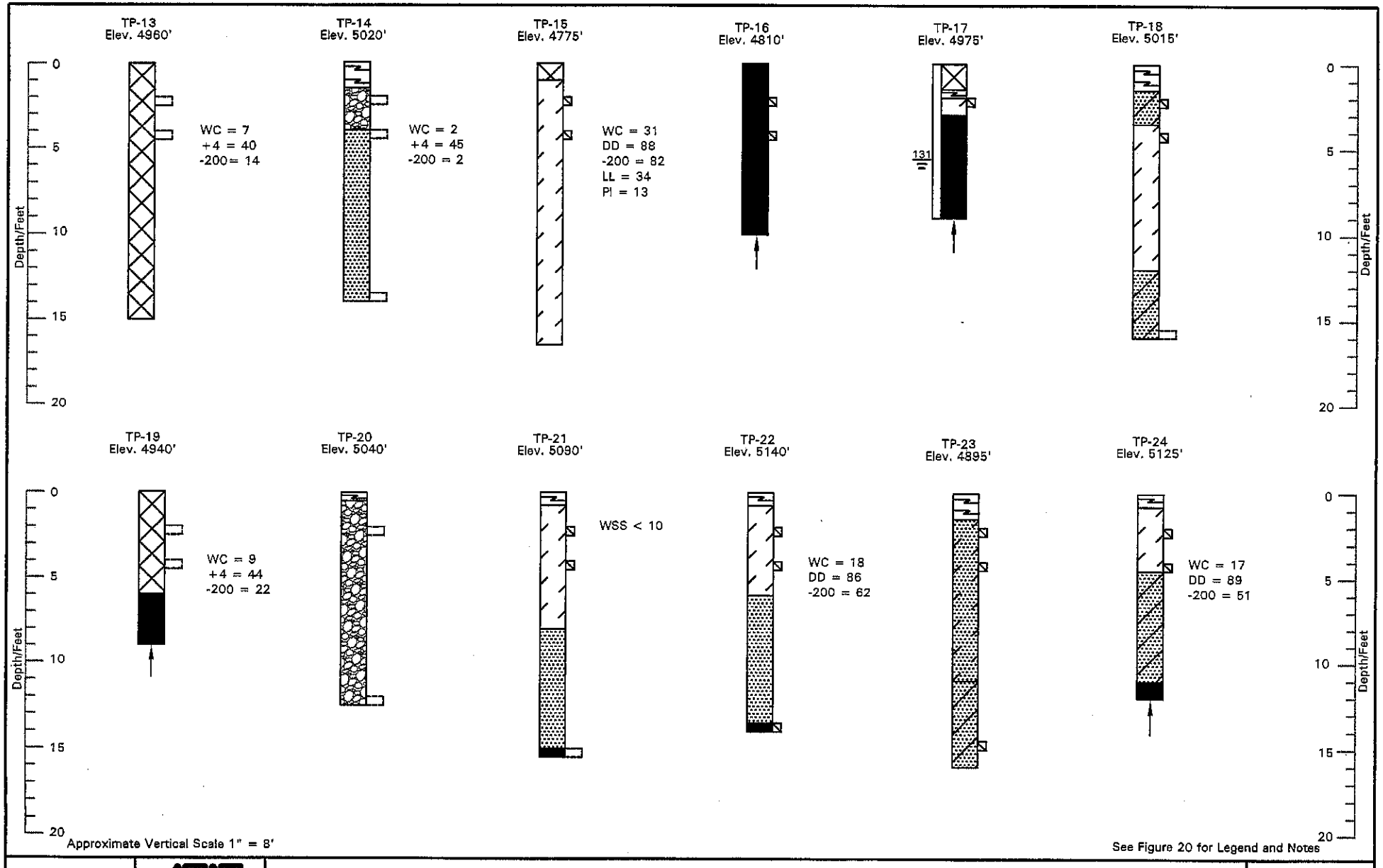
1020320	AGES	Locations of Test Pits, Hand Excavations, Boring and Stability Profiles	Figure 15



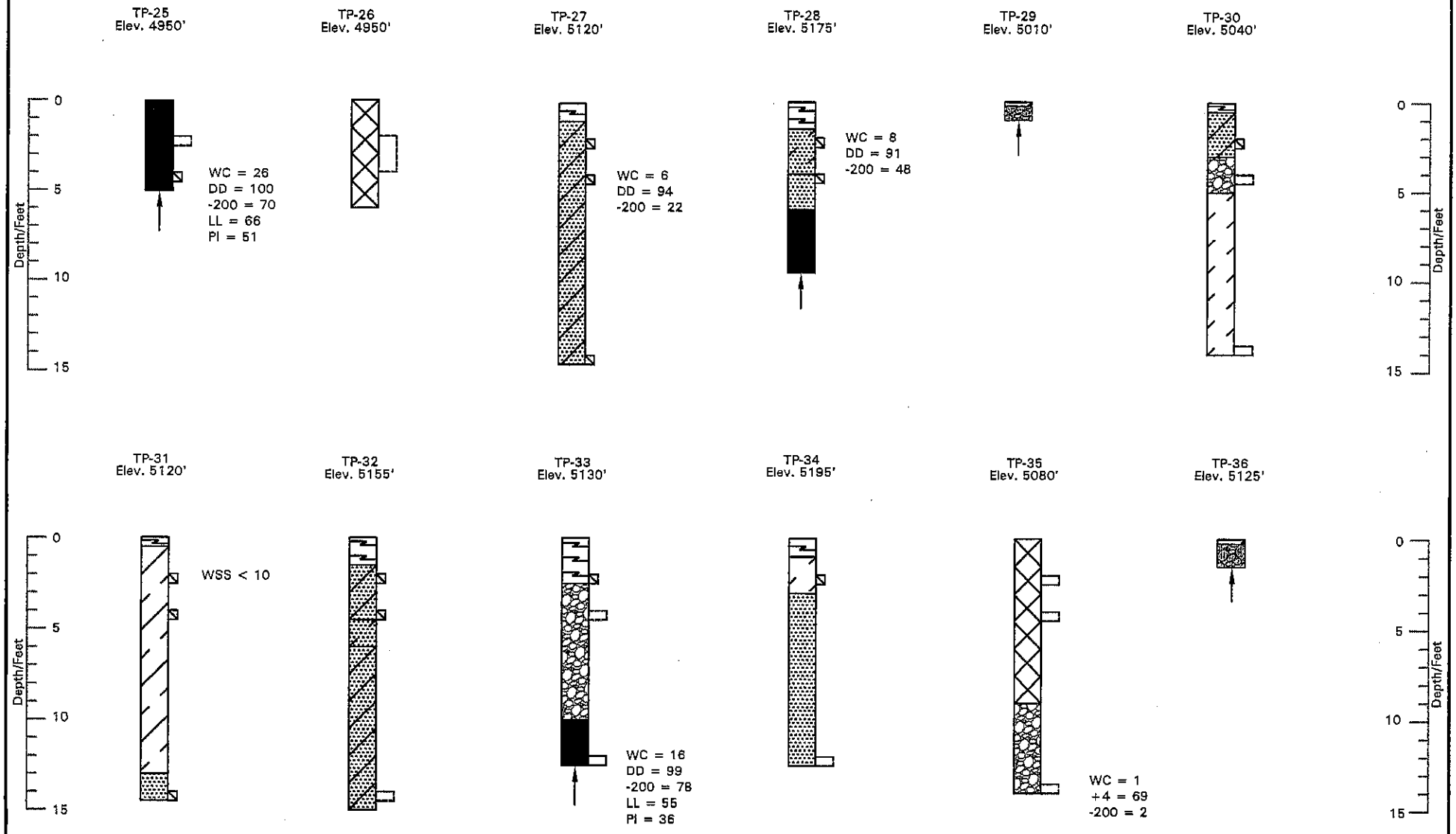
Approximate Vertical Scale 1" = 8'

See Figure 20 for Legend and Notes

1020320	AGES	Logs of Test Pits	Figure 16



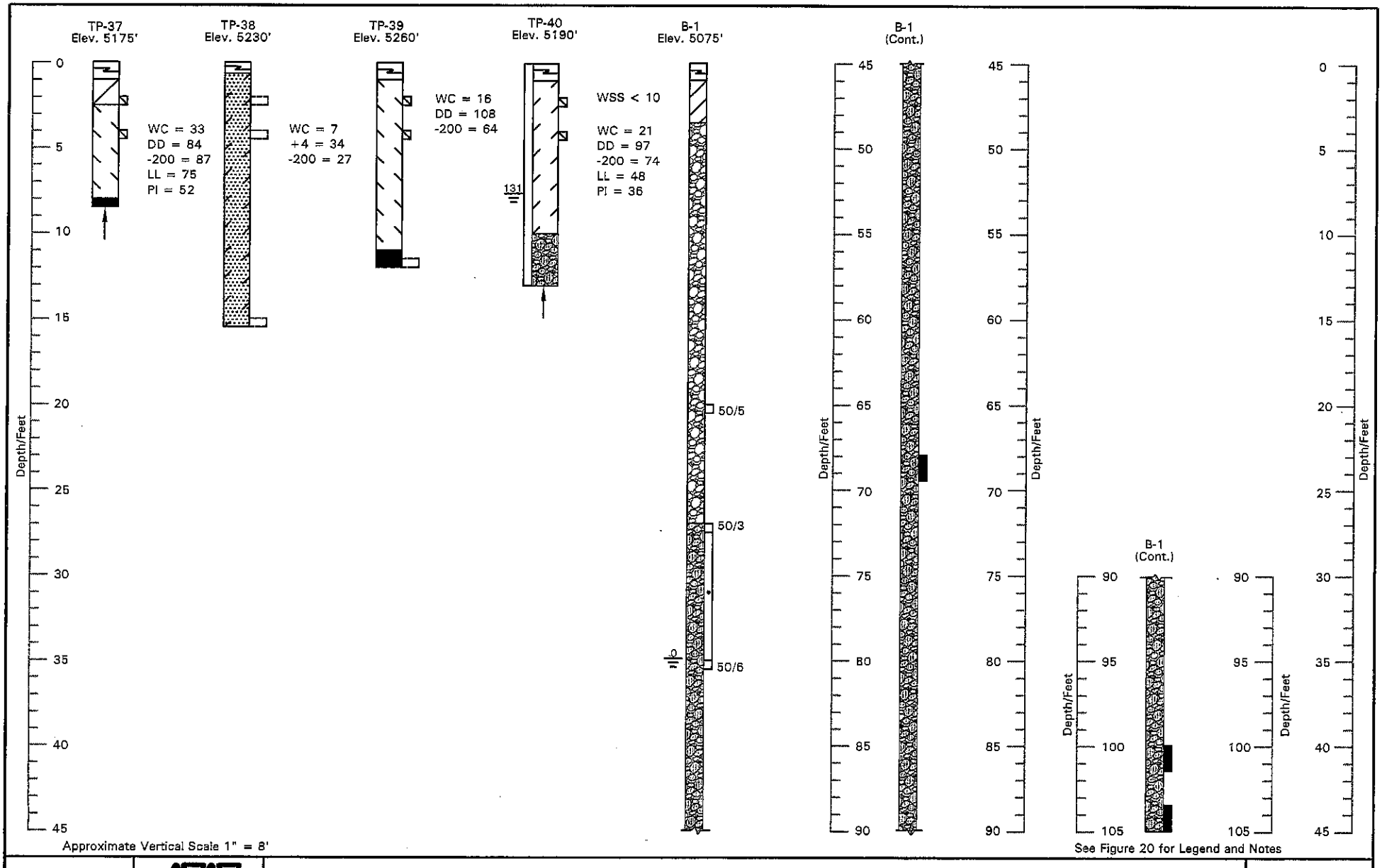
1020320 | AGES | Logs of Test Pits | Figure 17



Approximate Vertical Scale 1" = 8'

See Figure 20 for Legend and Notes

1020320	AGES	Logs of Test Pits	Figure 18



1020320

Logs of Test Pits and Boring

Figure 19

LEGEND:

Fill; lean clay, small to moderate amounts of sand and gravel, occasional clayey gravel, silty sand with gravel was encountered in Test Pit TP-13, pieces of asphalt, wood and debris were encountered in Test Pit TP-12, slightly moist, mottled, brownish-orange, brown, gray, red.

Topsoil; lean clay and silt, small to moderate amount of sand, occasional gravel, slightly moist to wet, brown to dark brown, roots.

Lean Clay (CL); small to moderate amounts of sand, occasional silt layers and gravel, medium stiff to very stiff, slightly moist to wet, brown, gray, red, some iron oxide staining.

Fat Clay (CH); ranges from medium to high plasticity, stiff to very stiff, slightly moist to moist, red, gray, white calcareous deposits. Occasional cobbles and boulders were encountered in Test Pit TP-37.

Silt and Lean Clay (ML/CL); interlayered, small amounts of sand, some gravels, medium dense/stiff to very stiff, slightly moist, brown to gray.

Clayey Sand (SC); occasional gravel, medium dense, slightly moist, brown, orangish brown, light gray, calareous.

Silty Sand (SM); small to moderate amounts of silt, occasional gravel and cobbles, medium dense, slightly moist, brown to orangish brown.

Poorly Graded Sand (SP); occasional silt, gravel, cobbles and boulders up to 2 feet in size, medium dense, slightly moist, light brown to brown, occasional iron oxide staining. slightly cemented sand in Test Pits TP-20 and TP-25.

Poorly Gradad Gravel (GP); small to moderate amount of silt and sand, cobbles and occasional boulders up to approximately 3 feet in size, medium dense, slightly moist, brown to brownish red, occasional slightly cemented gravel layers.

Tuffaceous Bedrock; primarily claystone and siltstone, occasional sandstone, minor conglomerate, slightly to highly weathered, small to moderate amount of sand, some gravelly layers, high plasticity, firm to very hard, slightly moist to wet, primarily gray to white with occasional red areas.

Bedrock; primarily conglomerate and sendstone, poorly to well-cemented, hard to very hard, slightly moist to wet, brown to reddish brown to gray, fractured clasts of limestone and quartzite.

Indicates relatively undisturbed hand drive sample taken.

10/12 California Drive sample taken. The symbol 10/12 indicates that 10 blows from a 140 pound automatic hammer falling 30 inches were required to drive the sampler 12 inches.

LEGEND CONT.

Indicates standard penetration test taken.

Indicates core sample taken.

Indicates disturbed sample taken.

Indicates relatively undisturbed block sample taken.

Indicates slotted 1 ½ inch PVC pipe installed in the test pit to the depth shown.

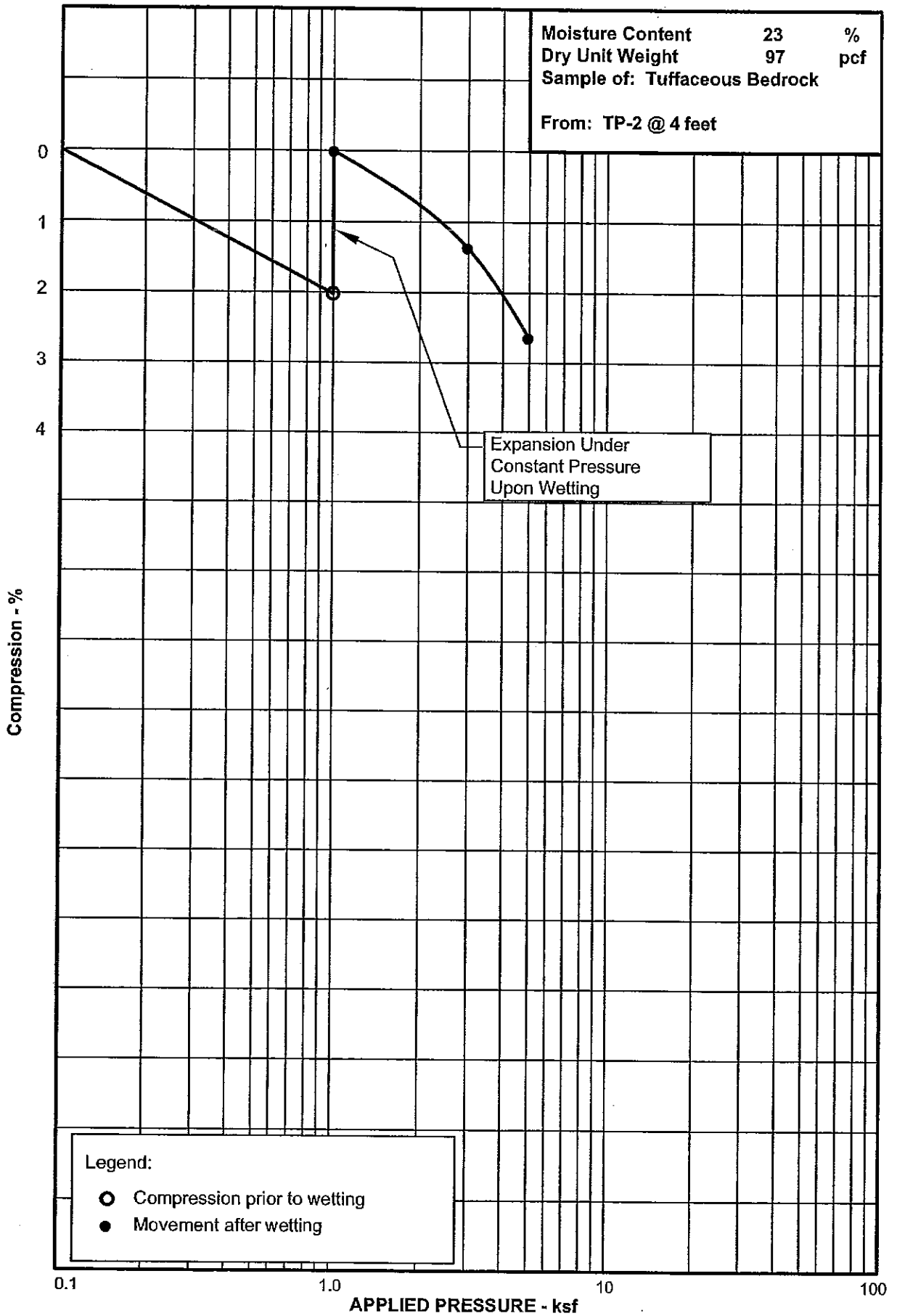
10 Indicates the depth to free water and the number days after excavating the measurement was taken.

Indicates practical backhoe refusal.

NOTES:

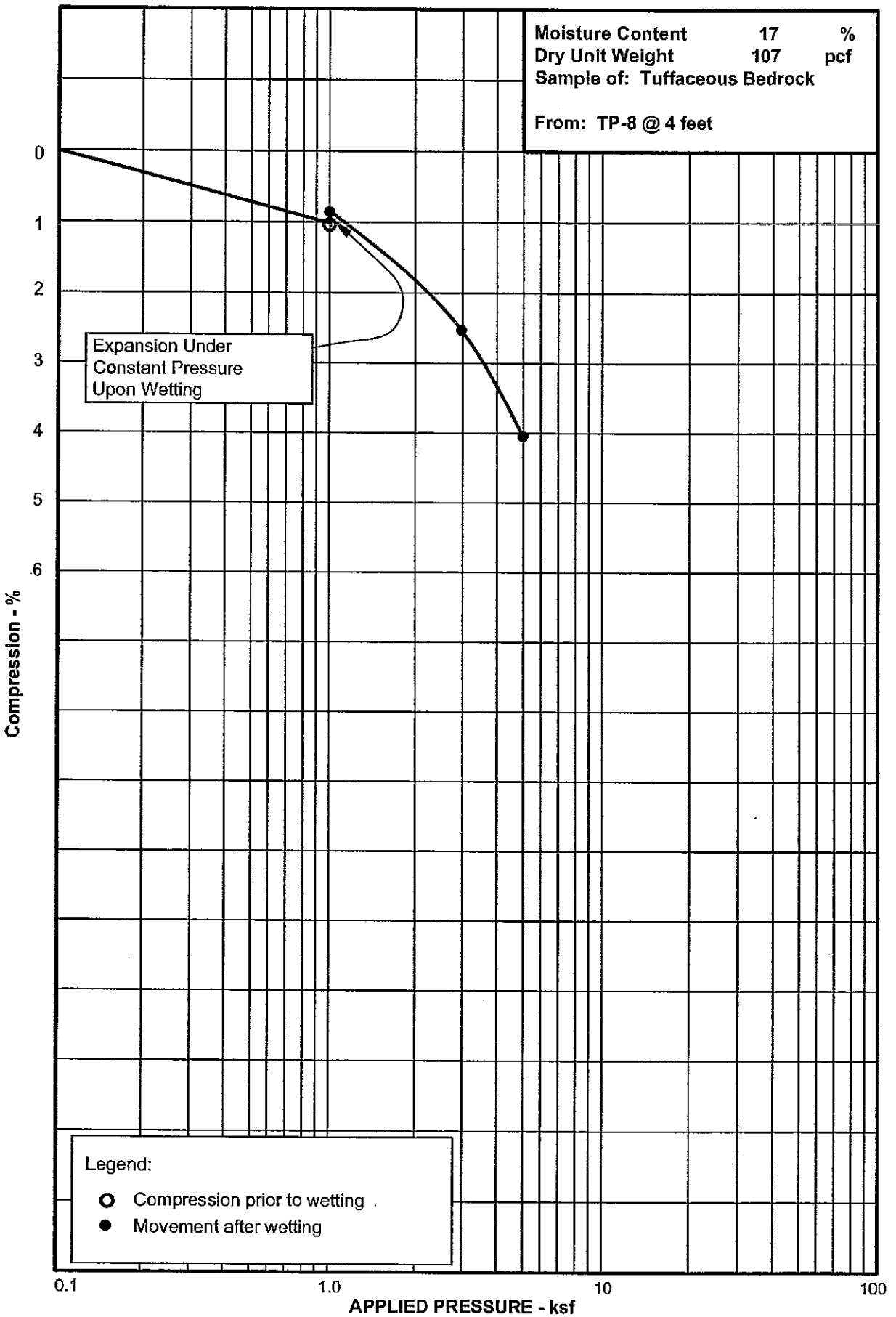
1. Test pits were excavated on June 20, 21, 24, 25 and 27, 2002 with a rubber-tired backhoe. The boring was drilled on August 20, 2002 with 8-inch diameter hollowstem auger.
2. Locations of the test pits and boring were measured approximately by pacing from features shown on the site plan provided.
3. Elevations of the test pits and boring were estimated by interpolating between contours presented on Figure 15.
4. The test pit and boring locations and elevations should be considered accurate only to the degree implied by the method used.
5. The lines between the materials shown on the test pit and boring logs represent the approximate boundaries between material types and the transitions may be gradual.
6. Water level readings shown on the logs were made at the time and under the conditions indicated. Fluctuations in the water level may occur with time.
7. WC = Water Content (%);
   DD = Dry Density (pcf);
   +4 = Percent Retained on the No. 4 Sieve;
   -200 = Percent Passing No. 200 Sieve;
   LL = Liquid Limit (%);
   PI = Plasticity Index (%);
   UC = Unconfined Compressive Strength (psf);
   WSS = Water Soluble Sulfates (ppm).

Applied Geotechnical Engineering Consultants, Inc.



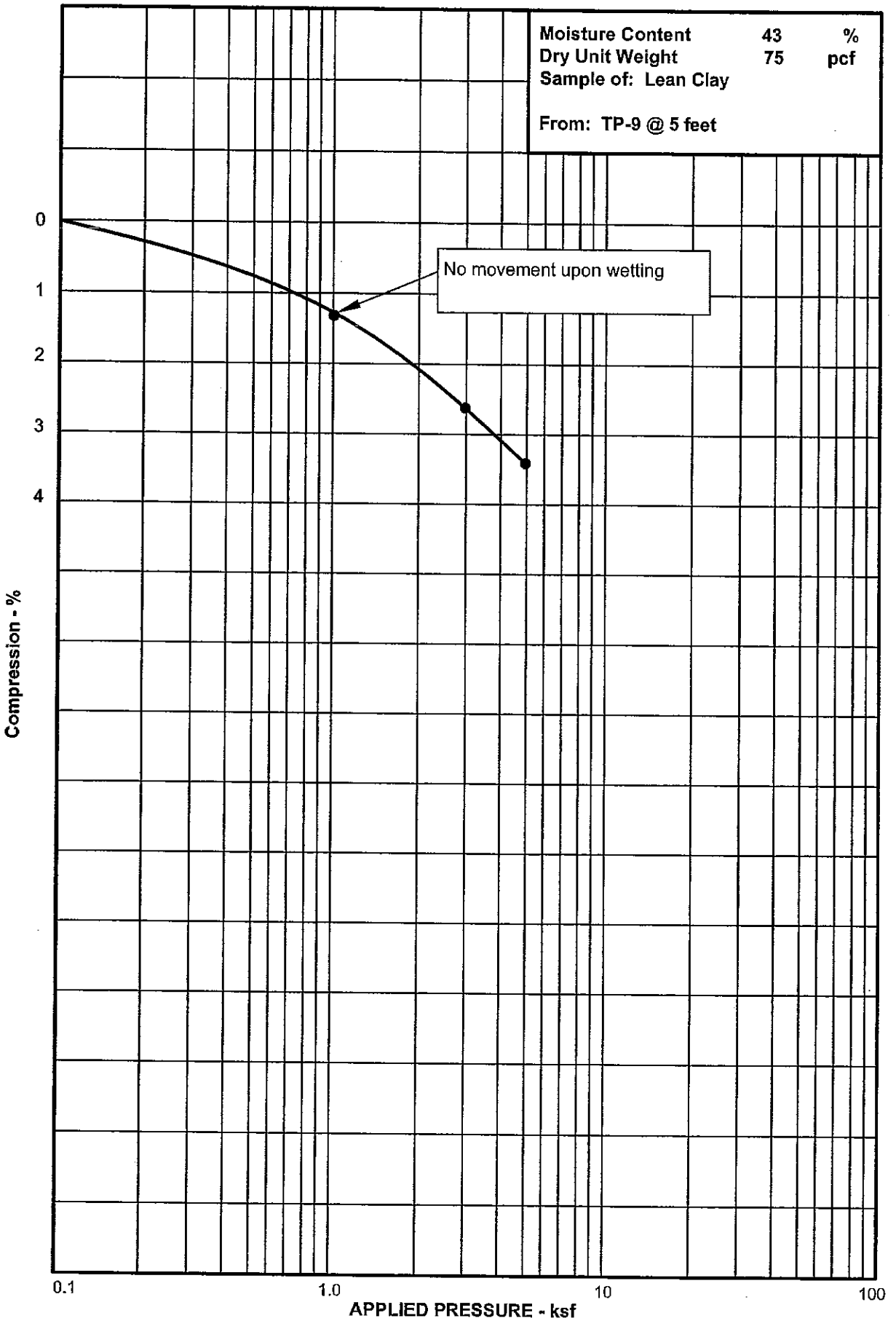
Project No. 1020320 CONSOLIDATION TEST RESULTS Figure 21

Applied Geotechnical Engineering Consultants, Inc.



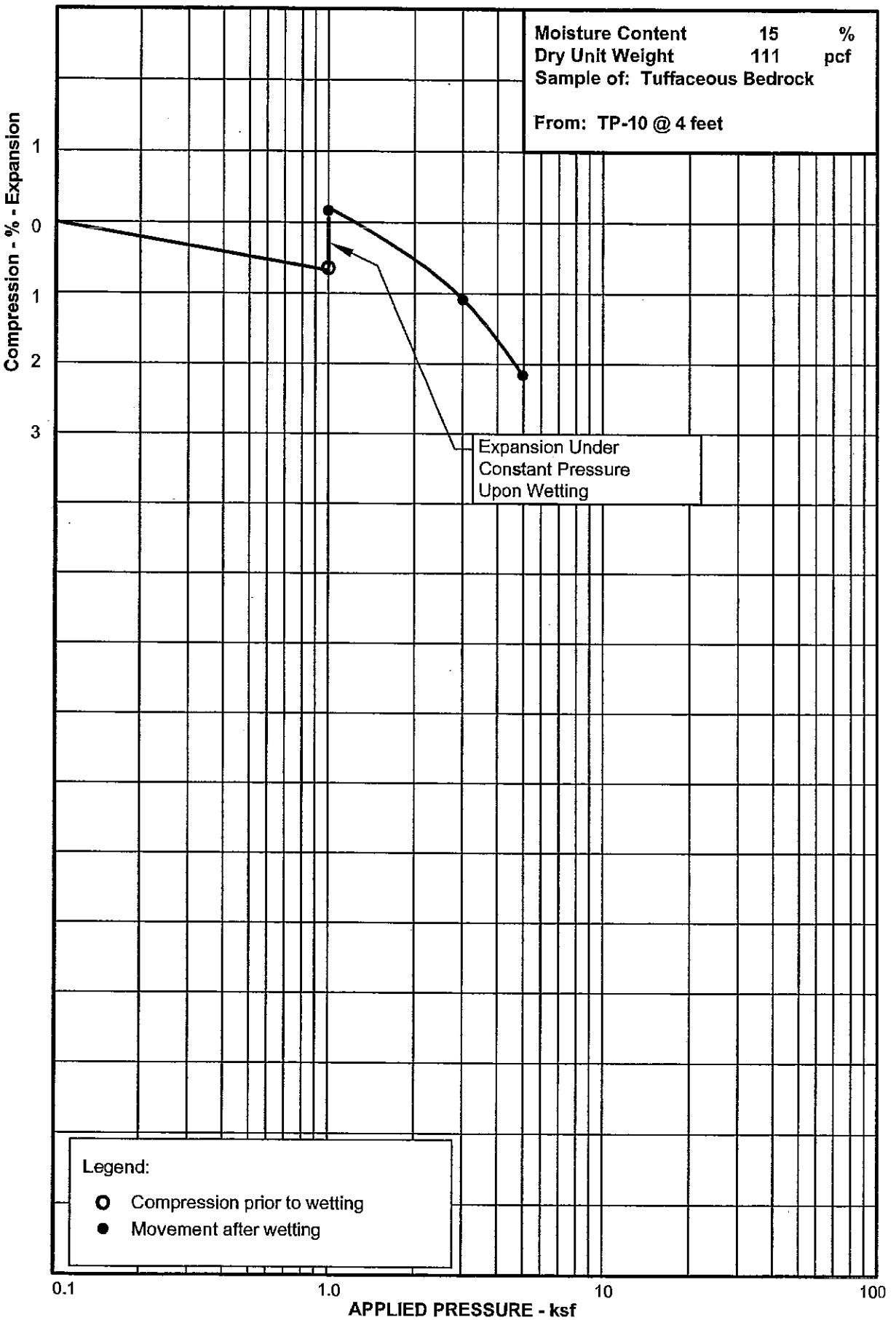
Project No. 1020320 CONSOLIDATION TEST RESULTS Figure 22

Applied Geotechnical Engineering Consultants, Inc.



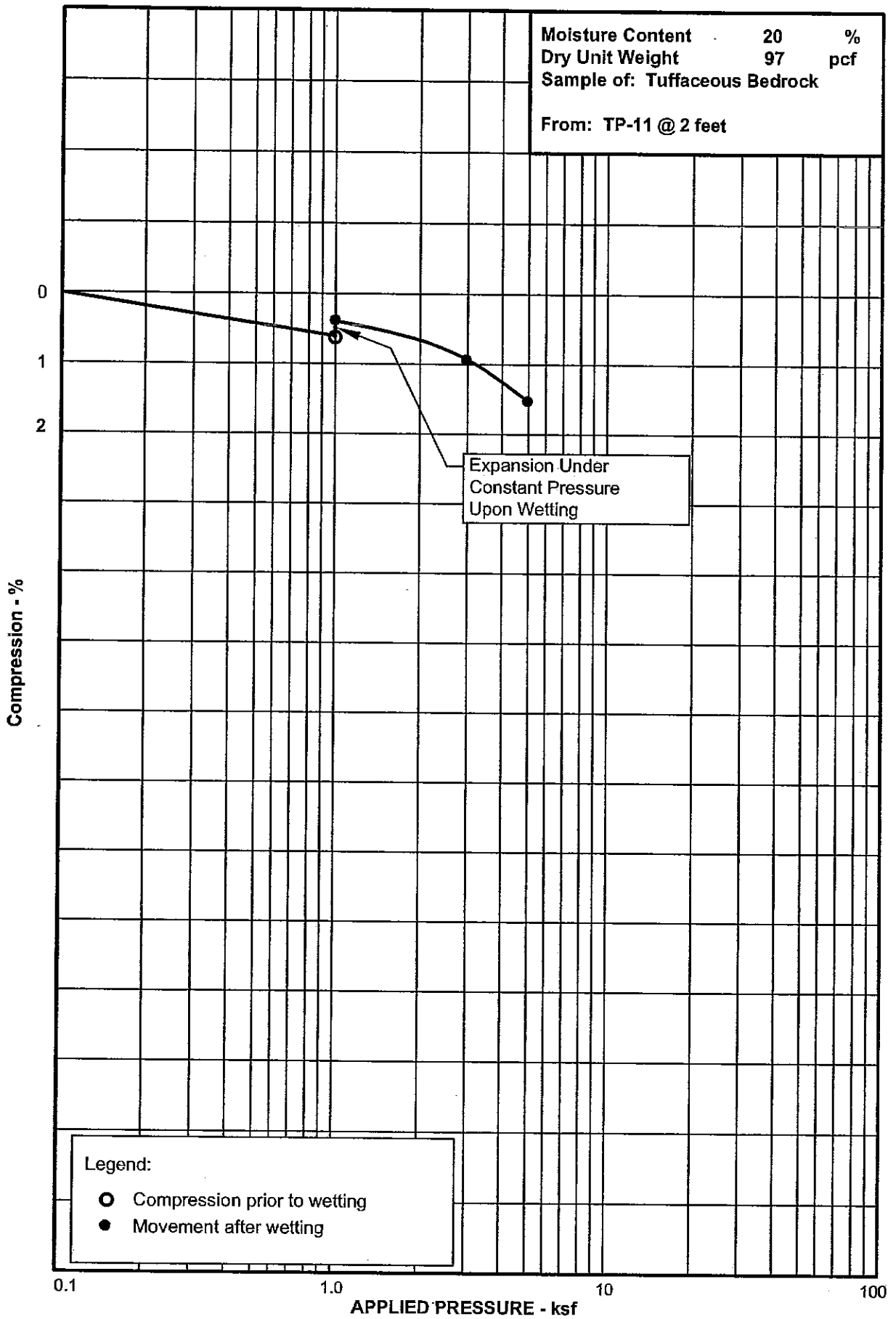
Project No. 1020320 CONSOLIDATION TEST RESULTS Figure 23

Applied Geotechnical Engineering Consultants, Inc.



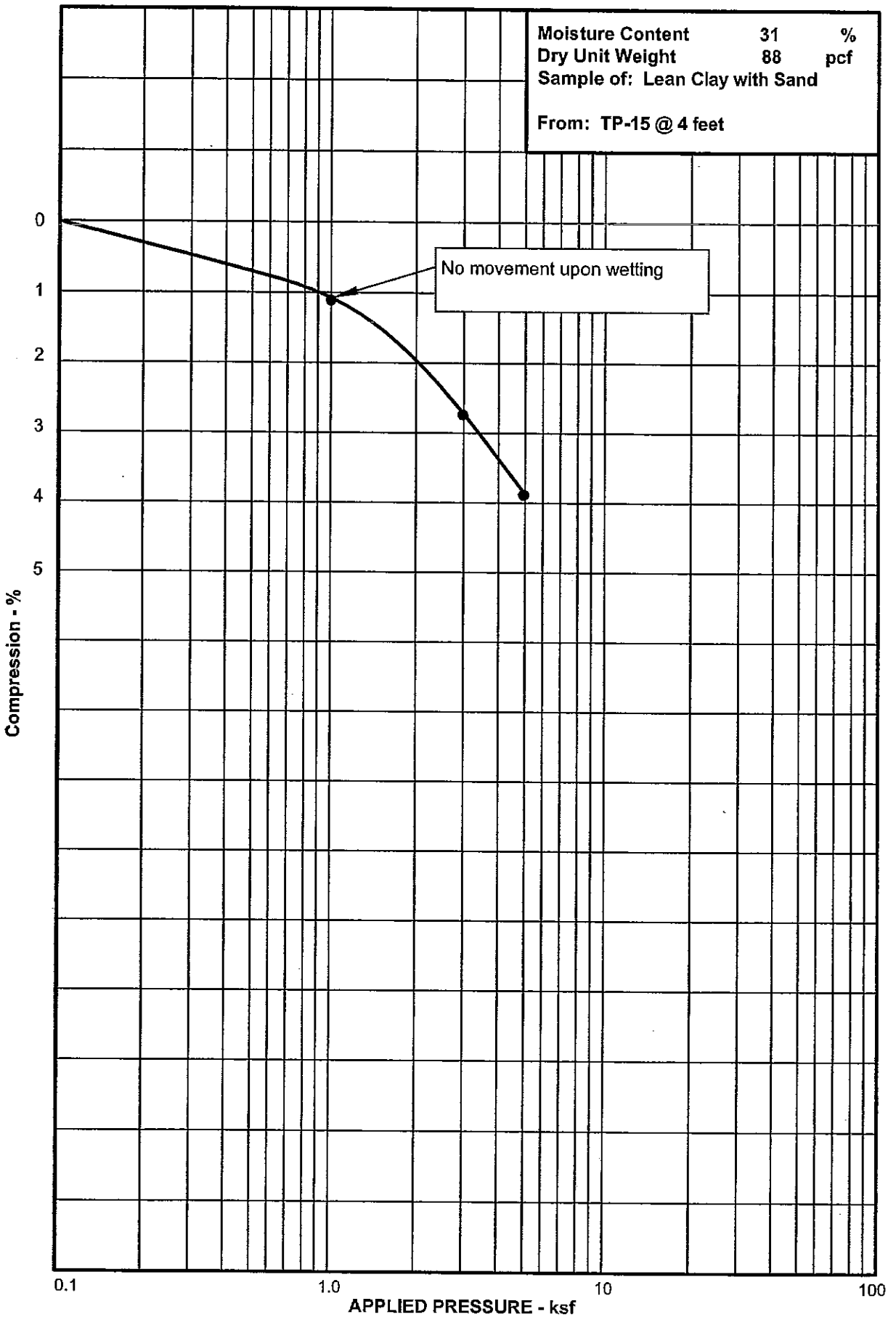
Project No. 1020320 CONSOLIDATION TEST RESULTS Figure 24

Applied Geotechnical Engineering Consultants, Inc.



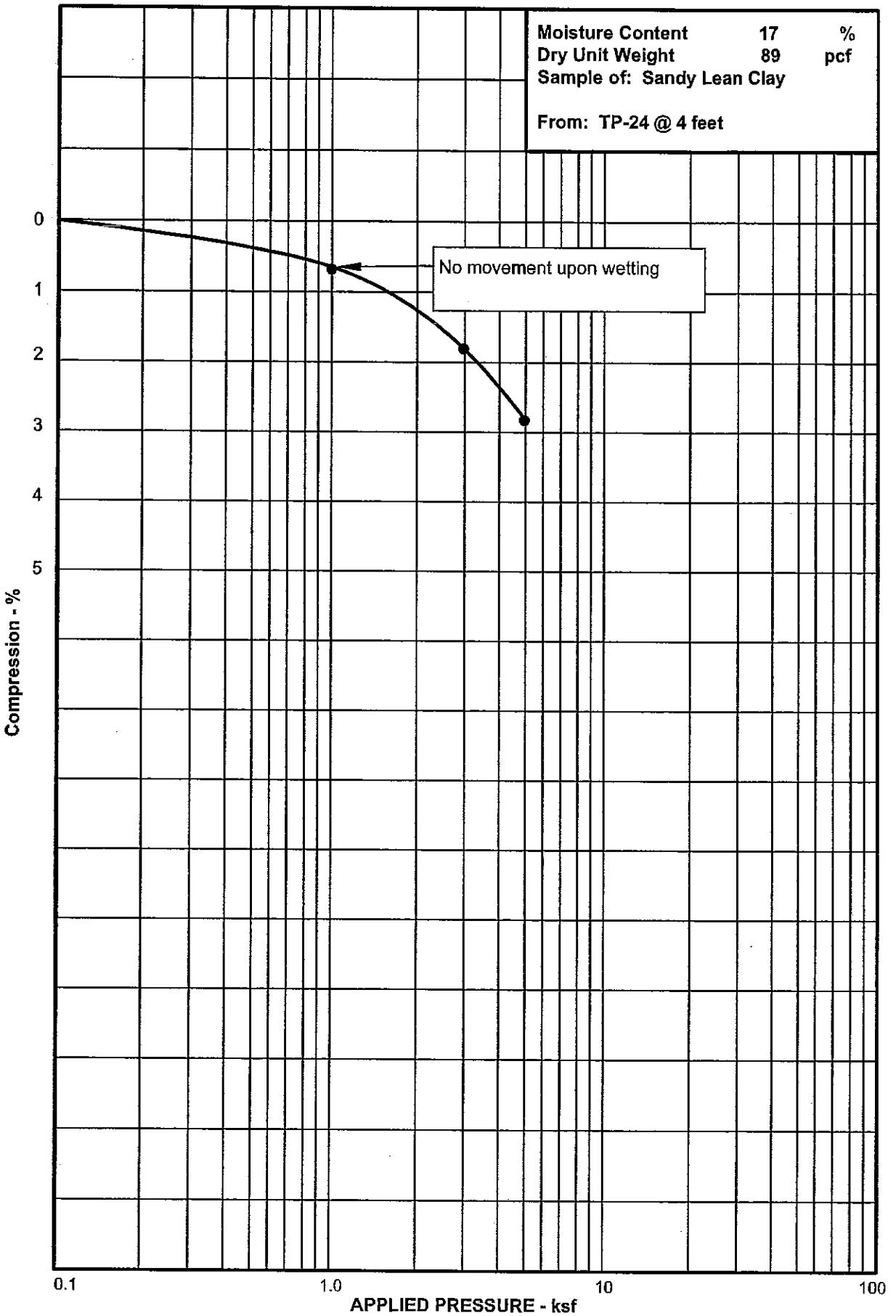
Project No. 1020320 CONSOLIDATION TEST RESULTS Figure 25

Applied Geotechnical Engineering Consultants, Inc.



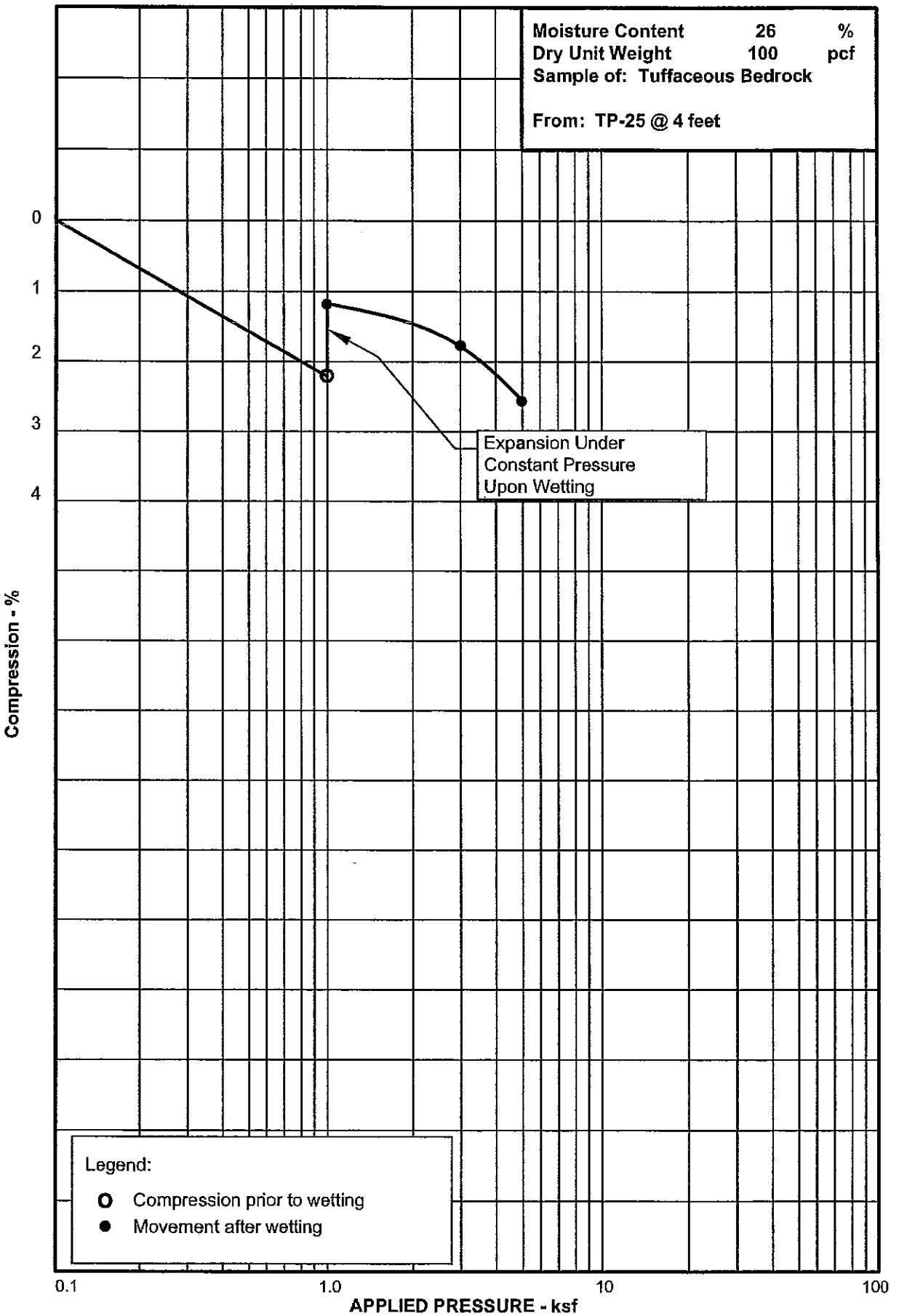
Project No. 1020320 CONSOLIDATION TEST RESULTS Figure 26

Applied Geotechnical Engineering Consultants, Inc.



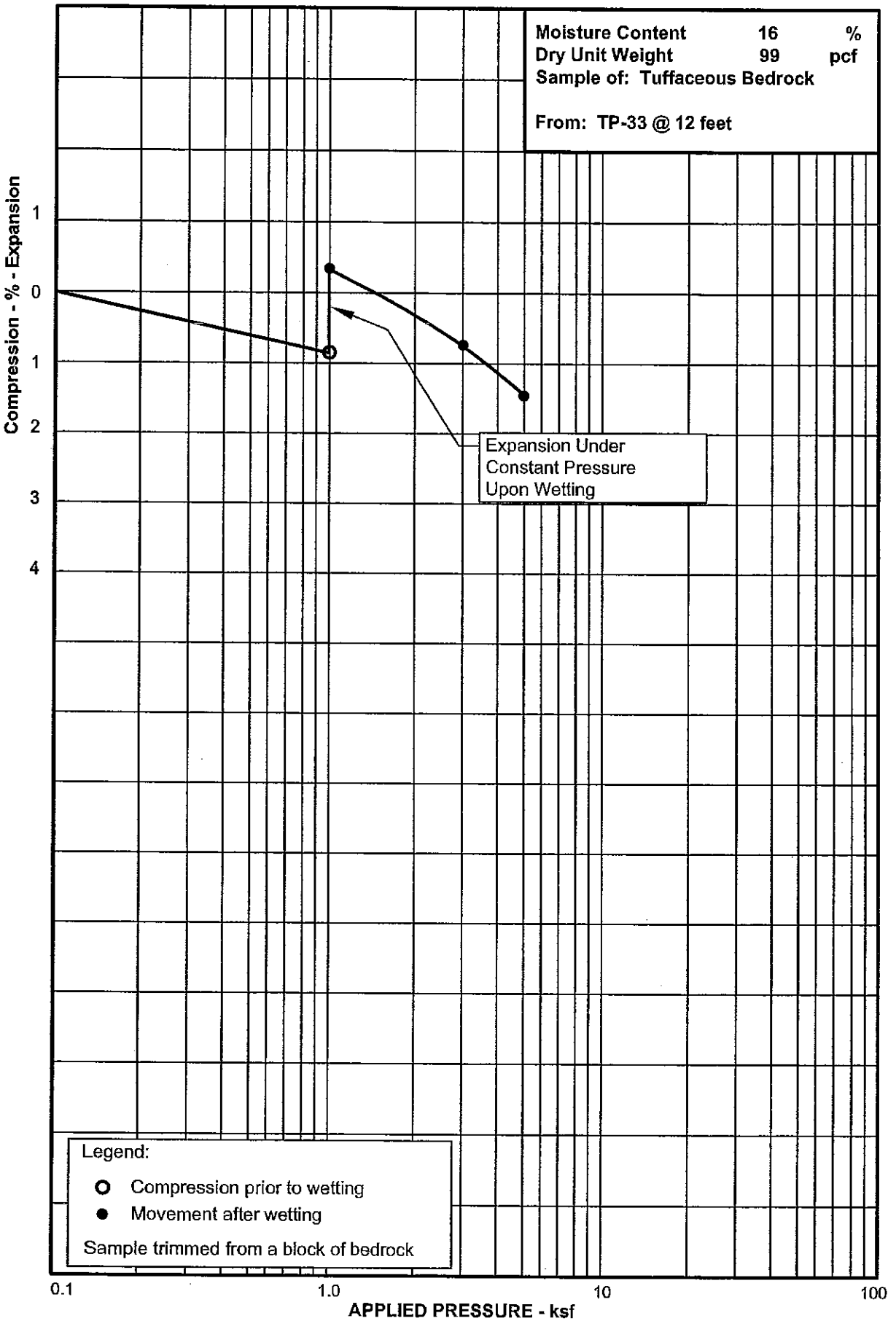
Project No. 1020320 CONSOLIDATION TEST RESULTS Figure 27

Applied Geotechnical Engineering Consultants, Inc.



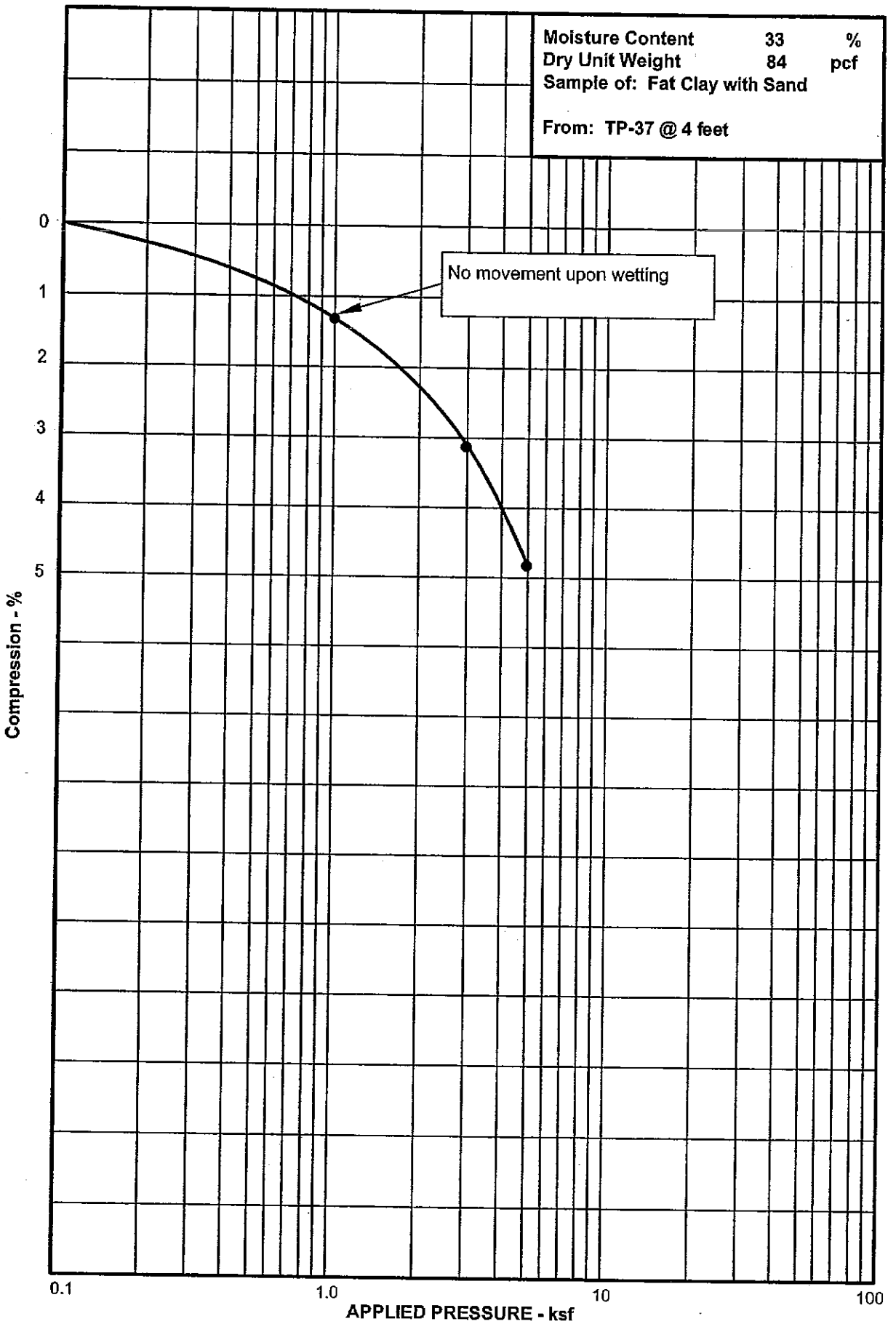
Project No. 1020320 CONSOLIDATION TEST RESULTS Figure 28

Applied Geotechnical Engineering Consultants, Inc.



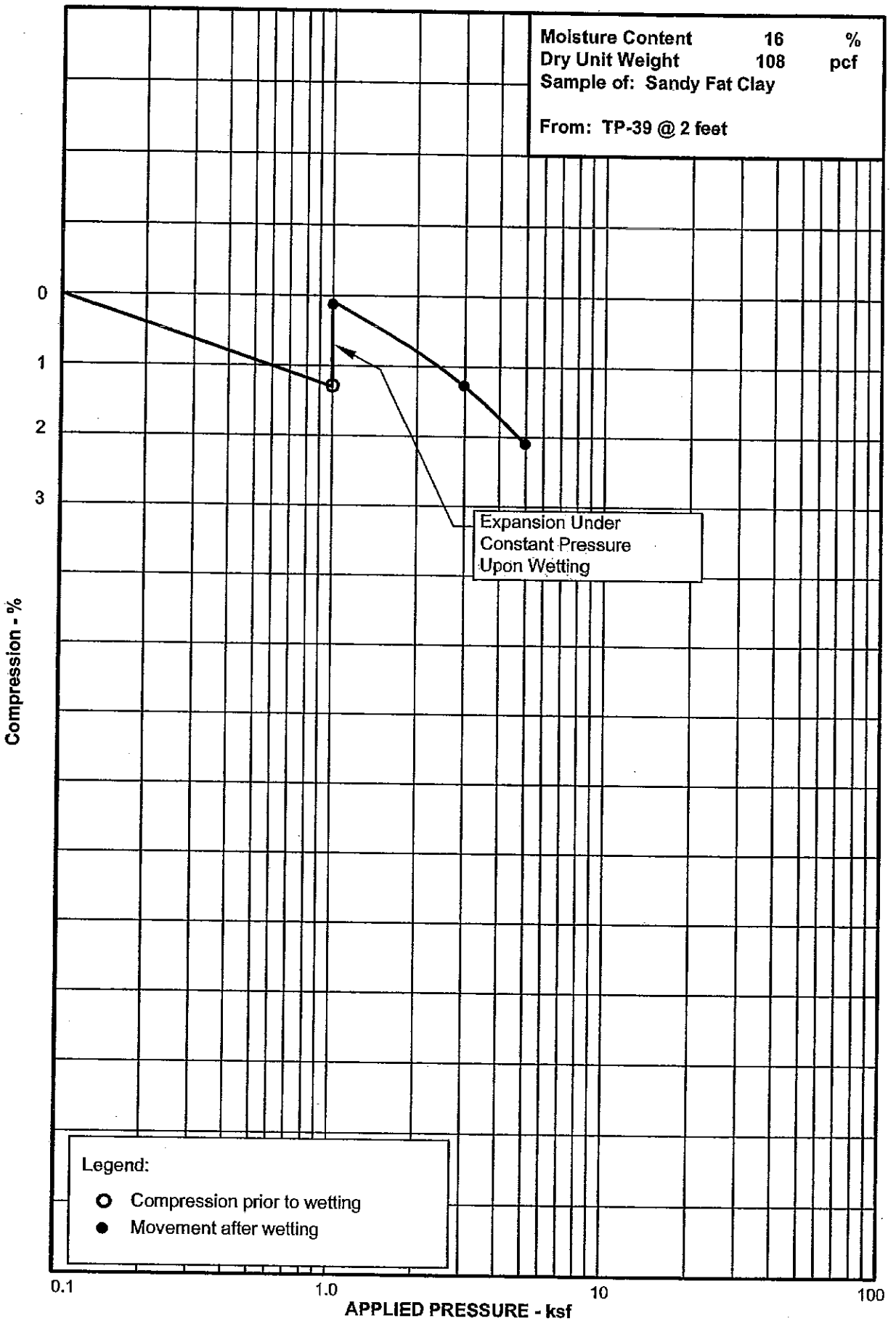
Project No. 1020320 **CONSOLIDATION TEST RESULTS** **Figure 29**

Applied Geotechnical Engineering Consultants, Inc.



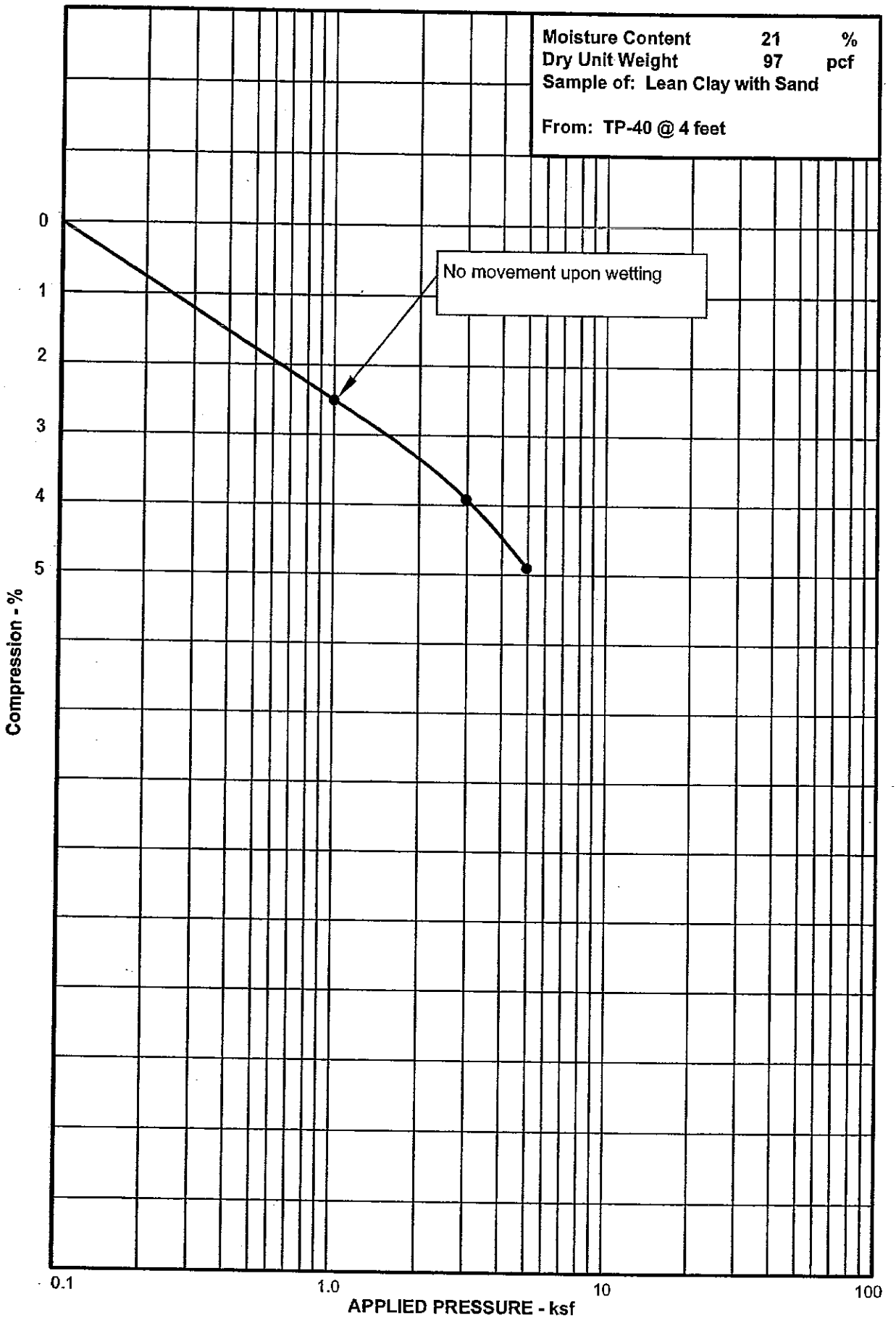
Project No. 1020320 CONSOLIDATION TEST RESULTS Figure 30

Applied Geotechnical Engineering Consultants, Inc.



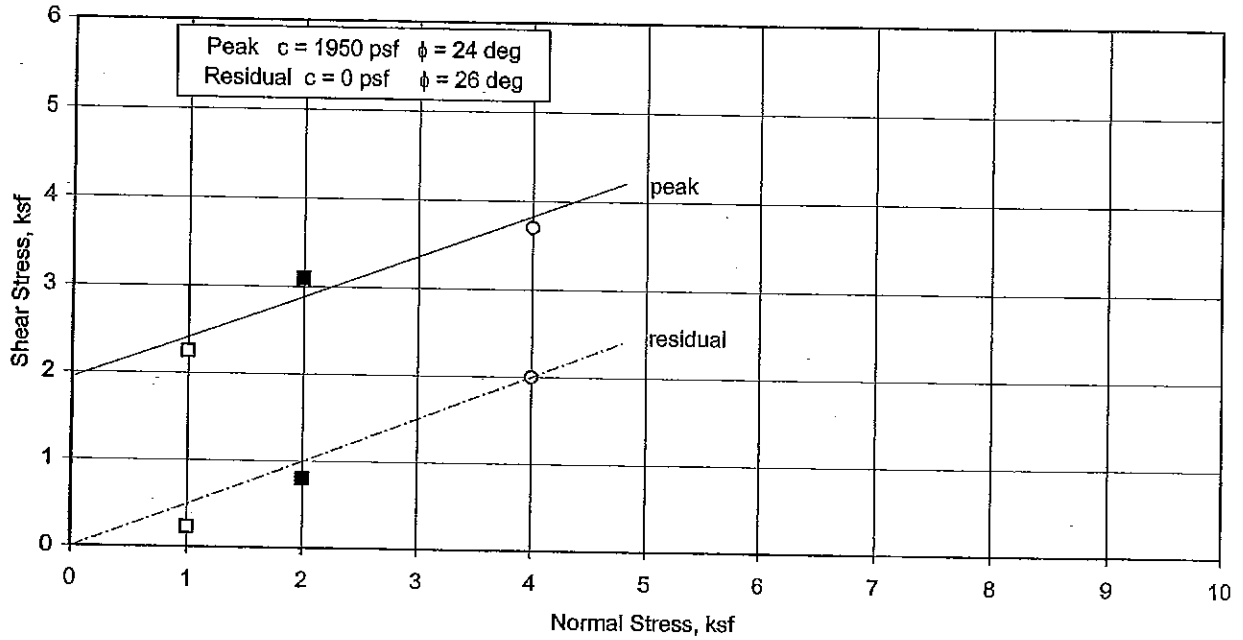
Project No. 1020320 CONSOLIDATION TEST RESULTS Figure 31

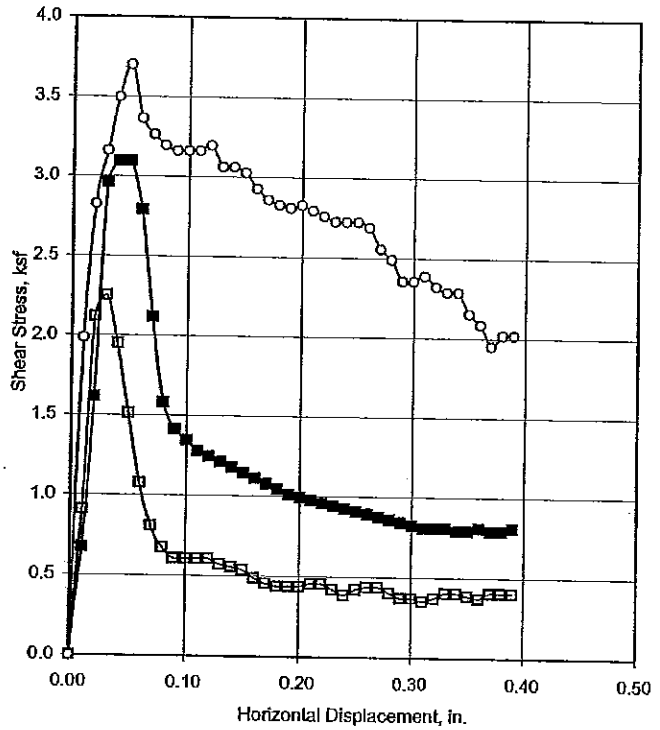
Applied Geotechnical Engineering Consultants, Inc.



Project No. 1020320 CONSOLIDATION TEST RESULTS Figure 32

Applied Geotechnical Engineering Consultants, Inc.





Test No. (Symbol)	1(□)	2(■)	3(○)
Sample Type	Remolded		
Length, in.	1.00	1.00	1.00
Diameter, in.	1.93	1.93	1.93
Dry Density, pcf	N/A	N/A	N/A
Moisture Content, %	N/A	N/A	N/A
Consolidation Load, ksf	1.0	2.0	4.0
Normal Load, ksf	1.0	2.0	4.0
Shear Stress, ksf (peak)	2.25	3.09	3.70
Shear Stress, ksf (residual)	0.23	0.80	2.00
Remarks	Strain Rate 0.05 in/min.		
	Sample remolded to approximately 92 pcf		
	at 30% moisture.		

Sample Index Properties	
Dry Density, pcf	N/A
Moisture Content, %	30
Liquid Limit, %	99
Plasticity Index, %	80
Percent Gravel	0
Percent Sand	17
Percent Passing No. 200 Sieve	83

Type of Test: Consolidated, Wetted

Sample Description: Tuffaceous Bedrock

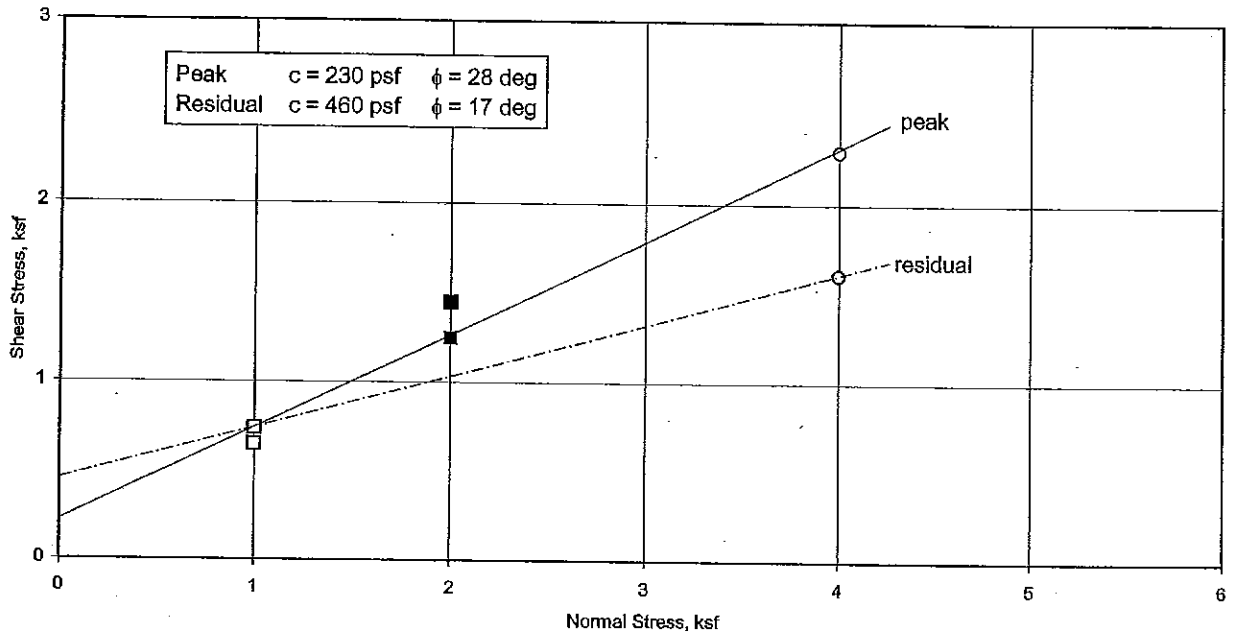
From: TP-8 @ 10'

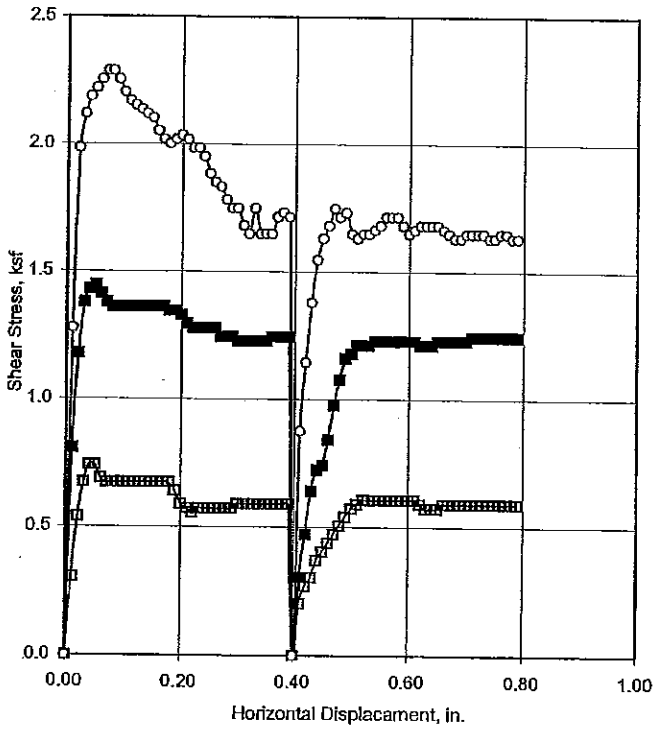
Project No. 1020320

Direct Shear Test Results

Figure 33

Applied Geotechnical Engineering Consultants, Inc.





Test No. (Symbol)	1(□)	2(■)	3(○)
Sample Type	Undisturbed		
Length, in.	0.75	0.75	0.75
Diameter, in.	1.93	1.93	1.93
Dry Density, pcf	N/A	N/A	N/A
Moisture Content, %	N/A	N/A	N/A
Consolidation Load, ksf	1.0	2.0	4.0
Normal Load, ksf	1.0	2.0	4.0
Shear Stress, ksf (peak)	0.74	1.45	2.29
Shear Stress, ksf (residual)	0.65	1.25	1.61
Remarks	Strain Rate 0.05 in/min.		

Sample Index Properties	
Dry Density, pcf	97
Moisture Content, %	14
Liquid Limit, %	31
Plasticity Index, %	14
Percent Gravel	0
Percent Sand	27
Percent Passing No. 200 Sieve	73

Type of Test: Consolidated, Wetted

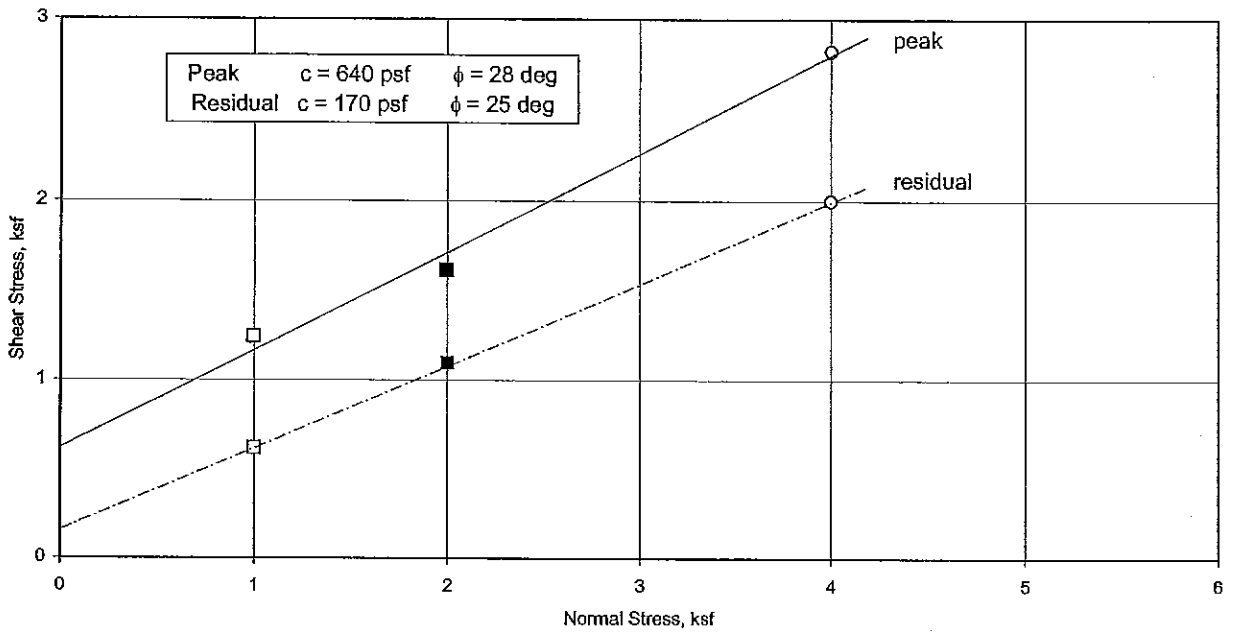
Sample Description: Lean Clay with Sand (CL) From: HB-1 "B"

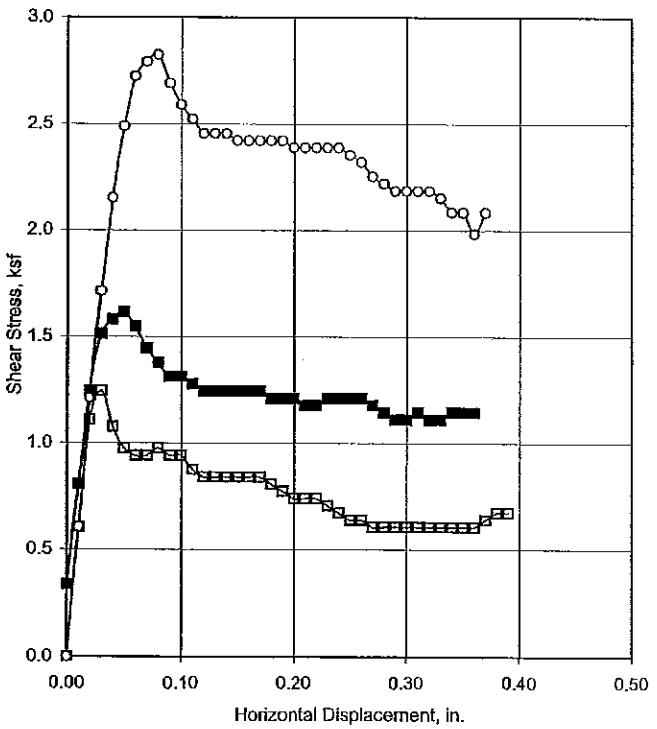
Project No. 1020320

Direct Shear Test Results

Figure 34

Applied Geotechnical Engineering Consultants, Inc.





Test No. (Symbol)	1(□)	2(■)	3(O)
Sample Type	Undisturbed		
Length, in.	1.00	1.00	1.00
Diameter, in.	1.93	1.93	1.93
Dry Density, pcf	N/A	N/A	N/A
Moisture Content, %	N/A	N/A	N/A
Consolidation Load, ksf	1.0	2.0	4.0
Normal Load, ksf	1.0	2.0	4.0
Shear Stress, ksf (peak)	1.24	1.61	2.82
Shear Stress, ksf (residual)	0.62	1.10	2.00
Remarks	Strain Rate 0.05 in/min.		

Sample Index Properties	
Dry Density, pcf	94
Moisture Content, %	26
Liquid Limit, %	31
Plasticity Index, %	8
Percent Gravel	0
Percent Sand	22
Percent Passing No. 200 Sieve	78

Type of Test: Consolidated, Wetted

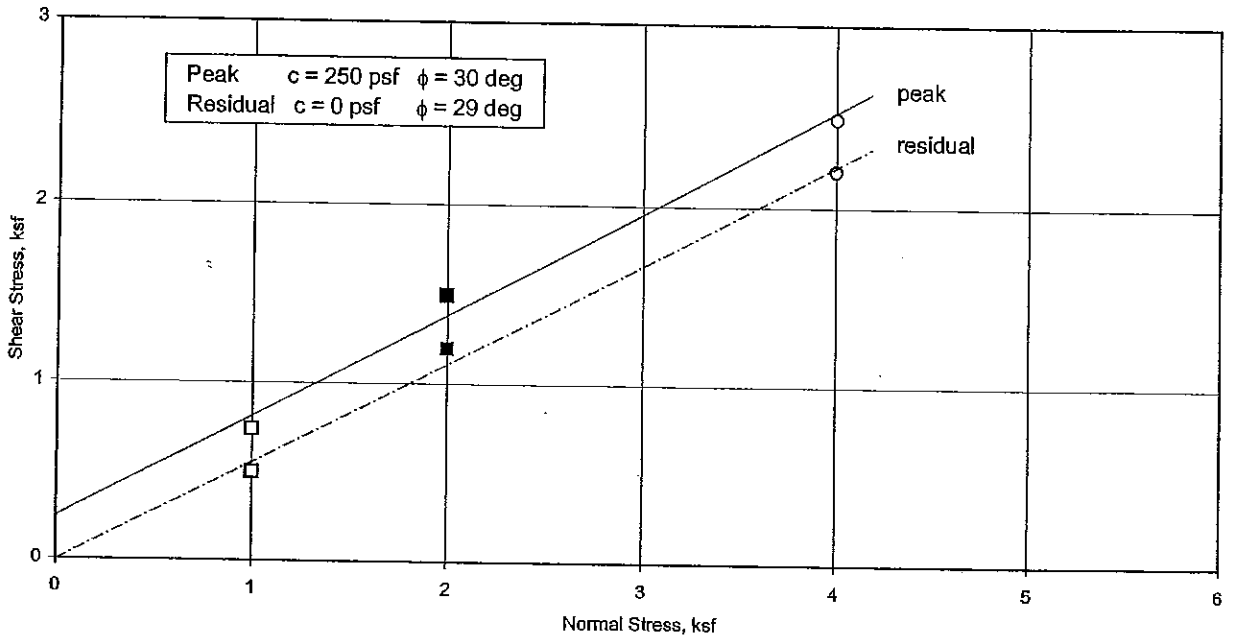
Sample Description: Silt with Sand (ML) From: HB-2 "A"

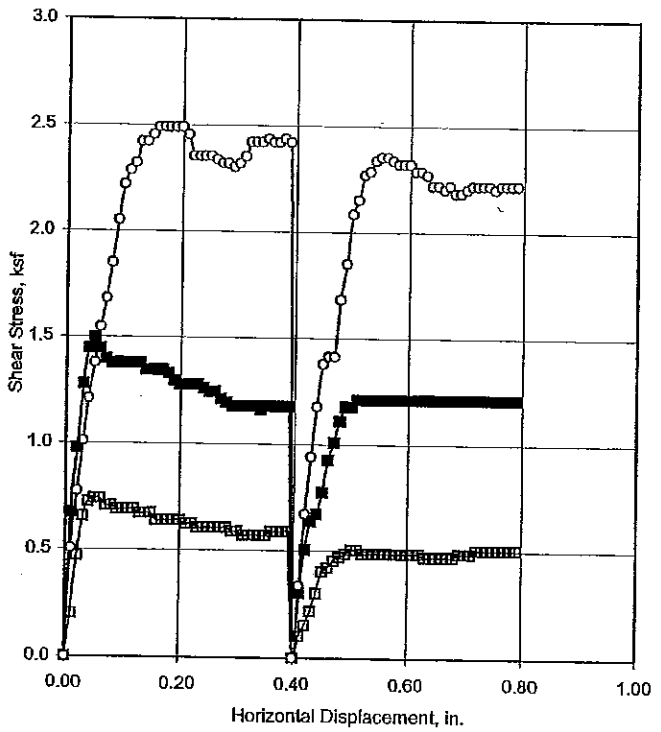
Project No. 1020320

Direct Shear Test Results

Figure 35

Applied Geotechnical Engineering Consultants, Inc.





Test No. (Symbol)	1(□)	2(■)	3(O)
Sample Type	Undisturbed		
Length, in.	0.75	0.75	0.75
Diameter, in.	1.93	1.93	1.93
Dry Density, pcf	N/A	N/A	N/A
Moisture Content, %	N/A	N/A	N/A
Consolidation Load, ksf	1.0	2.0	4.0
Normal Load, ksf	1.0	2.0	4.0
Shear Stress, ksf (peak)	0.74	1.50	2.49
Shear Stress, ksf (residual)	0.50	1.20	2.20
Remarks	Strain Rate 0.05 in/min.		

Sample Index Properties	
Dry Density, pcf	93
Moisture Content, %	26
Liquid Limit, %	31
Plasticity Index, %	16
Percent Gravel	0
Percent Sand	20
Percent Passing No. 200 Sieve	80

Type of Test: Consolidated, Wetted

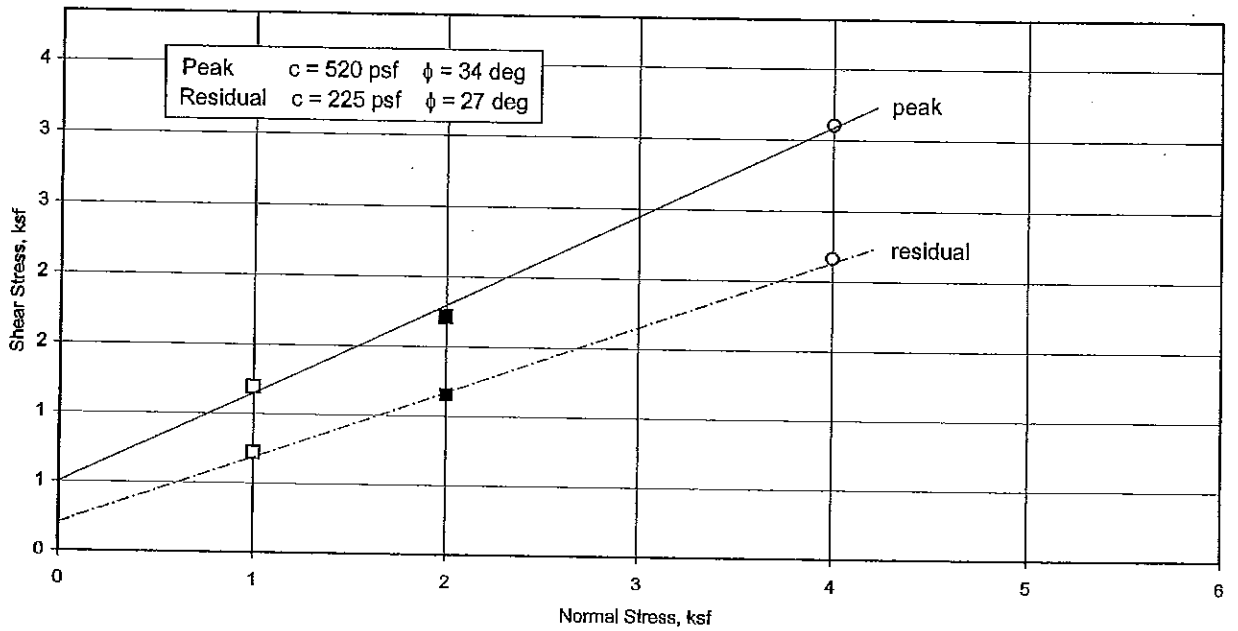
Sample Description: Lean Clay with Sand (CL) From: HB-2 "B"

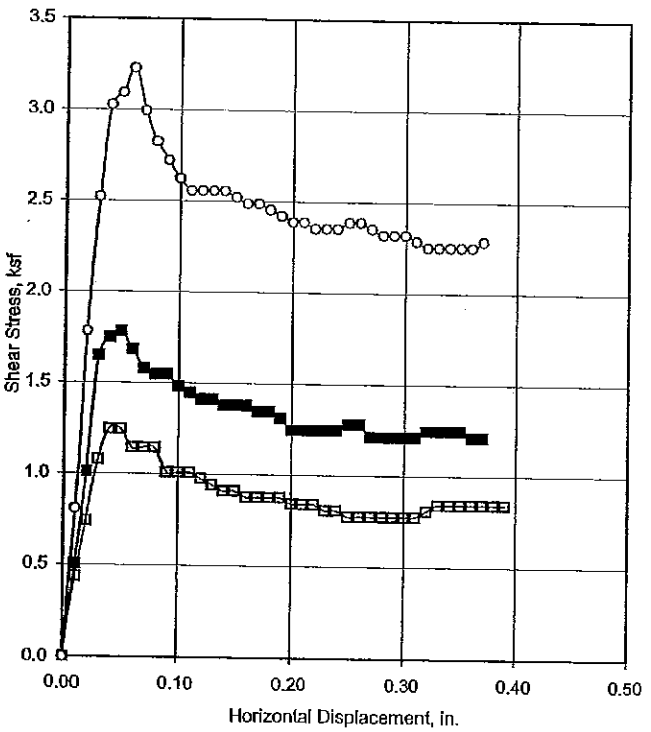
Project No. 1020320

Direct Shear Test Results

Figure 36

Applied Geotechnical Engineering Consultants, Inc.





Test No. (Symbol)	1(□)	2(■)	3(O)
Sample Type	Undisturbed		
Length, in.	1.00	1.00	1.00
Diameter, in.	1.93	1.93	1.93
Dry Density, pcf	N/A	N/A	N/A
Moisture Content, %	N/A	N/A	N/A
Consolidation Load, ksf	1.0	2.0	4.0
Normal Load, ksf	1.0	2.0	4.0
Shear Stress, ksf (peak)	1.24	1.78	3.23
Shear Stress, ksf (residual)	0.75	1.20	2.25
Remarks	Strain Rate 0.05 in/min.		

Sample Index Properties	
Dry Density, pcf	88
Moisture Content, %	31
Liquid Limit, %	38
Plasticity Index, %	21
Percent Gravel	0
Percent Sand	14
Percent Passing No. 200 Sieve	86

Type of Test: Consolidated,Wetted

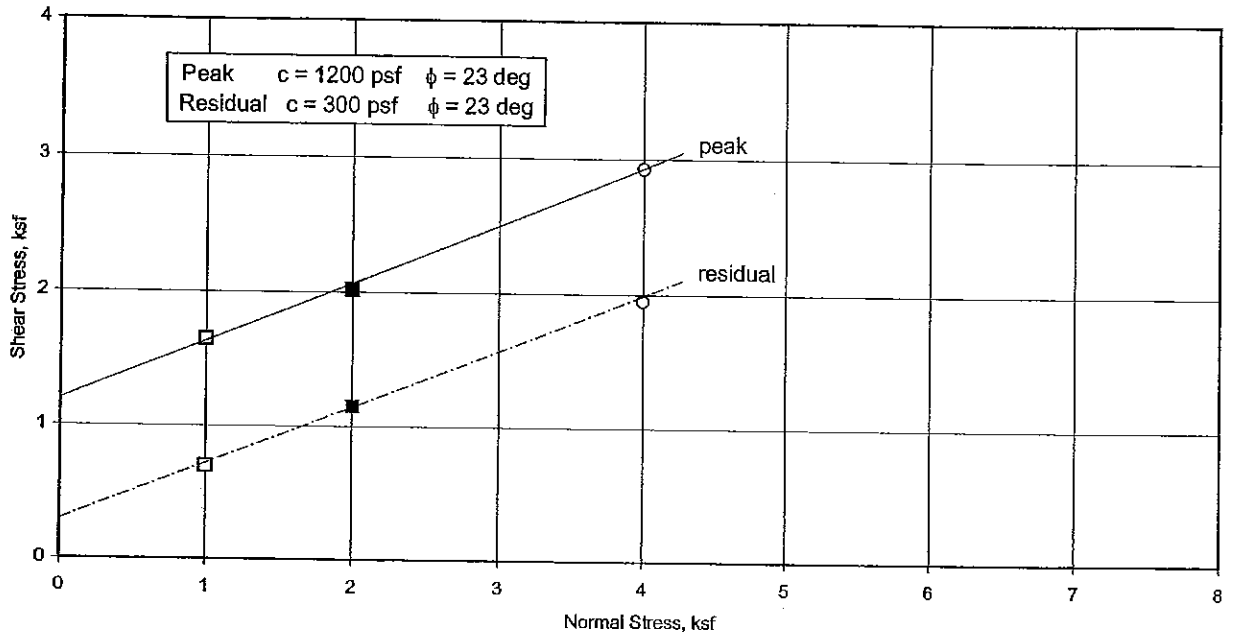
Sample Description: Lean Clay (CL) From: HB-3 "A"

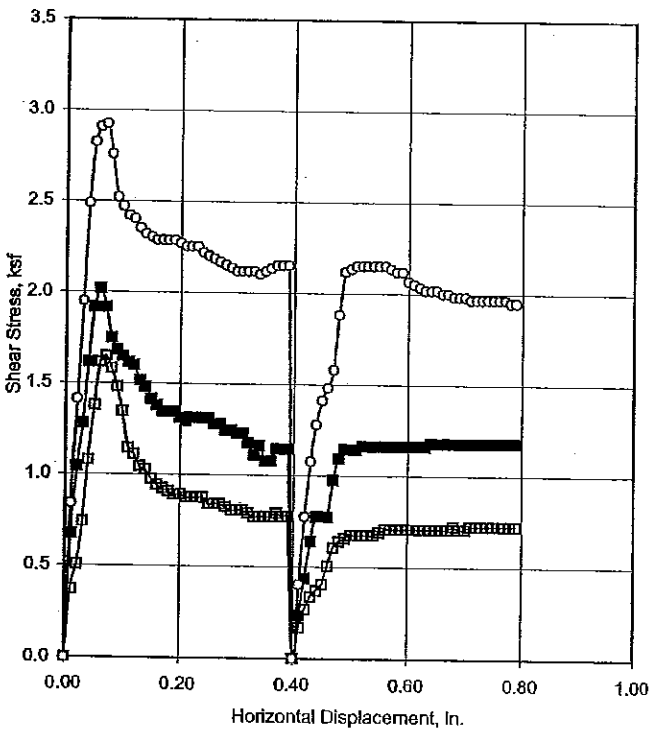
Project No. 1020320

Direct Shear Test Results

Figure 37

Applied Geotechnical Engineering Consultants, Inc.





Test No. (Symbol)	1(□)	2(■)	3(O)
Sample Type	Undisturbed		
Length, in.	0.75	0.75	0.75
Diameter, in.	1.93	1.93	1.93
Dry Density, pcf	N/A	N/A	N/A
Moisture Content, %	N/A	N/A	N/A
Consolidation Load, ksf	1.0	2.0	4.0
Normal Load, ksf	1.0	2.0	4.0
Shear Stress, ksf (peak)	1.65	2.02	2.93
Shear Stress, ksf (residual)	0.70	1.15	1.95
Remarks	Strain Rate 0.05 in/min.		

Sample Index Properties	
Dry Density, pcf	94
Moisture Content, %	27
Liquid Limit, %	38
Plasticity Index, %	23
Percent Gravel	-
Percent Sand	-
Percent Passing No. 200 Sieve	81

Type of Test: Consolidated, Wetted

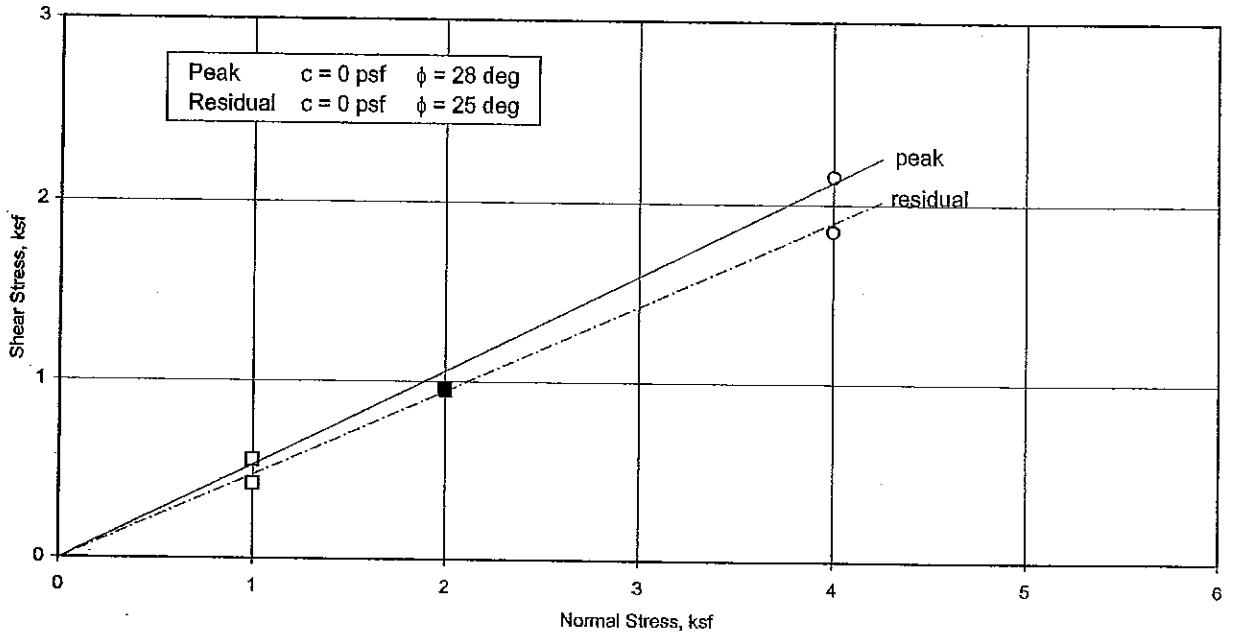
Sample Description: Lean Clay with Sand (CL) From: HB-3 "B"

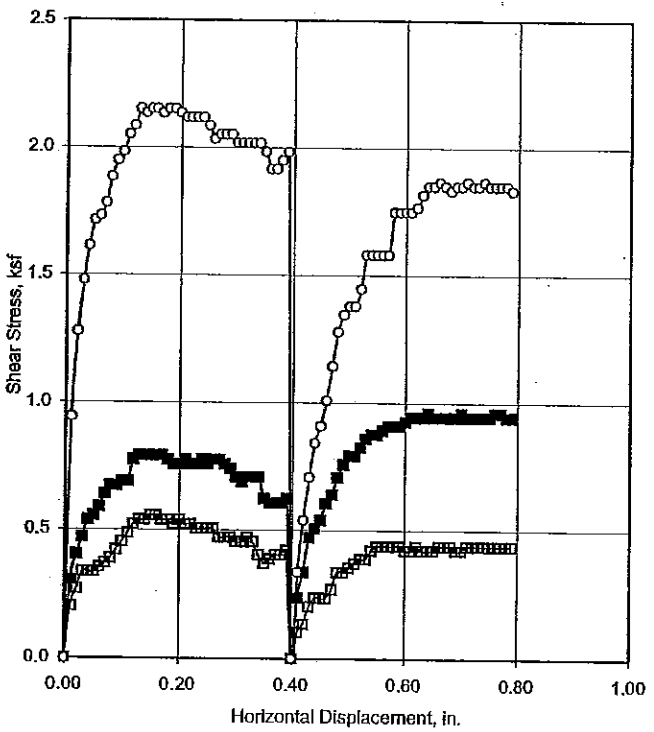
Project No. 1020320

Direct Shear Test Results

Figure 38

Applied Geotechnical Engineering Consultants, Inc.





Test No. (Symbol)	1(□)	2(■)	3(○)
Sample Type	Remolded		
Length, in.	1.00	1.00	1.00
Diameter, in.	1.93	1.93	1.93
Dry Density, pcf	N/A	N/A	N/A
Moisture Content, %	N/A	N/A	N/A
Consolidation Load, ksf	1.0	2.0	4.0
Normal Load, ksf	1.0	2.0	4.0
Shear Stress, ksf (peak)	0.55	0.96	2.15
Shear Stress, ksf (residual)	0.42	0.95	1.85
Remarks	Strain Rate 0.05 in/min.		

Sample Index Properties	
Dry Density, pcf	87
Moisture Content, %	33
Liquid Limit, %	43
Plasticity Index, %	28
Percent Gravel	-
Percent Sand	-
Percent Passing No. 200 Sieve	80

Type of Test: Consolidated, Wetted

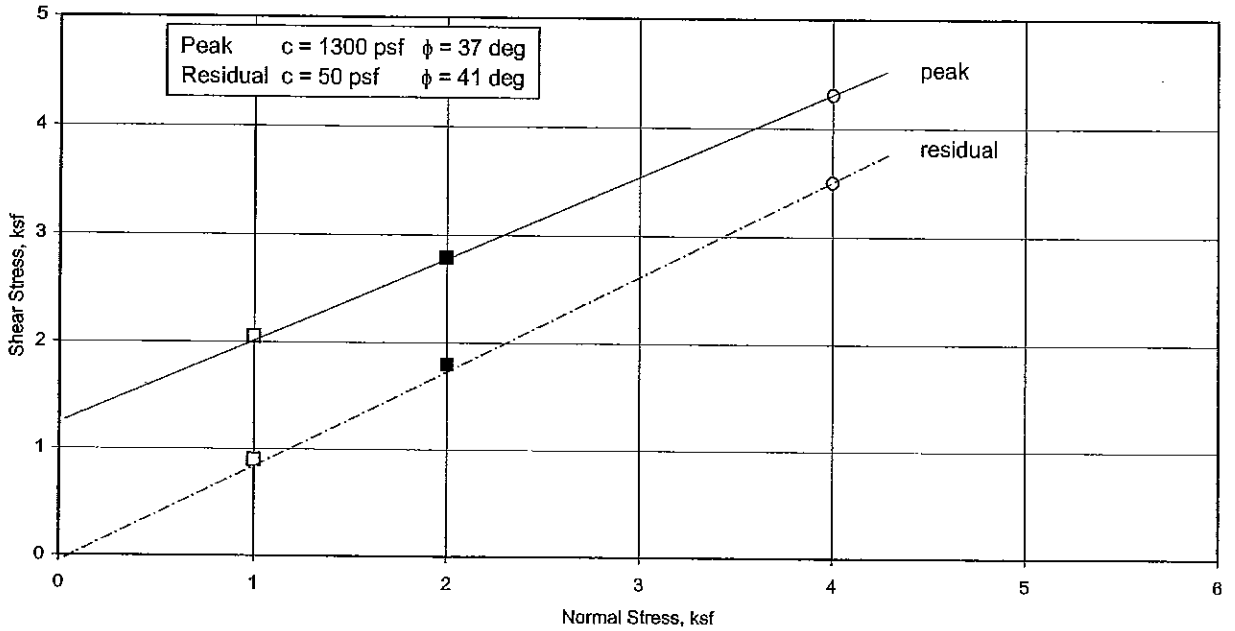
Sample Description: Lean Clay with Sand (CL) From: HB-3 "C"

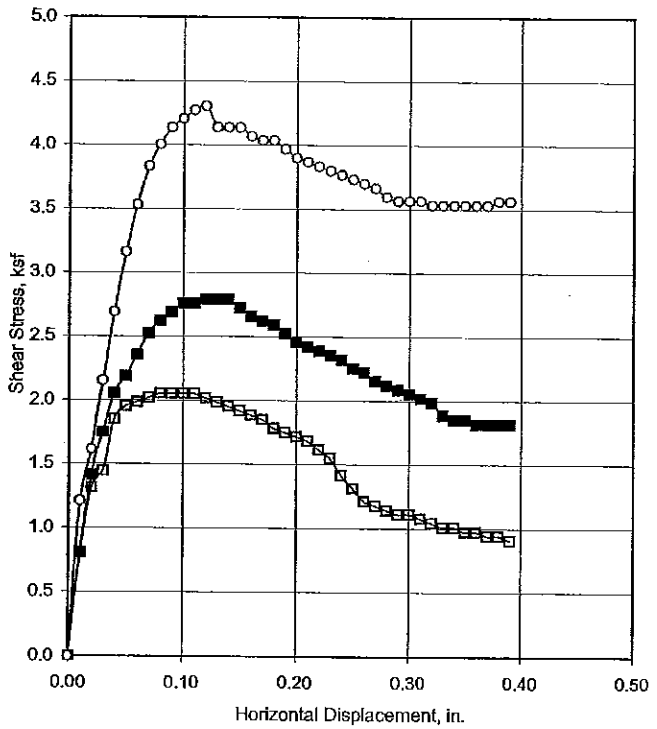
Project No. 1020320

Direct Shear Test Results

Figure 39

Applied Geotechnical Engineering Consultants, Inc.





Test No. (Symbol)	1(□)	2(■)	3(O)
Sample Type	Undisturbed		
Length, in.	1.00	1.00	1.00
Diameter, in.	1.93	1.93	1.93
Dry Density, pcf	N/A	N/A	N/A
Moisture Content, %	N/A	N/A	N/A
Consolidation Load, ksf	1.0	2.0	4.0
Normal Load, ksf	1.0	2.0	4.0
Shear Stress, ksf (peak)	2.05	2.79	4.30
Shear Stress, ksf (residual)	0.90	1.80	3.50
Remarks	Strain Rate 0.05 in/min.		

Sample Index Properties	
Dry Density, pcf	122
Moisture Content, %	4
Liquid Limit, %	38
Plasticity Index, %	20
Percent Gravel	23
Percent Sand	11
Percent Passing No. 200 Sieve	66

Type of Test: Consolidated, Wetted

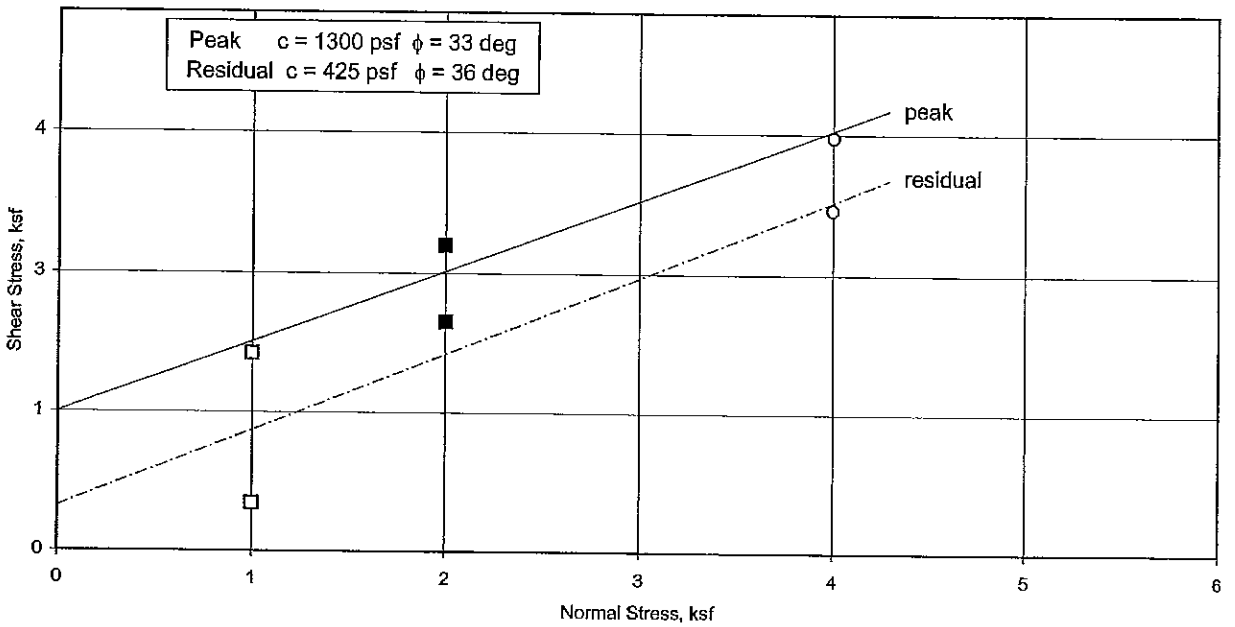
Sample Description: Lean Clay with Gravel (CL) From: HB-5 "A"

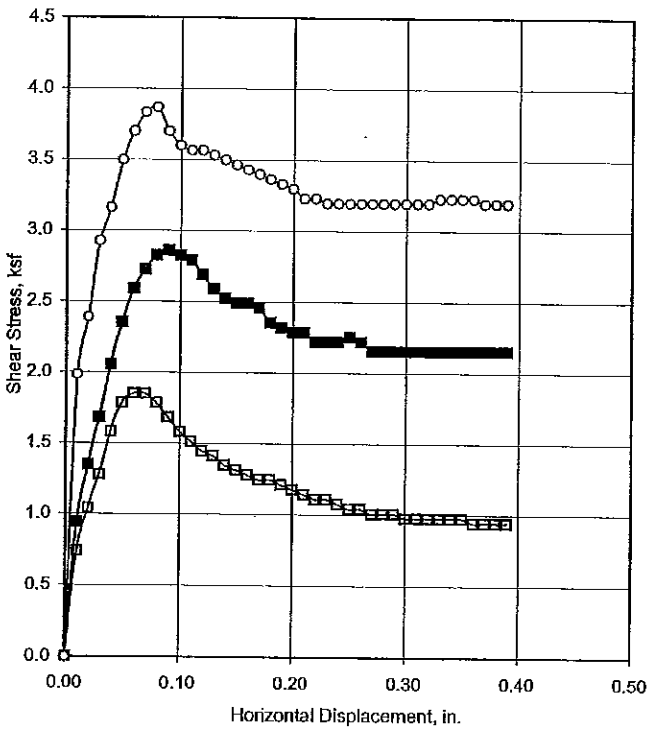
Project No. 1020320

Direct Shear Test Results

Figure 40

Applied Geotechnical Engineering Consultants, Inc.





Test No. (Symbol)	1(□)	2(■)	3(O)
Sample Type	Undisturbed		
Length, in.	1.00	1.00	1.00
Diameter, in.	1.93	1.93	1.93
Dry Density, pcf	N/A	N/A	N/A
Moisture Content, %	N/A	N/A	N/A
Consolidation Load, ksf	1.0	2.0	4.0
Normal Load, ksf	1.0	2.0	4.0
Shear Stress, ksf (peak)	1.85	2.86	3.87
Shear Stress, ksf (residual)	0.45	2.15	3.20
Remarks	Strain Rate 0.05 in/min.		

Sample Index Properties	
Dry Density, pcf	120
Moisture Content, %	5
Liquid Limit, %	45
Plasticity Index, %	31
Percent Gravel	9
Percent Sand	37
Percent Passing No. 200 Sieve	54

Type of Test: Consolidated, Wetted

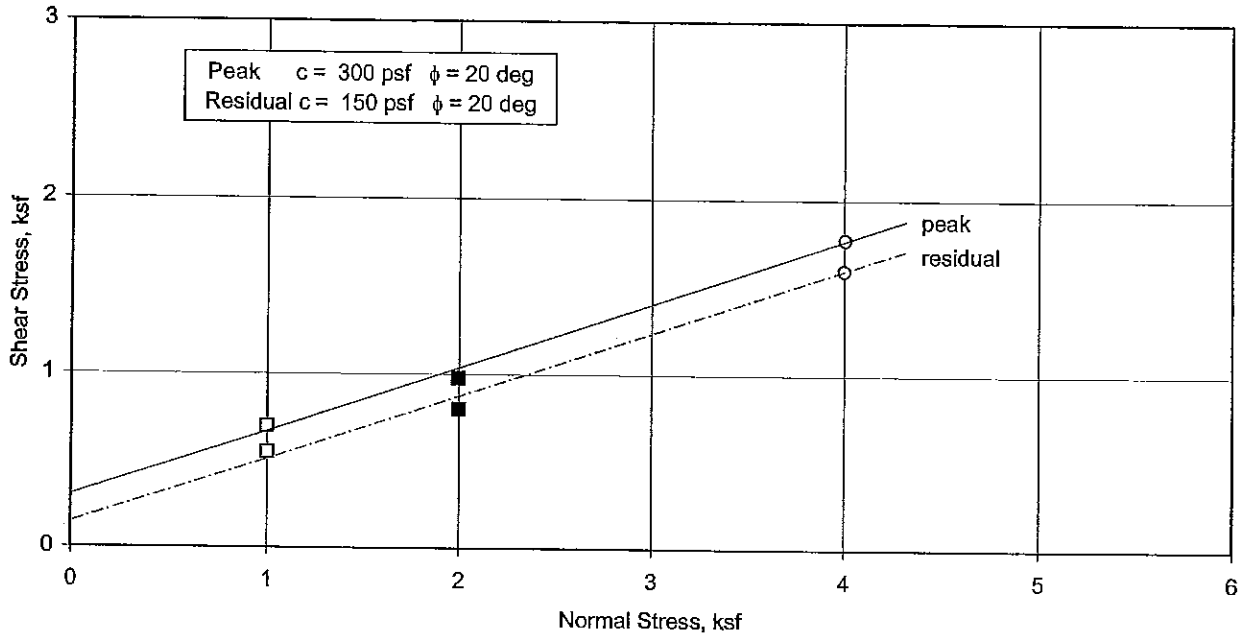
Sample Description: Sandy Lean Clay (CL) From: HB-5 "B"

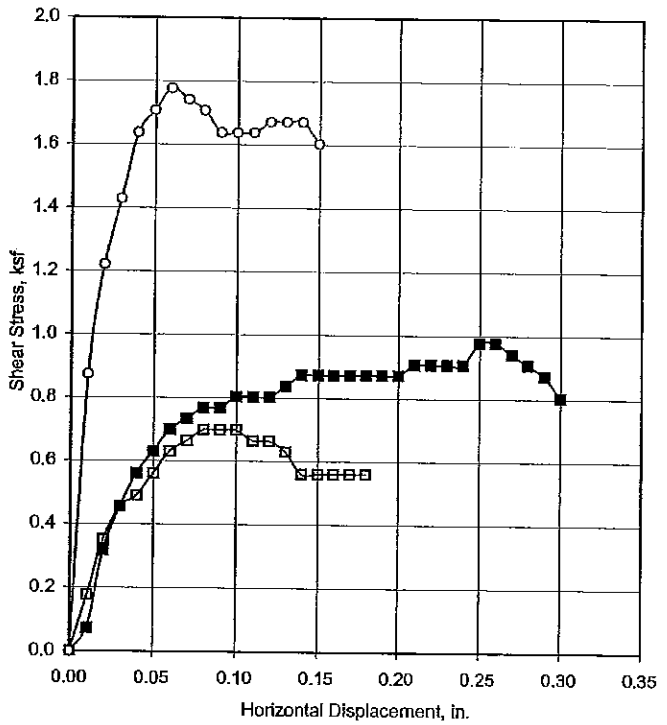
Project No. 1020320

Direct Shear Test Results

Figure 41

Applied Geotechnical Engineering Consultants, Inc.





Test No. (Symbol)	1(□)	2(■)	3(○)
Sample Type	Undisturbed		
Length, in.	1.00	1.00	0.75
Diameter, in.	1.93	1.93	1.93
Dry Density, pcf	N/A	N/A	N/A
Moisture Content, %	N/A	N/A	N/A
Consolidation Load, ksf	1.0	2.0	4.0
Normal Load, ksf	1.0	2.0	4.0
Shear Stress, ksf (peak)	0.70	0.98	1.78
Shear Stress, ksf (residual)	0.55	0.80	1.60
Remarks	Strain Rate 0.05 in/min.		

Sample Index Properties	
Dry Density, pcf	N/A
Moisture Content, %	N/A
Liquid Limit, %	25
Plasticity Index, %	9
Percent Gravel	0
Percent Sand	15
Percent Passing No. 200 Sieve	85

Type of Test: Consolidated Saturated

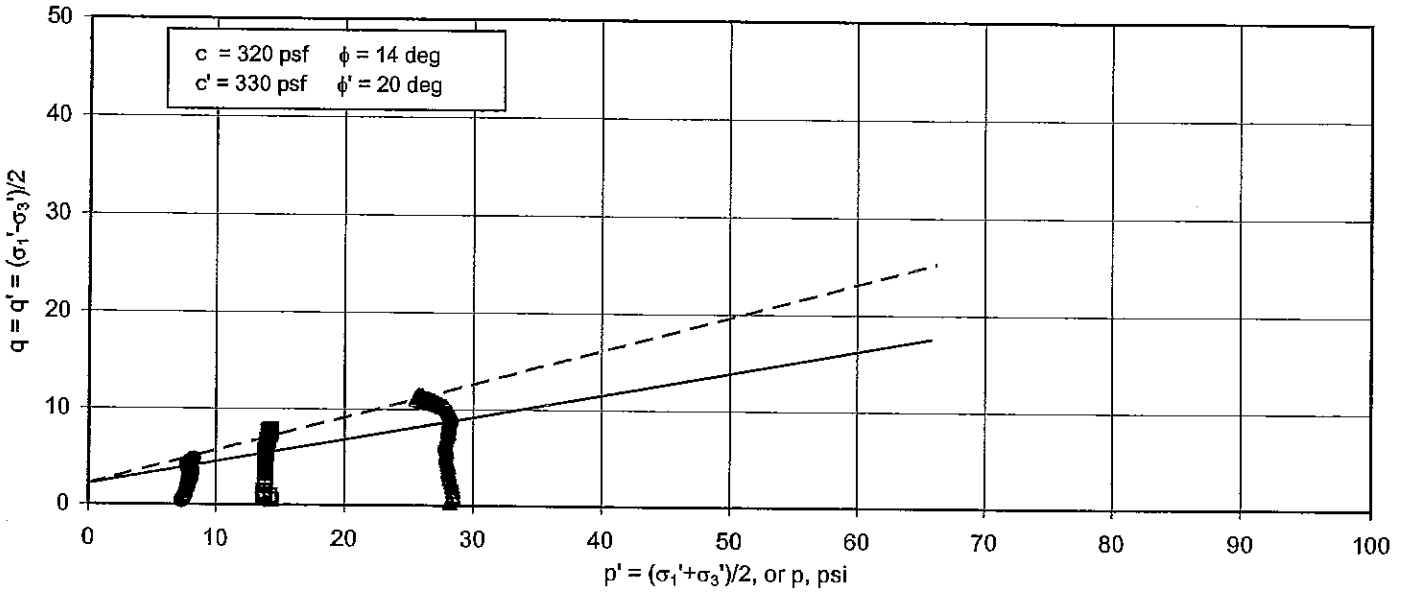
Sample Description: Lean Clay with Sand (CL) From: HB-5 "E"

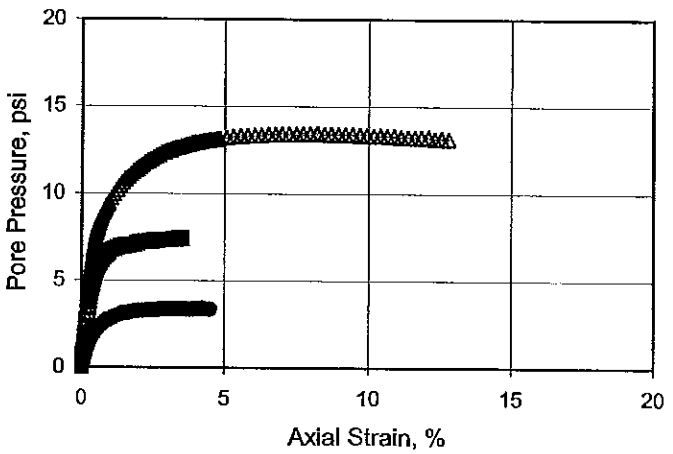
Project No. 1020320

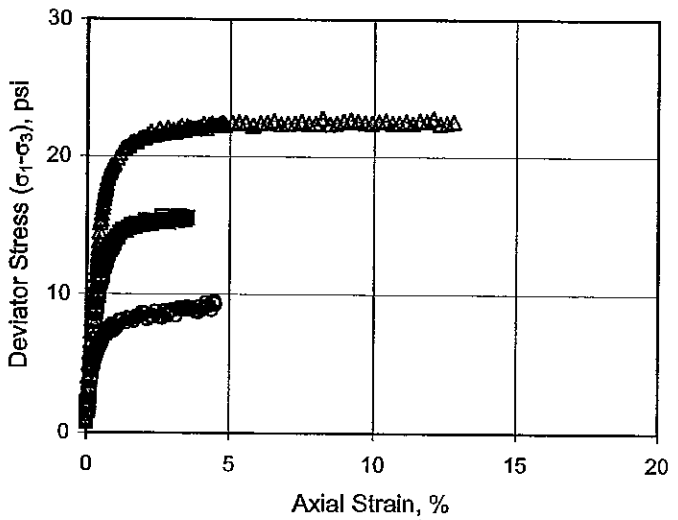
Direct Shear Test Results

Figure 42

Applied Geotechnical Engineering Consultants, Inc.







Test No. (Symbol)	○	□	△
Sample Type	Remolded		
Length, in.	3.99	3.83	3.71
Diameter, in.	1.92	1.76	1.64
Dry Density, pcf	85	N/A	N/A
Moisture Content, %	30	N/A	N/A
Consolidation Pressure, psi	6.9	13.9	27.8
"B" Parameter	0.98	0.98	0.98
Total Confining Stress (σ_3), psi	6.9	13.8	27.8
Total Axial Stress (σ_1), psi	16.0	29.3	49.9
Deviator Stress (σ_1-σ_3), psi	9.0	15.5	22.1
Effective Lateral Stress (σ_3'), psi	3.6	6.4	15.1
Effective Axial Stress (σ_1'), psi	12.6	21.9	37.3
Pore Pressure (μ), psi	3.4	7.4	12.7
Strain, %	3.5	3.5	3.5
Remarks			

Sample Index Properties	
Natural Dry Density, pcf	-
Natural Moisture Content, %	-
Liquid Limit, %	68
Plasticity Index, %	53
Percent Gravel	0
Percent Sand	26
Percent Passing No. 200 Sieve	74

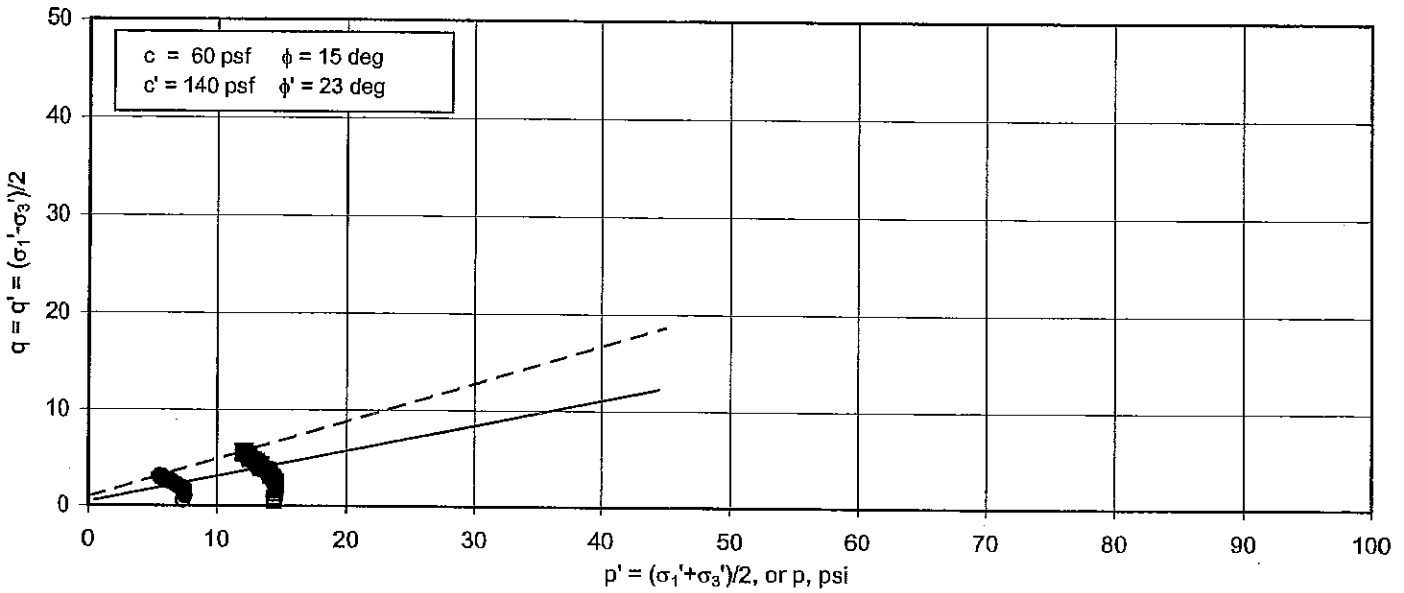
Sample Description Fat Clay with Sand (CH) Sample Location HB-5 "C"

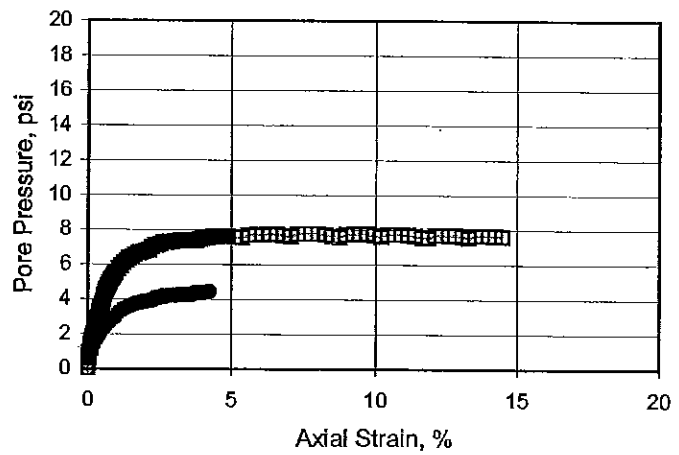
Project No. 1020320

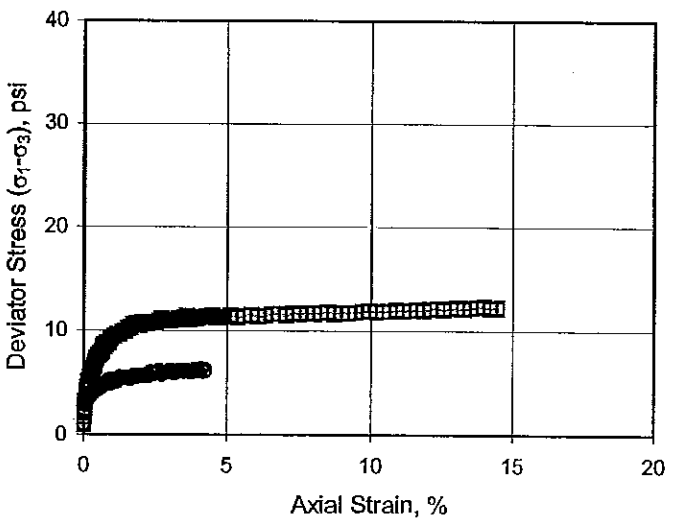
Triaxial Compression Test

Figure 43

Applied Geotechnical Engineering Consultants, Inc.







Test No. (Symbol)	○	□	
Sample Type	remolded		
Length, in.	3.97	3.82	
Diameter, in.	1.9	1.75	
Dry Density, pcf	80	N/A	
Moisture Content, %	30	N/A	
Consolidation Pressure, psi	6.9	13.9	
"B" Parameter	0.95	0.95	
Total Confining Stress (σ_3), psi	6.9	13.9	
Total Axial Stress (σ_1), psi	13.1	25.2	
Deviator Stress (σ_1-σ_3), psi	6.2	11.3	
Effective Lateral Stress (σ_3'), psi	2.5	6.3	
Effective Axial Stress (σ_1'), psi	8.7	17.7	
Pore Pressure (μ), psi	4.4	7.6	
Strain, %	4.3	4.3	
Remarks			

Sample Index Properties	
Natural Dry Density, pcf	
Natural Moisture Content, %	
Liquid Limit, %	46
Plasticity Index, %	33
Percent Gravel	5
Percent Sand	29
Percent Passing No. 200 Sieve	66

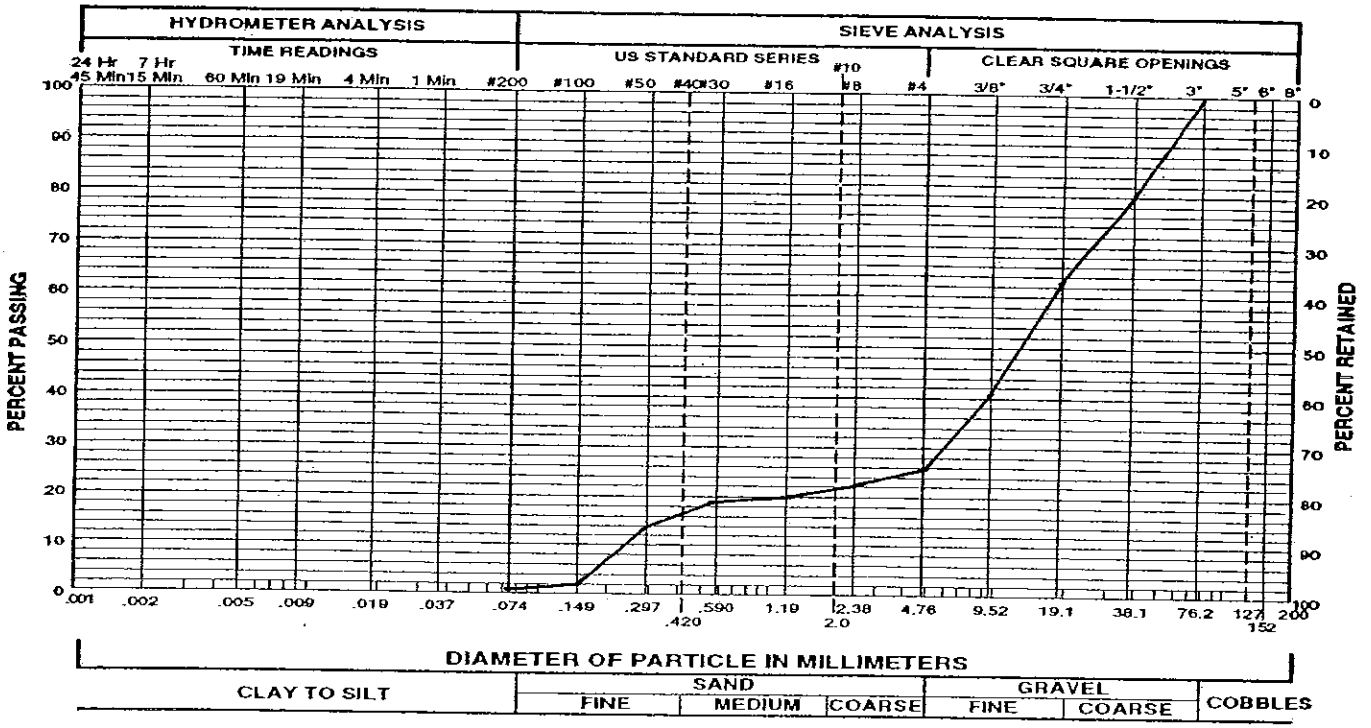
Sample Description Sandy Lean Clay (CL) Sample Location HB-5 "D"

Project No. 1020320

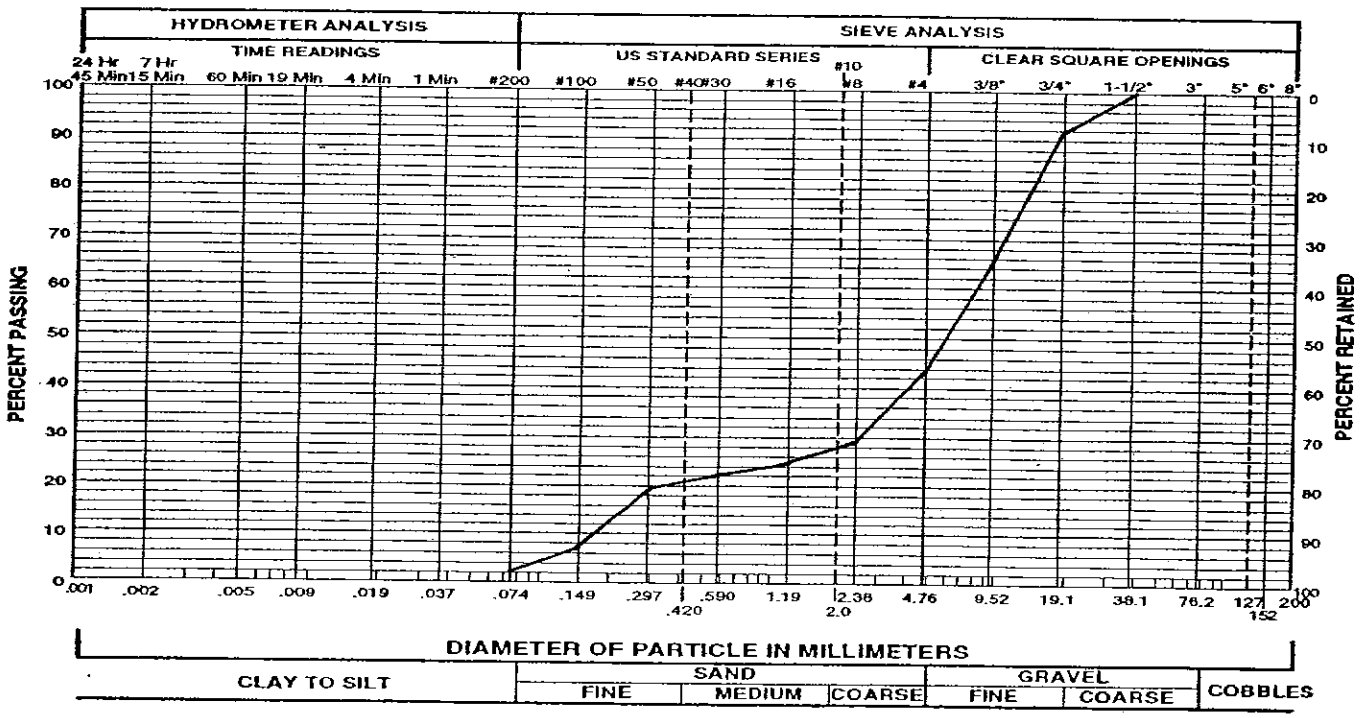
Triaxial Compression Test

Figure 44

Applied Geotechnical Engineering Consultants, Inc.



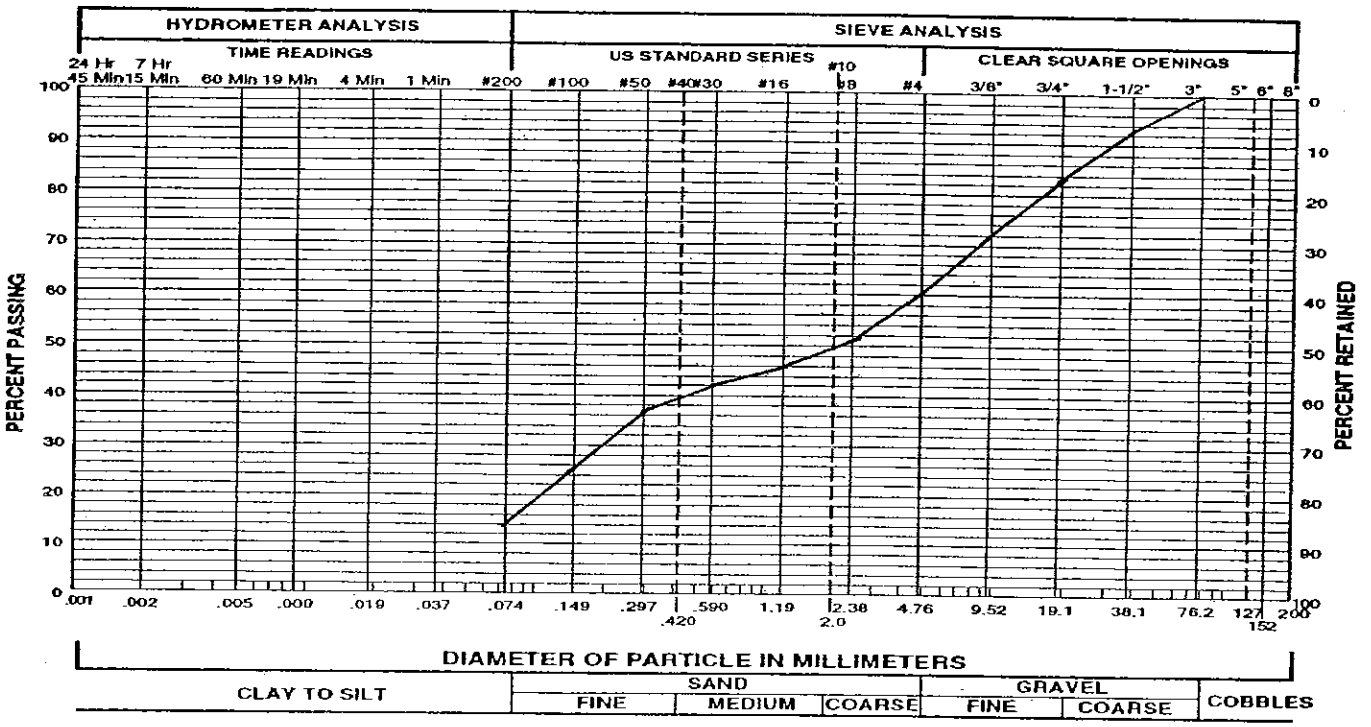
Gravel 74 % Sand 25 % Silt and Clay 1 %
Liquid Limit - % Plasticity Index - %
Sample of Poorly Graded Gravel with Sand (GP) From TP-5 @ 4 Feet



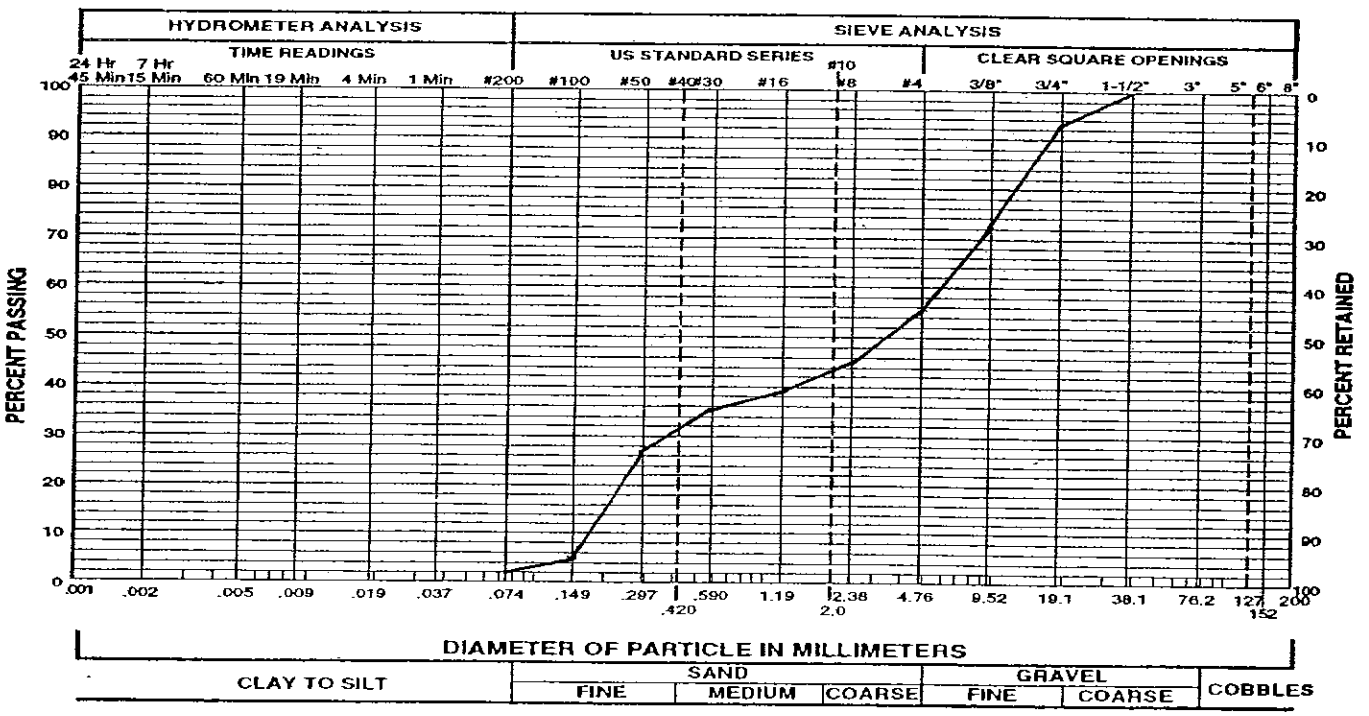
Gravel 57 % Sand 41 % Silt and Clay 2 %
Liquid Limit - % Plasticity Index - %
Sample of Poorly Graded Gravel with Sand (GP) From TP-7 @ 4 Feet

Project No. 1020320 **GRADATION TEST RESULTS** Figure 45

Applied Geotechnical Engineering Consultants, Inc.



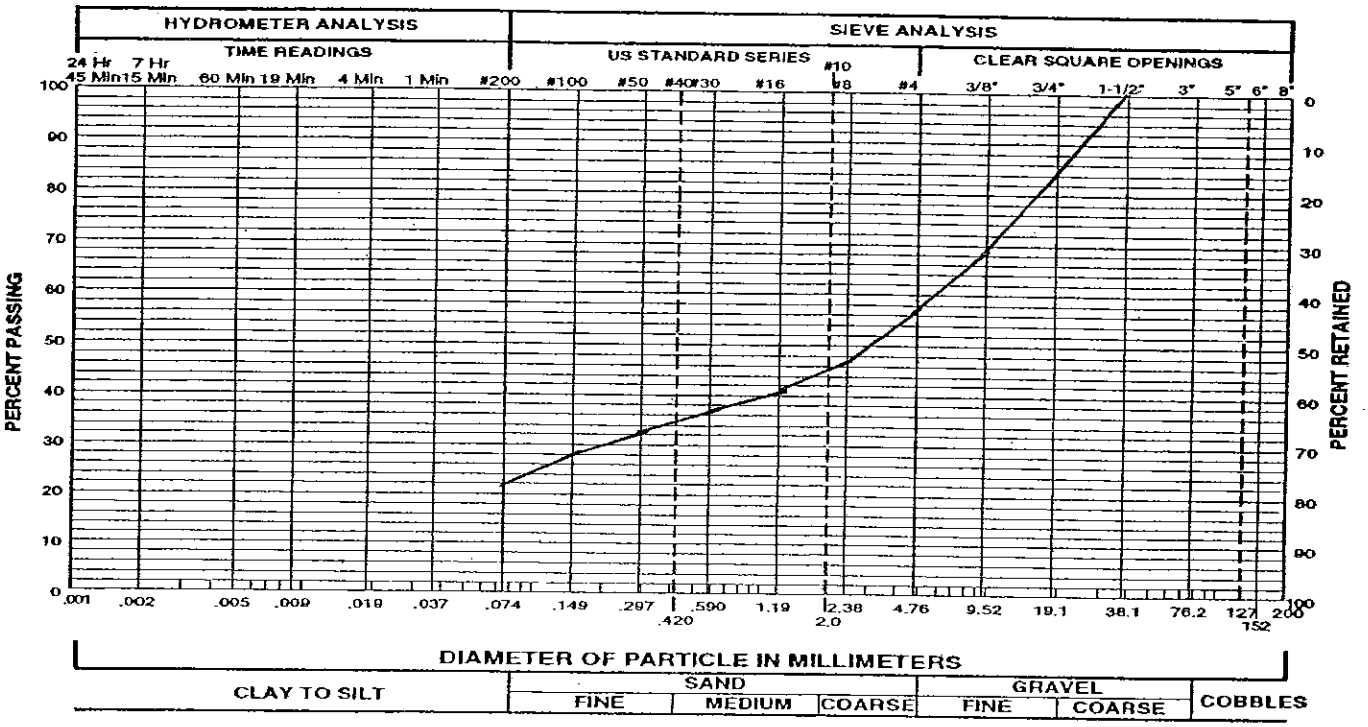
Gravel 40 % Sand 46 % Silt and Clay 14 %
Liquid Limit - % Plasticity Index - %
Sample of Fill; Silty Sand with Gravel From TP-13 @ 4 Feet



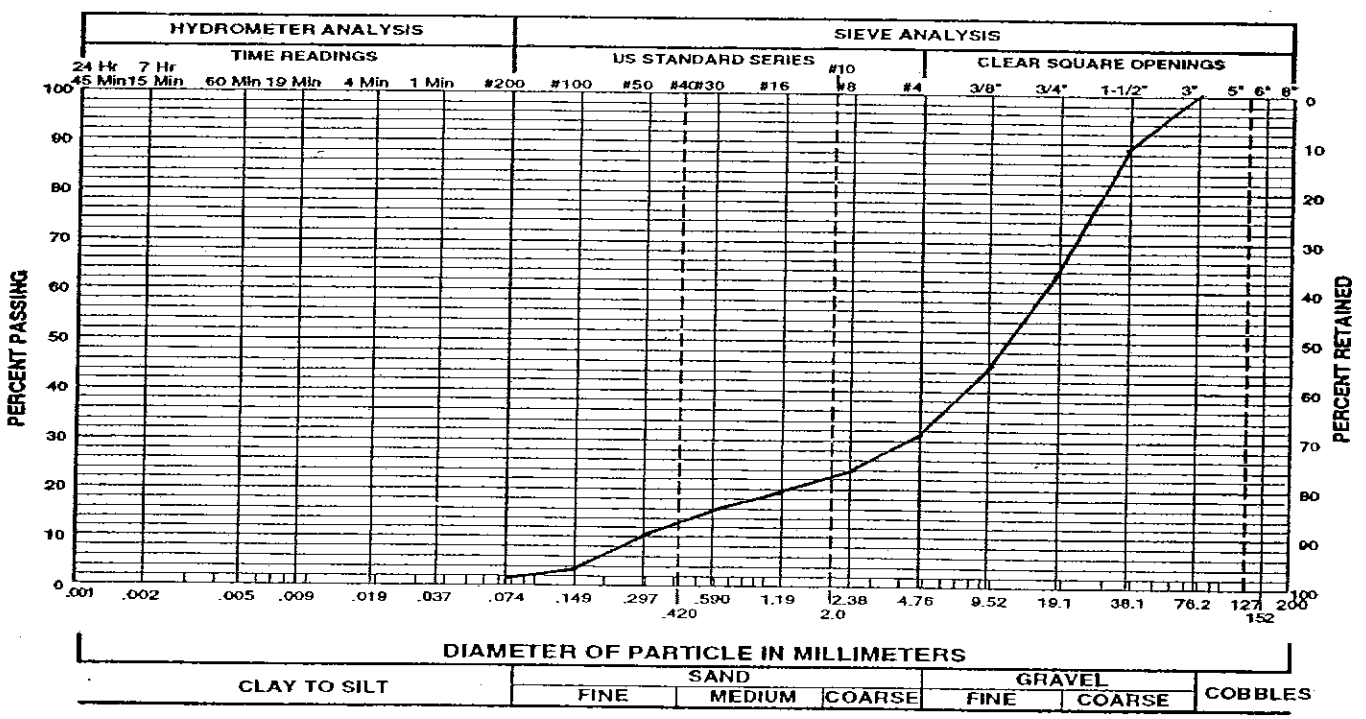
Gravel 45 % Sand 53 % Silt and Clay 2 %
Liquid Limit - % Plasticity Index - %
Sample of Poorly Graded Sand with Gravel (SP) From TP-14 @ 4 Feet

Project No. 1020320 **GRADATION TEST RESULTS** Figure 46

Applied Geotechnical Engineering Consultants, Inc.



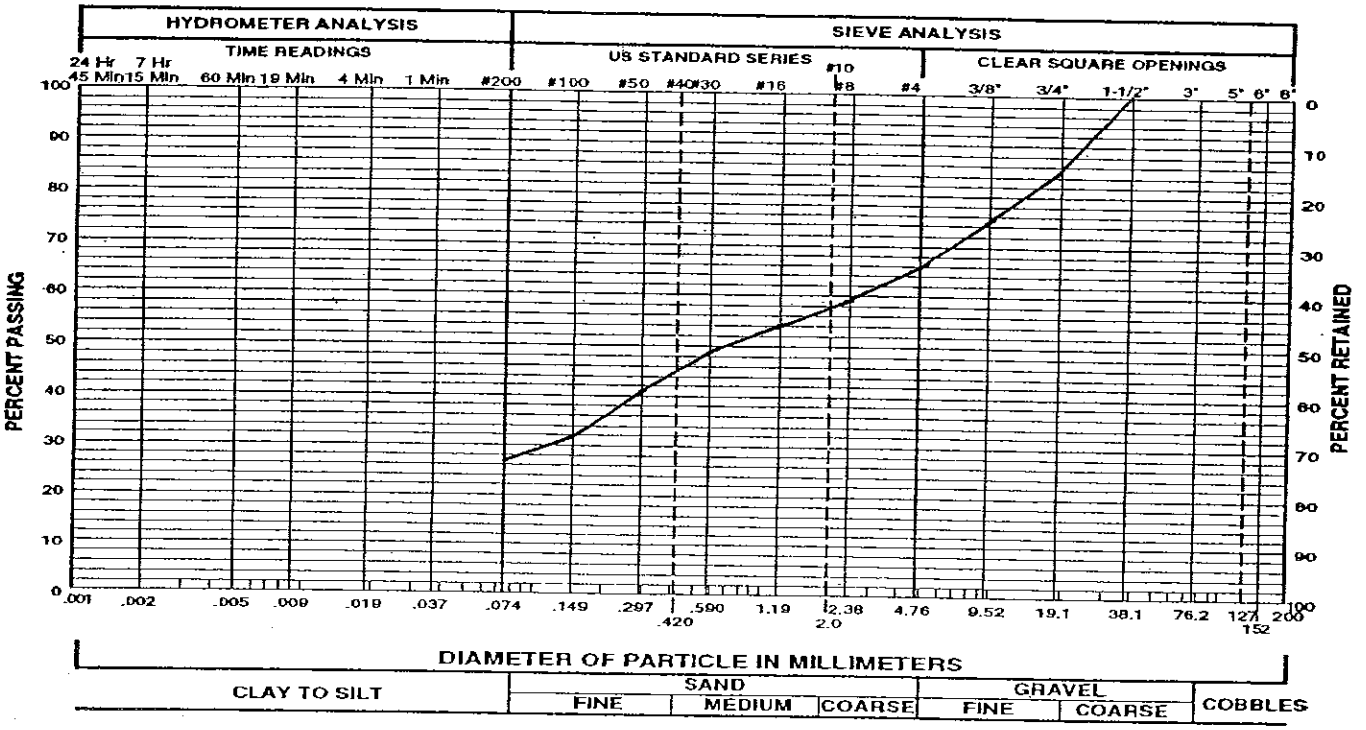
Gravel 44 % Sand 34 % Silt and Clay 22 %
Liquid Limit - % Plasticity Index - %
Sample of Fill; Clayey Gravel with Sand From TP-19 @ 4 Feet



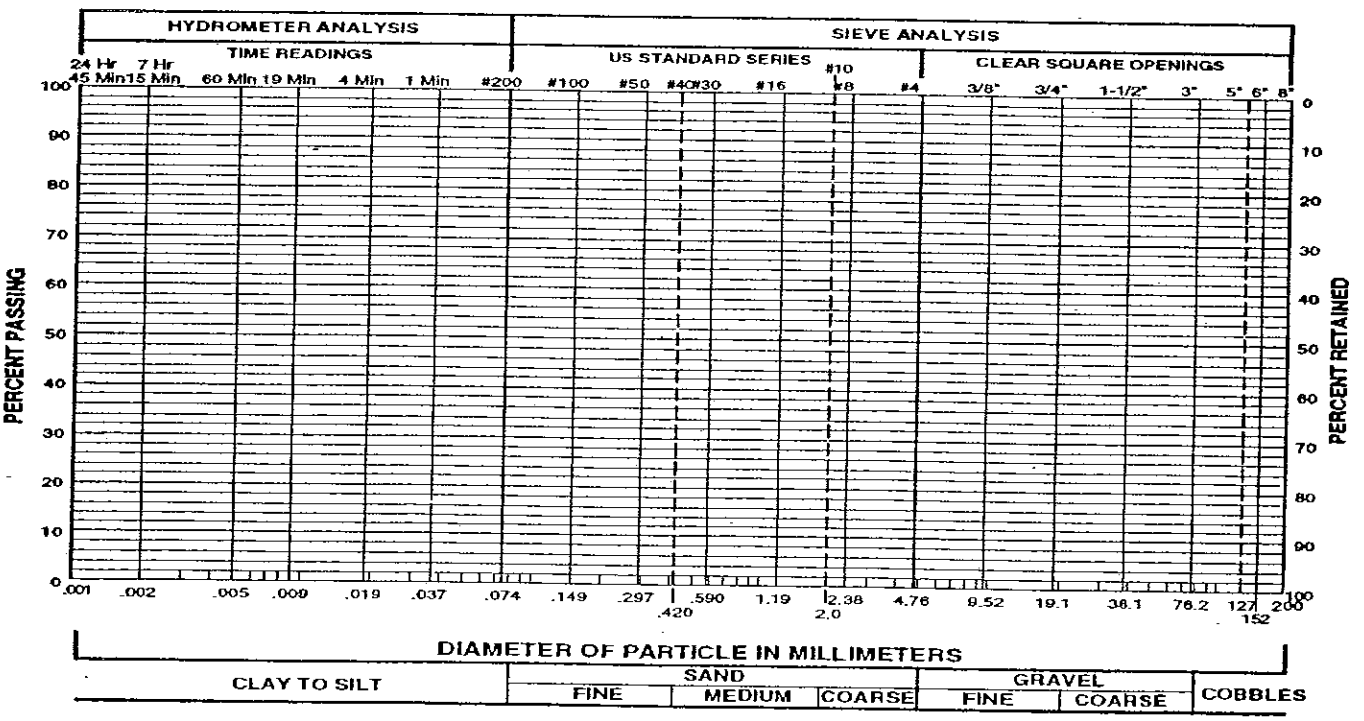
Gravel 69 % Sand 29 % Silt and Clay 2 %
Liquid Limit - % Plasticity Index - %
Sample of Poorly Graded Gravel with Sand (GP) From TP-35 @ 13½ Feet

Project No. 1020320 **GRADATION TEST RESULTS** Figure 47

Applied Geotechnical Engineering Consultants, Inc.

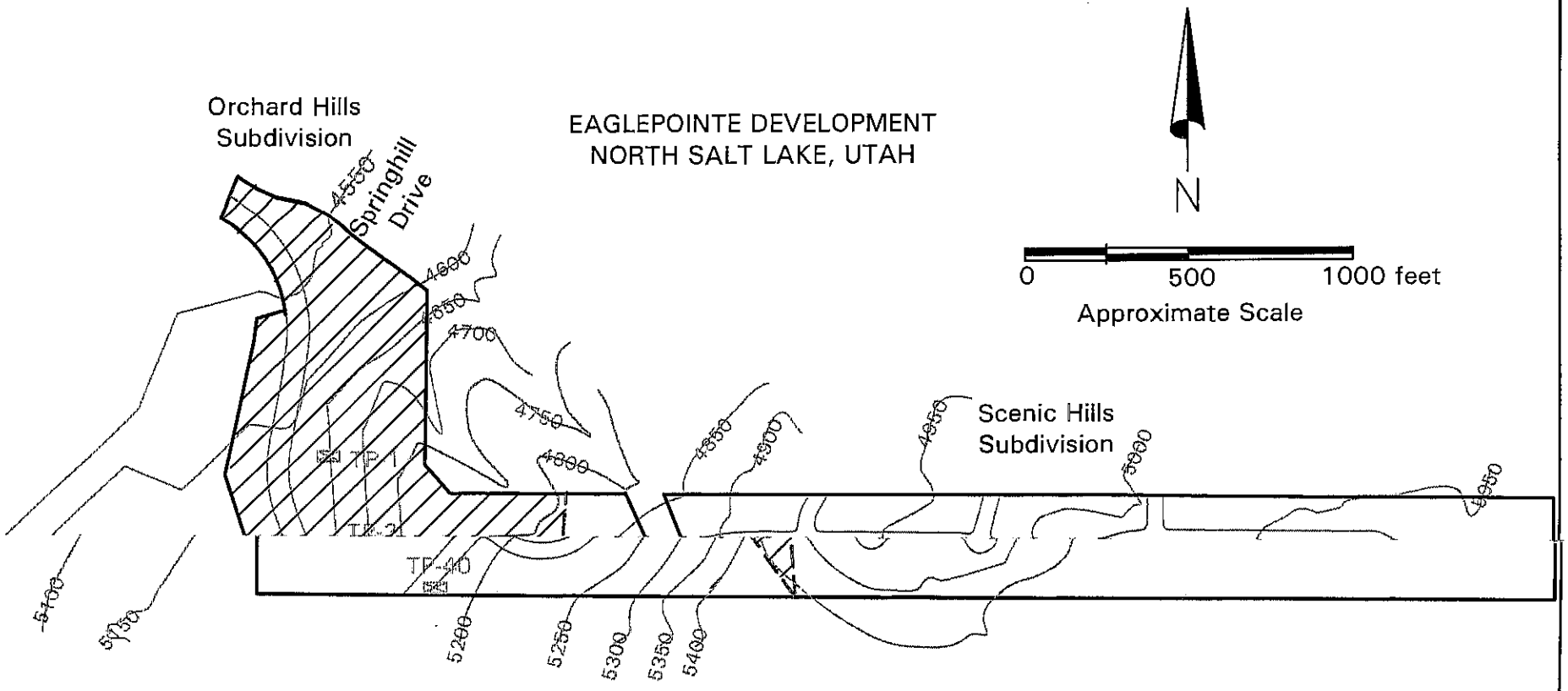


Gravel 34 % Sand 39 % Silt and Clay 27 %
Liquid Limit - % Plasticity Index - %
Sample of Clayey Sand with Gravel (SC) From TP-38 @ 4 Feet



Gravel_____ % Sand_____ % Silt and Clay_____ %
Liquid Limit_____ % Plasticity Index_____ %
Sample of_____ From _____

Project No. 1020320 **GRADATION TEST RESULTS** Figure 48



Legend:

- Trench
- Test Pit Location
- Boring Location
- Hand Excavation Location

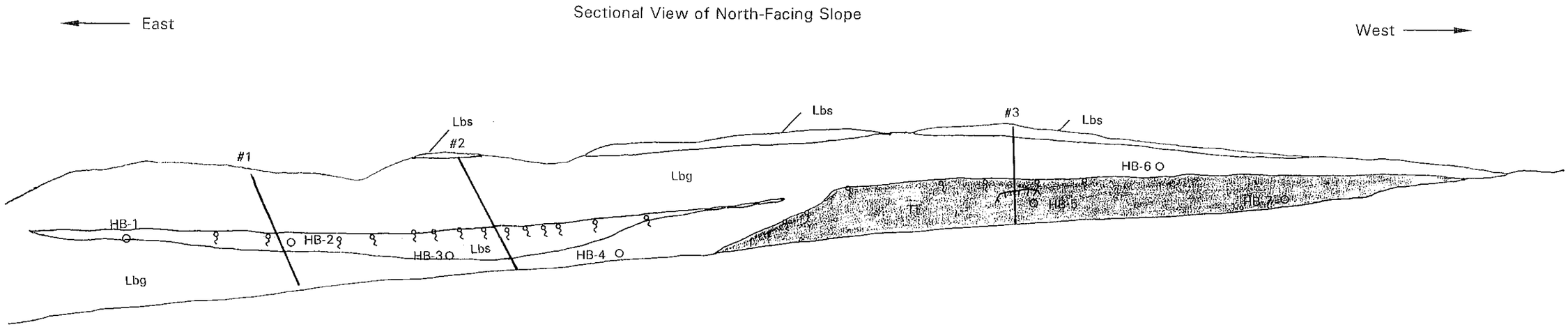
Indicates approximate areas of potentially moisture sensitive soil and/or bedrock.

1020320	AGEC	Approximate Areas of Potentially Moisture Sensitive Soil or Bedrock	Figure 49

APPENDIX A

SKETCH OF LARGE EXCAVATION SLOPE

Sectional View of North-Facing Slope



Legend:

- Tt - Tuffaceous deposits (Tertiary); primarily claystone and siltstone, slightly to highly weathered, firm to very hard, red with some gray to white areas.
- Lbg - Lake Bonneville gravels (uppermost Pleistocene); transgressive phase, poorly graded, well-rounded clasts, moderately well-sorted, gray-brown.
- Lbs - Lake Bonneville silt and clay (uppermost Pleistocene); fine-grained silt and clay, bedded, brown.
- Spring or seep
- Slope failure
- Hand excavation locations
- Profile locations

1020320	AGEC	Cut-Slope Profile	Appendix A

APPENDIX B

HYDROGEOLOGIC EVALUATION

HANSEN ALLEN & LUCE inc
ENGINEERS

Mr. Jim Nordquist
Applied Geotechnical Engineering Consultants
600 West Sandy Parkway
Salt Lake City, Utah 84070

June 8, 2001

RE: Eaglepointe Subdivision Hydrogeologic Review.

Dear Jim:

As requested we have reviewed the Eaglepointe information and data provided us. Specific efforts have been directed towards identifying local estimates and impacts of the ground water flow (volume) which have, or will be created by full development of the Eaglepointe project. In addition, we have reviewed and made estimates related to anticipated changes in the ground water table at Cross Section B-B' south of the Orchard Hills (Spring Hill Subdivision) area. Basic assumptions made during our analysis are defined on the attached calculation sheets.

It is not possible to obtain accurate estimates of infiltration rates, ground water flow velocities, and porosities without extensive engineering effort including significant field testing. As a result we are providing you herein our results based on the following basic geohydrologic and irrigation practices assumptions.

- Precipitation infiltration rates will vary between 5 and 15%.
- Irrigation application rates vary between 65 and 70 inches as per Jordan Valley Water Conservancy District (JVWCD) in a phone conversation with their horticulturalist on June 8th. During that conversation it was noted that a study which was completed by Utah State University for the northern Wasatch Front area showed irrigation application rates of up to 200 inches per year. JVWCD estimates average application rates between 65 and 70 inches per year. Calculations shown herein used estimates of 30, 48, and 70 inches.
- Worst case snowmelt conditions occur when February and March Snows melt with April warmer temperatures and rainfall. This case would provide for saturated near surface ground water conditions and increased ground water contributions.
- Snowmelt infiltration rates are estimated to be 50%.

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A summary of data calculations are provided in the following tables. The first table shows ground water flow volumes expected for undeveloped conditions as well as developed conditions with varying summer irrigation application rates. The table further identifies those volumes expected for the entire development versus those for the area tributary to Orchard Hills. The second table identifies anticipated changes to the ground water table resulting from development under two conditions. First, for summer conditions with varying irrigation application rates, and second for wintertime snowmelt contribution.

Summary of Change in Summer Ground Water Flow Volumes (Acre-Feet/Year)

Developed Condition	Infiltration Rate (%)	Irrigation Application Rate (in)			
		0	30	48	70
Entire Development					
Undeveloped	5	30.7	-	-	-
	15	92.0	-	-	-
Developed	5	-	43.2	54.1	67.5
	15	-	129.5	162.4	202.5
Development Tributary to Orchard Hills Area					
Undeveloped	5	16.0	-	-	-
	15	47.8	-	-	-
Developed	5	-	22.5	28.2	35.1
	15	-	67.3	84.4	105.3

Summary of Anticipated Changes in Ground Water Level near Orchard Hills (~~feet~~ inches)

Condition	Infiltration Rate (%)	Irrigation Application Rate (in)			
		0	30	48	70
Summer Period	5	-	7	13	20
	15	-	2	39	60
Winter Period	50	68.5	-	-	-

Based on information provided today from JVWCD, we would likely identify the best estimate potential summer impact to lie somewhere between the upper ranges of the figures identified. Averaging these four numbers gives an overall estimated summer rise in the ground water table (due to rainfall and irrigation) to be approximately 33 inches (2.75 feet). A direct comparison of this value versus that shown during spring snowmelt runoff shows that snowmelt is the worst case condition. Overall, the development helps reduce snowmelt contribution to ground water while increasing summer ground water contributions.

It should be remembered that these calculations and estimates are based on the assumptions made and may vary depending upon highly variable ground water conditions. A more extensive study and effort, potentially including ground water modeling which could take into account ground water flow dynamics would be required to provide more refined estimates. The data provided herein however should be adequate to provide you with a reasonable estimate of general impact.

Please call should you have any questions regarding the information presented herein, or if you would like additional details or analysis.

Sincerely;

HANSEN, ALLEN & LUCE, INC.

By:

David E. Hansen, Ph.D., P.E.
Principal / Project Manager

Filename: Jim Nordquist.06082001

HANSEN ALLEN & LUCE INC

CLIENT AGEC
PROJECT EAGLEPOINTE
FEATURE HYDROLOGY
PROJECT NO. 133.09.100

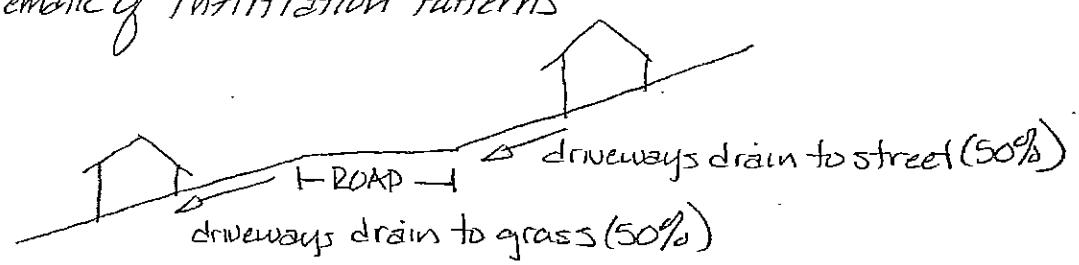
SHEET 1 OF 1
COMPUTED DEH
CHECKED GIP
DATE 6 JUNE 01

GOAL/OBJECTIVE: • Prepare a Water Balance showing Impact of Development on Subdivision
• Estimate change in GW level

ASSUMPTIONS

- No development prior to Eaglepointe
- Soil infiltrations can be averaged
- Base data provided by AGEC is correct
- Streets and driveways drain to Storm Water Collection System and do not contribute to GW. (50% of driveways)
- Sidewalks drain to lawns and parking strips and contribute to GW system
- Roofs drain to lawns
- Uphill driveways drain to the Street ±50%
- Downhill driveways drain to the Landscaped Area ±50%
- Missing acreage in Eaglepointe Development is to be applied to undeveloped area
- Use State Precip data: Avg Annual for Bountiful : 23.57 in
  Peak Monthly (May) : 3.22 in

Schematic of Infiltration Patterns



- Ground water infiltration ranges from 5-15% typically
- Areas of development

	Eaglepointe	Edgewood	Total	
Total Subdivision Area	278.8	33.5	312.3	
Undeveloped Area	40.9 (1)	0	40.9	
Developed Area	237.9	33.5	271.4	
Roof Area	32.6	9.9	42.5	
Pervious Area	142.9	17.0 (4)	159.9	
Roadway	47.2	4.1 (5)	51.2	
Sidewalk	3.0 (2)	0.2 (6)	3.3	Included
Driveways	12.2 (3)	2.3	14.5	
Landscaped Area	130.4 (7)	15.6	146.0	

(7) 10,000 ft²/lot * 548 lots / 43560 ft²/ac = 130.4 ac
(6) 5% of Road area - 3.4 * 0.05 = 0.2 ac
(5) 95% of Road area - 3.4 * .95 = 3.2 ac + 0.9 ac park/drive to street = 4.1
(4) Includes 15.6 ac landscaped + 1.4 ac drive/park to ground
(3) 80% of Driveways + Sidewalks = 0.8 * 15.2 = 12.2 ac
(2) [Streets, driveways, sidewalks] - [Roadway] = 62.4 - 47.2 = 15.2 ac (Sidewalks 20% = 3 ac)
(1) Includes 23.1 ac Identified plus 17.8 ac error in total acreages

Infiltration typically ranges between 5-15%

- Undeveloped Conditions

@ 5%

$$i = Precip(in) * \left(\frac{ft}{12in}\right) * acres * Inf\%$$

$$i = \frac{23.57 in}{12} * 312.3 ac * 0.05 = \underline{\underline{30.7 AF/yr}}$$

@ 15%

$$i = 30.7 * \frac{0.15}{0.05} = \underline{\underline{92.0 AF/YR}}$$

- Developed Conditions

Define Patterns:

Condition	Result
Undeveloped Area	5-15% Infiltrates
Roof Area	5-15% Infiltrates
Pervious Area	5-15% Infiltrates
Roadway	0% Infiltrates - Carried by Storm Drn
Sidewalk	5-15% Infiltrates
Driveways	50% of Area - 5-15% Infiltrates

Additional Additions:

Irrigation Water	Applied @ a rate of 30" / year, of which 5-15% infiltrates to ground water.

@ 5%

$$i = \frac{23.57}{12} * \overbrace{\left(40.9 + 42.5 + 159.9 + 3.3 + (0.5 * 14.5)\right)}^{253.9} * 0.05 + 146 * \frac{30"}{12} * 0.05$$

$$= 24.9 + 18.3 = \underline{\underline{43.2 AF/YR}}$$ (Up from 30.7 AF/YR undeveloped) $\underline{\underline{+41\%}}$

@ 15%

$$i = \left[\frac{23.57}{12}\left[40.9 + 42.5 + 159.9 + 3.3 + (14.5/2)\right] + 146 * \frac{30}{12}\right] * 0.15$$

$$= 74.8 + 54.8 = \underline{\underline{129.5 AF/YR}}$$ (Up from 92 AF/YR undeveloped) $\underline{\underline{+41\%}}$

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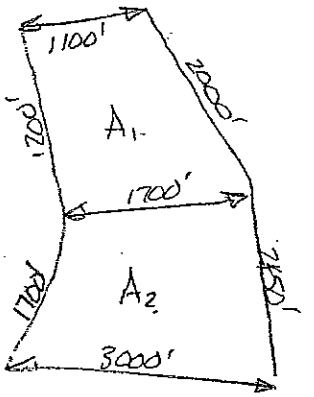
PROJECT Eaglepointe
FEATURE
PROJECT NO. 133.04.100

SHEET 3 OF 1
COMPUTED DEH
CHECKED GLP
DATE 6/7/2001

Evaluate Approx. Contributing area to Orchard Hills Area

Schematic of Area Contributing

Get Quick Area from Zone



$A_1 = \frac{1100+1700}{2} * \frac{2000+1200}{2}$

$= 1400 * 1600$

$= 2{,}240{,}000\ ft^2 = 51.4\ ac$

$A_2 = \frac{1700+3000}{2} * \frac{1700+2450}{2} = 2350 * 2075 = 4{,}876{,}250\ ft^2$
$= 111.94\ ac$

$A_1 + A_2 = 51.4 + 111.94 = 163.34$ Acres

Determine Volume of water entering Orchard Hills Area based on Ratio of total Area to Contributing Area

$Ratio = \frac{163.3\ ac}{312.3\ ac} = \underline{\underline{0.52}}$

Condition	Infiltration Assumption 5%		15%	
	All Area	0.52 Area	All Area	0.52 Area
Undeveloped	30.7 AF	16.0 AF	92.0 AF	47.8 AF
Developed	43.2 AF	22.5 AF	129.5 AF	67.3 AF

Estimate Change in Water Level due to development

Change is going to occur during Peak Runoff Period

Check May vs August Periods

May precip = 3.22 in Average

Lawn Watering = ±30"/yr

Distribution: Outside Watering May — October

Assume that Watering Pattern is

	Rate	%	Application
May	0.5	7	2.1
Jun	1	14	4.2
Jul	2	29	8.7
Aug	2	29	8.7
Sept	1	14	4.2
Oct	0.5	7	2.1
	7.0	100%	30.0"

Check Comparison between Months

Month	Rain	Irr	Total
May	3.22	2.1	5.32
Jun	1.29	4.2	5.49
Jul	0.96	8.7	9.66
Aug	0.98	8.7	9.68 ← PEAK
Sept	1.80	4.2	6.00
Oct	2.22	2.1	4.32

Rainfall Contribution

Undeveloped:

$i = \frac{0.98 in}{12} * 312.3 ac * .05 = 1.28 AF$ @ 5%

$* 0.15 = 3.83 AF$ @ 15%

Developed:

$i = \frac{0.98}{12} * 253.9 * 0.05 + 146 * \frac{8.7}{12} * 0.05 = 6.3 AF$ @ 5%

$i = \left[\left(\frac{0.98}{12} * 253.9\right) + \left(146 * \frac{8.7}{12}\right)\right] * 0.15 = 19.0 AF$ @ 15%

Percent Area that contributes to GW flow Conditions

Permeable Area / Total Area = 253.9 / 312.3 = 0.81 = 81%

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PROJECT NO. 133.04.100

SHEET 1 OF 1
COMPUTED DFH
CHECKED GLP
DATE 6/7/01

Change in Water Level is difference between applications of rainfall + irrigation, adjusted for porosity and % contributing Area

Undeveloped Change - Peak during May = 3.22 in

0.81 * 3.22 in * 5% inf. / 10% porosity = 1.3 inches
* 15% inf / 10% porosity = 3.9 inches

Developed Conditions - Peak during August P = 0.98" Irr = 8.7" } = 9.68"

0.81 * 9.68 in * 0.05 / 0.1 = 3.9 inches
* 0.15 = 11.8 inches

Summary - @ 5% - Δh = 3.9 - 1.3 inches = 2.6 inches
@ 15% - Δh = 11.8 - 3.9 inches = 7.9 inches

Add impact of reducing Capture width on South side of property (w = 3000') to 1100 ft on North side by Orchard Hills area

$\frac{3000}{1100}$ = 2.73 factor

Net increases

2.6 * 2.73 = 7.1 inches @ 5%
7.9 * 2.73 = 21.6 inches @ 15%

Water consumption may in reality approach 48" during the irrigation season - Check the impact if 48" is used

Month	Rain	Irrigation	Total
May	3.22	3.36	6.58
Jun	1.29	6.72	8.01
Jul	0.96	13.92	14.88
Aug	0.98	13.92	14.90 ← Peak
Sept	1.80	6.72	8.52
Oct	2.22	3.36	5.58
		48.0"	58.47"

Developed

$i = \frac{0.98}{12} * 253.9 * 0.05 + 146 * \frac{13.92}{12} * 0.05 = 9.5$ AF @ 5%

$i = \frac{0.15}{12}\left[0.98 * 253.9 + 146 * 13.92\right] = 28.5$ AF @ 15%

Water Level Increase

$0.81 * 14.90 * 0.05/0.1 = 6.0$ inches
$* 0.15 = 18.1$ inches

$\Delta h = 6.0 - 1.3 = 4.7$ inches @ 5%
$18.1 - 3.9 = 14.2$ inches @ 15%

Effect of reduction in flow path width

$4.7 * 2.73 = 12.8$ inches @ 5%

$14.2 * 2.73 = 38.8$ inches @ 15%

Contact w/ JVWCD indicated up to 65-70" is being applied!

Use 70"

Month	Rain	Irr	Total
Aug	0.98	20.30	21.28

Developed

$i = \frac{0.05}{12}\left[0.98 * 253.9 + 146 * 20.30\right] = 13.4$ AF @ 5%
$= 40.1$ AF @ 15%

Water Level Increase

$0.81 * 21.28 * 0.05/0.1 = 8.6$ inches
$* 0.15 = 25.9$ inches

$\Delta h = (8.6 - 1.3'')\,2.73 = 19.9''$ @ 5%

$(25.9'' - 3.9'')\,2.73 = 59.9''$ @ 15%

CHECK POTENTIAL SNOWMELT IMPACTS.

RAINFALL PATTERNS

MONTH	Precip (in)
OCT	2.22
NOV	2.06
DEC	1.88
JAN	1.99
FEB	2.01
MAR	2.46
APR	2.69

- Worst case Snowmelt will occur in Springtime
- Assume Feb + March snows acumulate, then thaw with April Rainfall.

Rainfall = 2.01 + 2.46 + 2.69 = 7.16 inches
Irrigation = Ø inches

Developed:

$$i = \frac{0.05}{12}\left[7.16 * 253.9 + 1.46 * 0\right] = 7.57 \text{ AF @ } 5\%$$
$$= 22.72 \text{ AF @ } 15\%$$

Water Level Increase - With Sublimation, Refusal + Soil Moisture Detention - Infiltration is 50%

0.85 * 7.16 * 0.5/0.1 = 29 inches @ 50%

Δh = 29 - 3.9 = 25.1 inches

25.1 * 2.73 = 68.5 inches

CHECK UNDEVELOPED CONDITIONS

1.0 * 7.16 * 0.5/0.1 = 35.80 inches @ 50%

Δh = 35.8 - 3.9 = 31.9 inches

31.9 * 2.73 = 87.1 inches

NET IMPACT - DEVELOPMENT REDUCES SNOWMELT RUNOFF ≈ 20% (G.W. Component)

HANSEN ALLEN & LUCE inc
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Mr. Jim Nordquist
Applied Geotechnical Engineering Consultants
600 West Sandy Parkway
Salt Lake City, Utah 84070

December 17, 2002

RE: Response to comments regarding Eaglepointe Subdivision Hydrogeologic Review dated June 8, 2001.

Dear Jim:

As requested we have reviewed comments regarding our June 8, 2001 evaluation of the Eaglepointe subdivision and provide the following response. As you will note, a response is given for each point identified in the review letter regarding our evaluation.

COMMENT: The hydrologic evaluation presented is based on speculative assumptions.

Response: When conducting an evaluation of this type there are two basic approaches that can be taken depending upon the amount of data available, one based on assumptions, and the other a more rigorous approach based on specific local data. If little data is available, then assumptions used must include what is professionally believed to be a reasonable range of expected data. An evaluation on the other hand using site specific data can provide a tighter understanding of local conditions, however, since variation exists in all data sets, even this evaluation must acknowledge some degree of error. The acquisition of sufficient site specific data to provide the latter evaluation would require the setup and acquisition of actual rainfall/runoff measurements during and following several local rainstorms. Such a monitoring program would not only be expensive but require significant time, both issues of which are prohibitive for most projects.

As a result it was decided to proceed with an evaluation based on reasonable assumptions for the area. Assumptions made were clearly identified in the calculations provided in the June 8, 2001 letter. As shown in the letter, sound engineering decisions can be made if after making the assumptions it can be shown that the results are conservative. For example, if after using extreme conditions it is concluded that impacts are inconsequential, then it stands to reason that more

realistic values will only provide less consequential results. Such was the basis of the analysis completed.

QUESTION: Do the percolation estimates take into account surface slope or is a single infiltration value assumed regardless of slope?

Response: As can be seen in the calculation sheets, the analysis completed did not specifically take into account slope, but used a range of infiltration rates.

QUESTION: Are the calculations of rise/fall in groundwater cumulative, ie. Do they look at the cumulative effect of the development over several summer/winter cycles?

Response: No, as can be seen in the calculations they show undeveloped conditions based on average annual rainfall; and developed conditions based on rainfall plus irrigation during summer months, and snowfall and runoff contributions during February through April (typical peak snowfall and runoff months).

A more detailed evaluation attempting to predict cumulative conditions would require a month by month evaluation of rainfall and snowfall events, and would require that additional assumptions be made. Conditions or assumptions that would be most difficult to predict or evaluate include those related to the timing of snowfall versus runoff, which itself is highly impacted by such items as variable freeze / thaw and soil saturation conditions. If such an evaluation were to be required, it would be highly desirable to coordinate with the individual(s) reviewing the work so that appropriate methodologies can be developed and assumptions made that are mutually acceptable.

QUESTION: What is the percent increase in the summer when nearly 4 times the annual precipitation will be applied as irrigation to lawns and landscaped areas?

Response: Projections of volume and water level increases based on developed conditions are outlined within the tables shown on Page 2 of the June 8, 2001 letter.

QUESTION: Are the assumption based (acknowledged by the authors) infiltration rates and the other seepage migration parameters used to draw report conclusions adequate for the intended purpose of identifying impacts to off-site facilities?

Response: It is our opinion that the assumptions used are reasonable for the analysis, otherwise they would not have been used. We would however welcome and give consideration to any comments made regarding the assumptions should it be suggested that differing values may be more appropriate.

Please call should you have any questions regarding the information presented herein, or if you would like additional details or analysis.

Sincerely;

HANSEN, ALLEN & LUCE, INC.

By:

David E. Hansen, Ph.D., P.E.
Principal / Project Manager

APPENDIX C

APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS, INC.

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

PROJECT NUMBER 1020320

SAMPLE LOCATION		NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (PCF)	GRADATION			ATTERBERG LIMITS		UNCONFINED COMPRESSIVE STRENGTH (PSF)	WATER SOLUBLE SULFATE (PPM)	SAMPLE CLASSIFICATION
TEST PIT	DEPTH (FEET)			GRAVEL (%)	SAND (%)	SILT/ CLAY (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)			
TP-2	4	23	97			96	81	63			Tuffaceous Bedrock
TP-4	4½									<10	Fill; Lean Clay
TP-5	4	2		74	25	1					Poorly Graded Gravel with Sand (GP)
TP-7	4	1		57	41	2					Poorly Graded Gravel with Sand (GP)
TP-8	4	17	107			70	62	49			Tuffaceous Bedrock
	10	30				83	99	80			Tuffaceous Bedrock
TP-9	5	43	75			96	46	23			Lean Clay (CL)
TP-10	4	15	111			80	56	41			Tuffaceous Bedrock
TP-11	2	20	97			66	54	37			Tuffaceous Bedrock
TP-13	4	7		40	46	14					Fill; Silty Sand with Gravel
TP-14	4	2		45	53	2					Poorly Graded Sand with Gravel (SP)

APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS, INC.

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SUMMARY OF LABORATORY TEST RESULTS

PROJECT NUMBER 1020320

SAMPLE LOCATION		NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (PCF)	GRADATION			ATTERBERG LIMITS		UNCONFINED COMPRESSIVE STRENGTH (PSF)	WATER SOLUBLE SULFATE (PPM)	SAMPLE CLASSIFICATION
TEST PIT	DEPTH (FEET)			GRAVEL (%)	SAND (%)	SILT/ CLAY (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)			
TP-15	4	31	88			82	34	13			Lean Clay with Sand (CL)
TP-19	4	9		44	34	22					Fill; Clayey Gravel with Sand
TP-21	2									<10	Silty Clay (CL-ML)
TP-22	4	18	86			62					Sandy Lean Clay (CL)
TP-24	4	17	89			51					Sandy Lean Clay (CL)
TP-25	4	26	100			70	66	51			Tuffaceous Bedrock
TP-27	4	6	94			22					Silty Sand (SM)
TP-28	2	8	91			48					Clayey Sand (SC)
TP-31	2									<10	Silty Clay (CL-ML)
TP-33	12	16	99			78	55	36			Tuffaceous Bedrock
TP-35	13½	1		69	29	2					Poorly Graded Gravel with Sand (GP)

APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS, INC.

TABLE I
SUMMARY OF LABORATORY TEST RESULTS

PROJECT NUMBER 1020320

SAMPLE LOCATION		NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (PCF)	GRADATION			ATTERBERG LIMITS		UNCONFINED COMPRESSIVE STRENGTH (PSF)	WATER SOLUBLE SULFATE (PPM)	SAMPLE CLASSIFICATION
TEST PIT	DEPTH (FEET)			GRAVEL (%)	SAND (%)	SILT/ CLAY (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)			
TP-37	4	33	84			87	75	52			Fat Clay with Sand (CH)
TP-38	4	7		34	39	27					Clayey Sand with Gravel (SC)
TP-39	2	16	108			64					Sandy Fat Clay (CL)
TP-40	2									<10	Lean Clay with Sand (CL)
	4	21	97			74	48	36			Lean Clay with Sand (CL)
	Sample										
HB-1	"B"	14	97	0	27	73	31	14			Lean Clay with Sand (CL)
HB-2	"A"	26	94	0	22	78	31	8			Silt with Sand (ML)
	"B"	26	93	0	20	80	31	16			Lean Clay with Sand (CL)
HB-3	"A"	31	88	0	14	86	38	21			Lean Clay (CL)
	"B"	27	94			81	38	23			Lean Clay with Sand (CL)
	"C"	33	87			80	43	28			Lean Clay with Sand (CL)
HB-5	"A"	4	122	23	11	66	38	20			Lean Clay with Gravel (CL)
	"B"	5	120	9	37	54	45	31			Sandy Lean Clay (CL)
	"C"			0	26	74	68	53			Fat Clay with Sand (CH)
	"D"			5	29	66	46	33			Sandy Lean Clay (CL)
	"E"			0	15	85	25	9			Lean Clay with Sand (CL)