Appendix D
Soils Report

HEPWORTH · PAWLAK GEOTECHNICAL

PRELIMINARY GEOTECHNICAL STUDY
PROPOSED ASSISTED LIVING COMPLEX
LOTS 2, 3 AND 4, FILING 26, EAGLE RANCH
SYLVAN LAKE ROAD
EAGLE, COLORADO

JOB NO. 112 269B

SEPTEMBER 20, 2012

PREPARED FOR:

EAGLE COUNTY GOVERNMENT
ATTN: JILL KLOSTERMAN
P. O. BOX 850
EAGLE, COLORADO 81631
jill.klosterman@eaglecounty.us

TABLE OF CONTENTS

PURPOSE AND SCOPE OF STUDY

This report presents the results of a preliminary geotechnical study for the proposed assisted living complex to be located on Lots 2, 3 and 4, Filing 26, Eagle Ranch, Sylvan Lake Road, Eagle, Colorado. The project site is shown on Figures 1 and 1A. The purpose of the study was to develop recommendations for the preliminary building foundation and site grading design. The study was conducted in general accordance with our proposal for geotechnical engineering services to Eagle County Government dated July 18, 2012. The field exploration scope was verbally modified by Rick Ullum to include a boring in the two future development areas at the east and west ends of the property.

A field exploration program consisting of exploratory borings was conducted to obtain information on the subsurface conditions. Samples of the subsoils obtained during the field exploration were tested in the laboratory to determine their classification, compressibility or swell and other engineering characteristics. The results of the field exploration and laboratory testing were analyzed to develop preliminary recommendations for foundation design of the proposed buildings, and for the site grading and on-site pavement section thickness designs. This report summarizes the data obtained during this study and presents our conclusions, recommendations and other geotechnical engineering considerations based on the assumed construction and the subsurface conditions encountered.

BACKGROUND INFORMATION

Hepworth-Pawlak Geotechnical, Inc. previously performed preliminary geologic and geotechnical studies for the Eagle Ranch Development, submitting our findings in reports dated November 16, 1998, February 25, 1999, Job No. 197 567 and a study of known sinkholes at Eagle Ranch, report dated June 30, 2000, Job No. 197 567. We also performed a preliminary geotechnical study for subdivision development that included the subject development property, report dated November 30, 2006, Job No. 106 0668. Information from these reports has been considered in the preparation of this report.

PROPOSED CONSTRUCTION

The development conceptually consists of a large footprint of interconnected buildings that will provide various functions, nurse units, common center and living units located in roughly the central portion of the site and flanked on the east and west ends by future independent living units. The buildings are assumed to be 1 to 2 stories in height with slab-on-grade or structural floors above crawlspace. There will be paved drives and parking lot areas within the development. We assume foundation loadings for the buildings will be relatively light and typical of the assumed construction. We expect the site grading will be relatively minor with cut and fill depths up to about 6 to 8 feet. Existing buried utilities on the property will likely be abandoned or re-routed as part of the development.

When building location, grading and loading information have been developed, we should be notified to re-evaluate the recommendations presented in this report.

SITE CONDITIONS

The parcel is vacant and about 5 acres in size. The ground surface appears mostly natural except for minor surface grading including occasional end dumped fill piles and a small gravel parking area near the northeast end off of Sylvan Lake Road. There are existing buried electric and sewer lines through the site. The terrain is gently to strongly sloping down generally to the northwest at about 3 to 6% grades. A steep hillside is located beyond the west end of the property. Elevation difference across the proposed development is about 15 feet ranging from 6620 to 6605 feet. The previous shallow depression in southwest central part of the site shown by the circular contours on Figure 1 has been graded over by site development of Freestone Road and housing project located to the southwest of the current project development area. Vegetation consists of grass and weeds with trees along Sylvan lake Road. The improvement survey of the current site is shown on Figure 1A.

GEOLOGIC CONDITIONS

The soils at the site consist of alluvial fan deposits from erosion of the steep hilly terrain mainly to the southwest of the site that overlie river terrace gravels. Geologic conditions which may impact the development appear limited to the potential for sinkhole development and compressible soils. There is no debris flow hazard risk at subject development area. Surface runoff from the steep hillside to the west side of the site should be considered in the site drainage design. A discussion of the compression potential of the soils is included in the "Preliminary Geotechnical Recommendations" section of this report. Based on the subsurface conditions, we believe that Site Class D from 2009 IBC Table 1613.5.2 can be used for the seismic building design of the foundations.

Eagle Ranch is underlain by Pennsylvania age Eagle Valley Evaporite bedrock. The evaporite contains gypsum deposits. Dissolution of the gypsum under certain conditions can cause sinkholes to develop and can produce areas of localized subsidence. A series of three small sinkholes identified in our June 30, 2000 report as Sinkhole Area B is located about 300 feet to the south of the property on the adjacent school property and is not an issue to the current proposed development. The small depression area in the west-central perimeter of the site may be related to utility construction and is now covered over by Freestone Road.

Our exploratory borings for the current study were relatively shallow, but no indications of subsurface voids were encountered. Based on our present knowledge of the site, it cannot be said for certain that sinkholes will not develop. In our opinion, the risk of ground subsidence at the current proposed development area is low and similar to other subdivisions in the area, but the owner should be aware of the potential for sinkhole development. If further evaluation of the sinkhole potential at the site is desired, we should be contacted.

FIELD EXPLORATION

The field exploration for the project was conducted on August 20, 21 and 22, 2012. Nine exploratory borings were drilled at the locations shown on Figure 1 to evaluate the general subsurface conditions. The borings were advanced with 4 inch diameter continuous flight augers powered by a truck-mounted CME-45B drill rig. The borings were logged by a representative of Hepworth-Pawlak Geotechnical, Inc. Approximate location of the 6 borings drilled for the 2006 preliminary geotechnical study are also shown on Figure 1.

Samples of the subsoils were taken with 1⅜ inch and 2 inch I.D. spoon samplers. The samplers were driven into the subsoils at various depths with blows from a 140 pound hammer falling 30 inches. This test is similar to the standard penetration test described by ASTM Method D-1586. The penetration resistance values are an indication of the relative density or consistency of the subsoils and hardness of the bedrock. Depths at which the samples were taken and the penetration resistance values are shown on the Logs of Exploratory Borings, Figures 2 and 3. The samples were returned to our laboratory for review by the project engineer and testing.

SUBSURFACE CONDITIONS

Graphic logs of the subsurface conditions encountered at the site are shown on Figures 2 and 3. The subsoils encountered, below typically about 1 foot of organic topsoil or disturbed natural soils, typically consist of stiff and moist to medium stiff/soft and wet, silty sandy clay underlain at depths from about 14 to 26 feet by medium dense to dense, silty sandy gravel with cobbles that extended to the typical depths drilled of 20 to 31 feet. At Boring 6, claystone bedrock was encountered below the gravel soils at a depth of 33 feet that extended to the boring depth of 40 feet. The upper soils contained zones of silty sand volcanic ash between depths of about 3 to 10 feet. At Borings 8 and 9, about 5 to 9 feet of mixed silt and clay with gravel and cobbles was encountered over the clay soils. The subsoils are similar to those encountered in the previous borings drilled on the site as part of our 2006 preliminary study.

Laboratory testing performed on samples obtained from the borings included natural moisture content and density, gradation analyses, Atterberg limits, and unconfined compressive strength. Results of swell-consolidation testing performed on relatively undisturbed drive samples of the silty clay soils up to about 10 feet deep, presented on Figures 5, 6 and 7 generally indicate low to moderate compressibility under conditions of loading and wetting. The silty sand (volcanic ash) sample from Boring 4 at 4 feet showed a low collapse potential when wetted under a constant 1,000 psf surcharge load. Atterberg limits testing indicate the silty clay soils generally have low plasticity. The unconfined compressive strength testing indicates the moist clays have stiff consistency and the wet clays have soft consistency. The laboratory testing is summarized in Table 1.

Free water was encountered in the borings at the time of drilling and when checked 1 or more days following drilling at depths from about 17 to 22 feet below ground surface. The subsoils were slightly moist to moist in the upper 5 to 10 feet, becoming very moist and wet with depth. The groundwater levels are similar to those measured in the 2006 preliminary study.

ENGINEERING ANALYSIS

Development of the site as conceptually planned appears feasible based on geotechnical considerations. Spread footings or monolithic slabs bearing on the natural soils should be adequate for foundation support of the assumed lightly loaded buildings, with some risk of settlement. The risk of settlement is due to the compressible nature of the silty clay soils and if the upper soils become wetted. Placing a certain depth of granular structural fill below the footings would help to reduce the settlement potential. The existing sewer and electric line trenches should be avoided by the infrastructure and buildings. If avoidance is not possible, the potential backfill settlement should be mitigated such as by removal of the backfill and placement of highly compacted select granular backfill.

For settlement sensitive or heavily loaded structures, screwpiles or driven piles may be needed for foundation support. Concrete filled pipe piles are a suitable driven pile type and should develop their structural capacity when driven to refusal. Provided below are recommendations for spread footings and monolithic slabs. If recommendations for screwpiles or driven piles are desired, we should be contacted.

Groundwater levels measured at the site indicate that basement level should be feasible based on the current groundwater levels. Deeper excavations may encounter soft and very moist soils and it may not be feasible to construct spread footing supported foundations.

PRELIMINARY DESIGN RECOMMENDATIONS

Provided in the following sections are geotechnical recommendations considered suitable for planning and preliminary design at the site. We should review the proposed development and perform additional analyses as needed. This may need to include additional subsurface exploration.

FOUNDATIONS

Considering the subsurface conditions encountered in the exploratory borings and the nature of the proposed construction, we recommend the buildings be founded with spread footings or monolithic slabs bearing on the natural soils or compacted structural fill with some risk of settlement. In general, structural fill below building foundations should be limited to about 3 to 4 feet depth to limit settlement potential of the fill. A site specific evaluation of the bearing soils should be performed at each building site.

The design and construction criteria presented below should be observed for a spread footing or monolithic slab foundation system.

1) Footings or monolithic slabs placed on the undisturbed natural soils should be designed for an allowable bearing pressure of 1,200 psf. Based on experience, we expect settlement of footings designed and constructed as discussed in this section will be about 1 to 1½ inches for lightly loaded foundations.

2) Spread footings bearing on a minimum 3 feet of compacted structural fill can be designed for an allowable soil bearing pressure of 2,000 psf. The structural fill should consist of a fairly well graded granular import material compacted to at least 98% of the maximum standard Proctor density at a moisture content within about 2% of optimum. The structural fill should extend laterally beyond the edges of the footings a distance equal to at least the depth of fill below the footing. Expected settlements are similar to above.

3) The footings should have a minimum width of 18 inches for continuous walls and 2 feet for isolated pads.

4) Exterior footings and footings beneath unheated areas should be provided with adequate soil cover above their bearing elevation for frost protection. Placement of foundations at least 42 inches below exterior grade is typically used in this area.

5) Continuous foundation walls should be heavily reinforced top and bottom to span local anomalies and better withstand the effects of some differential settlement such as by assuming an unsupported length of at least 14 feet. Monolithic slabs should be heavily reinforced with both longitudinal and transverse steel. Foundation walls acting as retaining structures should also be designed to resist lateral earth pressures as discussed in the "Foundation and Retaining Walls" section of this report.

6) The existing fill, topsoil and any loose or disturbed soils should be removed and the footing bearing level extended down to the firm natural soils. The exposed soils in footing area should then be adjusted to near

optimum moisture content and compacted. Soft subgrade areas should be stabilized as needed such as by placing a heavy weight geogrid covered by at least 1 foot of crushed angular gravel.

7) A representative of the geotechnical engineer should observe all footing excavations and test structural fill prior to concrete placement to evaluate bearing conditions.

FOUNDATION AND RETAINING WALLS

Foundation walls and retaining structures which are laterally supported and can be expected to undergo only a slight amount of deflection should be designed for a lateral earth pressure computed on the basis of an equivalent fluid unit weight of at least 55 pcf for backfill consisting of the on-site soils. Cantilevered retaining structures which are separate from the building and can be expected to deflect sufficiently to mobilize the full active earth pressure condition should be designed for a lateral earth pressure computed on the basis of an equivalent fluid unit weight of at least 45 pcf for backfill consisting of the on-site soils.

All foundation and retaining structures should be designed for appropriate hydrostatic and surcharge pressures such as adjacent footings, traffic, construction materials and equipment. The pressures recommended above assume drained conditions behind the walls and a horizontal backfill surface. The buildup of water behind a wall or an upward sloping backfill surface will increase the lateral pressure imposed on a foundation wall or retaining structure. An underdrain should be provided to prevent hydrostatic pressure buildup behind walls.

Backfill should be placed in uniform lifts and compacted to at least 90% of the maximum standard Proctor density at a moisture content near optimum. Backfill in pavement and walkway areas should be compacted to at least 95% of the maximum standard Proctor density. Care should be taken not to overcompact the backfill or use large equipment near the wall, since this could cause excessive lateral pressure on the wall. Some settlement of deep foundation wall backfill should be expected, even if the material is placed correctly, and could result in distress to facilities constructed on the backfill.

The lateral resistance of foundation or retaining wall footings will be a combination of the sliding resistance of the footing on the foundation materials and passive earth pressure against the side of the footing. Resistance to sliding at the bottoms of the footings can be calculated based on a coefficient of friction of 0.35. Passive pressure of compacted backfill against the sides of the footings can be calculated using an equivalent fluid unit weight of 300 pcf. The coefficient of friction and passive pressure values recommended above assume ultimate soil strength. Suitable factors of safety should be included in the design to limit the strain which will occur at the ultimate strength, particularly in the case of passive resistance. Fill placed against the sides of the footings to resist lateral loads should be compacted to at least 95% of the maximum standard Proctor density at a moisture content near optimum.

NON STRUCTURAL FLOOR SLABS

The natural on-site soils, exclusive of topsoil, are suitable to support lightly loaded slab-on-grade construction. There could be some settlement of slabs due to the compressible soils. Providing 2 to 3 feet of structural fill below slabs should help limit settlements. To reduce the effects of some differential movement, non-structural floor slabs should be separated from all bearing walls and columns with expansion joints which allow unrestrained vertical movement. Floor slab control joints should be used to reduce damage due to shrinkage cracking. The requirements for joint spacing and slab reinforcement should be established by the designer based on experience and the intended slab use. A minimum 4 inch layer of free-draining gravel should be placed beneath basement level slabs (if provided) to facilitate drainage. This material should consist of minus 2 inch aggregate with at least 50% retained on the No. 4 sieve and less than 2% passing the No. 200 sieve.

All fill materials for support of floor slabs should be compacted to at least 95% of maximum standard Proctor density at a moisture content near optimum. Required fill can consist of the on-site soils devoid of vegetation, topsoil and oversized rock, or suitable granular soils can be imported.

UNDERDRAIN SYSTEM

Although free water was encountered below probable excavation depths during our exploration, it has been our experience in the area and where clay soils are present that local perched groundwater can develop during times of heavy precipitation or seasonal runoff. Frozen ground during spring runoff can also create a perched condition. We recommend below-grade construction, such as retaining walls, crawlspace and basement areas, be protected from wetting and hydrostatic pressure buildup by an underdrain system.

The drains should consist of drainpipe placed in the bottom of the wall backfill surrounded above the invert level with free-draining granular material. The drain should be placed at each level of excavation and at least 1 foot below lowest adjacent finish grade and sloped at a minimum 1% to a suitable gravity outlet. Free-draining granular material used in the underdrain system should contain less than 2% passing the No. 200 sieve, less than 50% passing the No. 4 sieve and have a maximum size of 2 inches. The drain gravel backfill should be at least 1½ feet deep.

SITE GRADING

Structural fill should be properly placed properly placed and compacted to limit settlements of the fill. Fill below the subdivision infrastructure should typically be compacted to at least 95% of the maximum standard Proctor density (SPD) at a moisture content within about 2% of optimum. Fill depths greater than about 8 to 10 feet should be compacted to at least 100% SPD. Prior to fill placement, the subgrade should be carefully prepared by removing all vegetation and topsoil, scarifying to a depth of about 8 inches adjusting to near optimum moisture content, and compacting to at least 95% of the maximum standard Proctor. Nonstructural fill should be compacted to at least 90% SPD.

Based on a maximum standard Proctor density value of about 110 pcf and an average in-situ dry density of about 95 pcf for the upper soils, indicates the compaction shrinkage value to be about 5 to 10% for the specified 95% SPD compaction. Due to loss during hauling and some overcompaction, we suggest a compaction shrinkage factor of 10 to 14% be used. Moisture conditioning of the on-site soils will be needed prior to their placement as structural fill.

SURFACE DRAINAGE

Positive surface drainage is an important aspect of the project to prevent wetting of the bearing soils. The following drainage precautions should be observed during construction and maintained at all times after the buildings have has been completed:

1) Inundation of the foundation excavations and underslab areas should be avoided during construction.
2) Exterior backfill should be adjusted to near optimum moisture and compacted to at least 95% of the maximum standard Proctor density in pavement and slab areas and to at least 90% of the maximum standard Proctor density in landscape areas.
3) The ground surface surrounding the exterior of the building should be sloped to drain away from the foundation in all directions. We recommend a minimum slope of 12 inches in the first 10 feet in unpaved areas and a minimum slope of 2½ inches in the first 10 feet in paved areas. Free-draining wall backfill should be capped with at least 2 feet of the on-site soils to reduce surface water infiltration.
4) Roof downspouts and drains should discharge well beyond the limits of all backfill.
5) Landscaping which requires regular heavy irrigation should be located at least 5 feet from foundation walls.

PAVEMENT SECTION THICKNESS

We understand that asphalt pavement is proposed for the drive and parking areas. Traffic loadings have not been provided. The subgrade soils encountered at the site are typically low plasticity silty clay with an AASHTO classification of A-6 and A-4 with Group Indices of 2 to 9. The silty clay soils are considered a relatively poor support for pavement sections and moderately susceptible to frost heave. We estimate a typical Hveem stabilometer 'R' value of about 6 for the silty clay soils. Providing 1 to 2 feet of granular subbase material below the pavement section would help limit the frost heave potential.

We assumed an 18 kip equivalent daily load application (EDLA) of about 15 for drives and an EDLA of about 5 for the automobile parking lot areas. Construction traffic could increase the assumed EDLA. Using a Regional Factor of 2.0, a serviceability index of 2.0, an 'R' value of 6 and the above assumed EDLA values, the following alternate minimum pavement sections are provided.

Location	Alternative Number	Asphalt (inches)	Base Course (inches)	Subbase (inches)
Drives	1	5	6	-
"	2	4	9	-
"	3	3	4	12
Parking	1	4	6	-
"	2	3	9	-

The asphalt should be a batched hot mix, approved by the engineer and placed and compacted to the project specifications. The base course and subbase should meet CDOT Class 6 and Class 2 specifications, respectively. All base course, subbase and required subgrade fill should be compacted to at least 95% of the maximum standard Proctor density within about 2% of optimum moisture content. For tight turning areas and areas subjected to typical truck traffic, such as trash pick-up and loading docks, consideration should be given to using 6 inches of portland cement concrete on 4 inches of base course as the pavement section. The concrete should have a minimum 28 day compressive strength of 4,500 psi and be air entrained.

Required fill to establish design subgrade level can consist of the on-site soils excluding topsoil and oversized rocks, or suitable imported granular soils approved by the geotechnical engineer. Prior to fill placement, the subgrade should be stripped of all topsoil, scarified to a depth of 8 inches, adjusted to near optimum moisture content, and compacted to at least 95% of standard Proctor density. In soft or wet areas, the subgrade may require drying or stabilization prior to fill placement. A geogrid and/or subexcavation and replacement with aggregate base materials may be needed for the stabilization. The subgrade should be proofrolled. Areas that deflect excessively should be corrected before placing pavement materials. The subgrade improvements and placement and compaction of base and asphalt materials should be monitored on a regular basis by a representative of the geotechnical engineer. Once traffic loadings have been determined by the traffic study, we should review our pavement section recommendations.

LIMITATIONS

This study has been conducted in accordance with generally accepted geotechnical engineering principles and practices in this area at this time. We make no warranty either express or implied. The conclusions and recommendations submitted in this report are based upon the data obtained from the exploratory borings drilled at the locations indicated on Figure 1, the proposed type of construction and our experience in the area. Our services do not include determining the presence, prevention or possibility of mold or other biological contaminants (MOBC) developing in the future. If the client is concerned about MOBC, then a professional in this special field of practice should be consulted. Our findings include interpolation and extrapolation of the subsurface conditions identified at the exploratory borings and variations in the subsurface conditions may not become evident until excavation is performed. If conditions encountered during construction appear different from those described in this report, we should be notified so that re-evaluation of the recommendations may be made.

This report has been prepared for the exclusive use by our client for planning and preliminary design purposes. We are not responsible for technical interpretations by others of our information. As the project evolves, we should provide continued consultation and field services during construction to review and monitor the implementation of our recommendations, and to verify that the recommendations have been appropriately interpreted. Significant design changes may require additional analysis or modifications to the recommendations presented herein. We recommend on-site observation of excavations and foundation bearing strata and testing of structural fill by a representative of the geotechnical engineer.

Respectfully Submitted,

HEPWORTH - PAWLAK GEOTECHNICAL, INC.

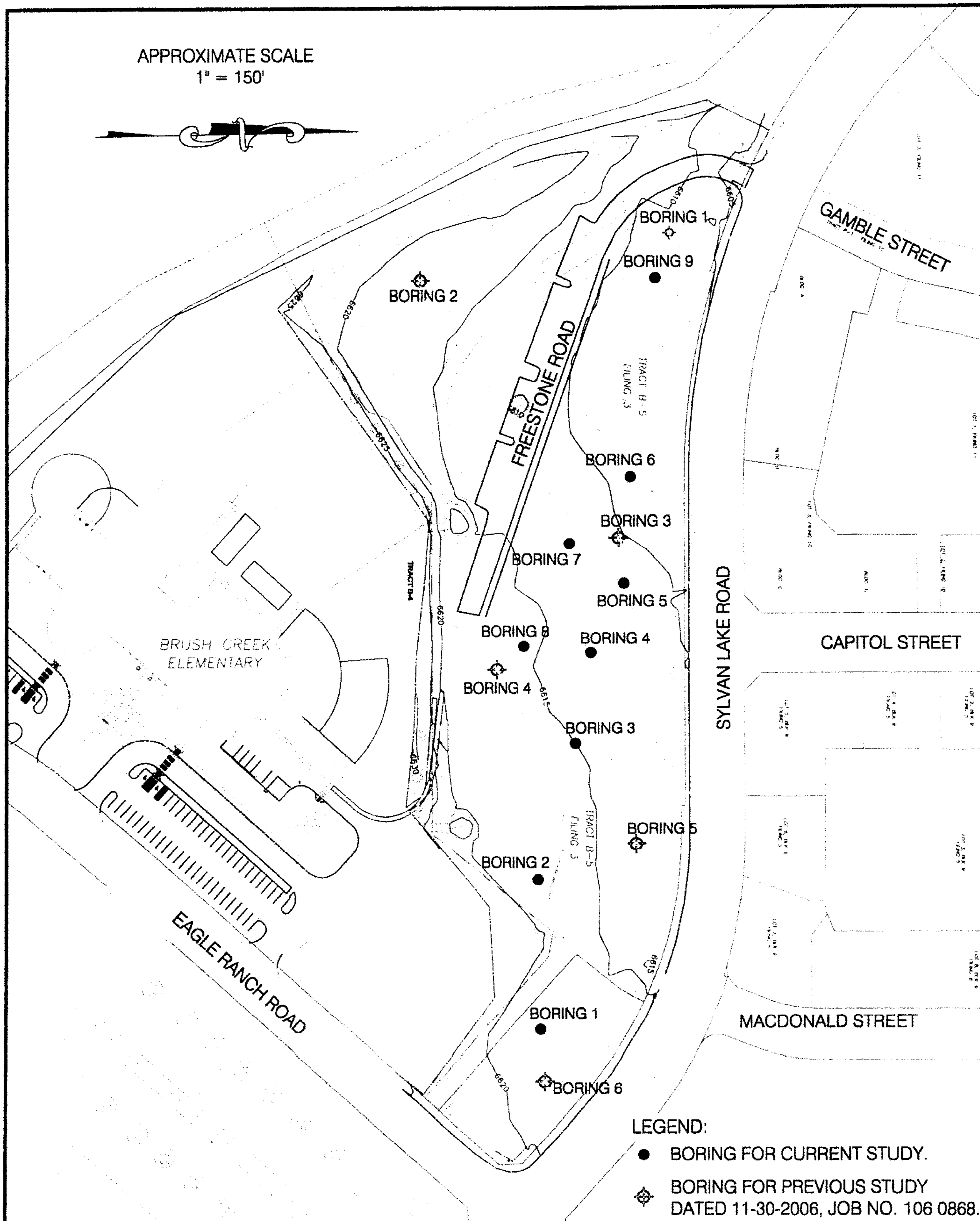
STEVEN L. PAWLAK
REGISTERED
15222
9/25/12
PROFESSIONAL ENGINEER
STATE OF COLORADO

Steven L. Pawlak, P.E.

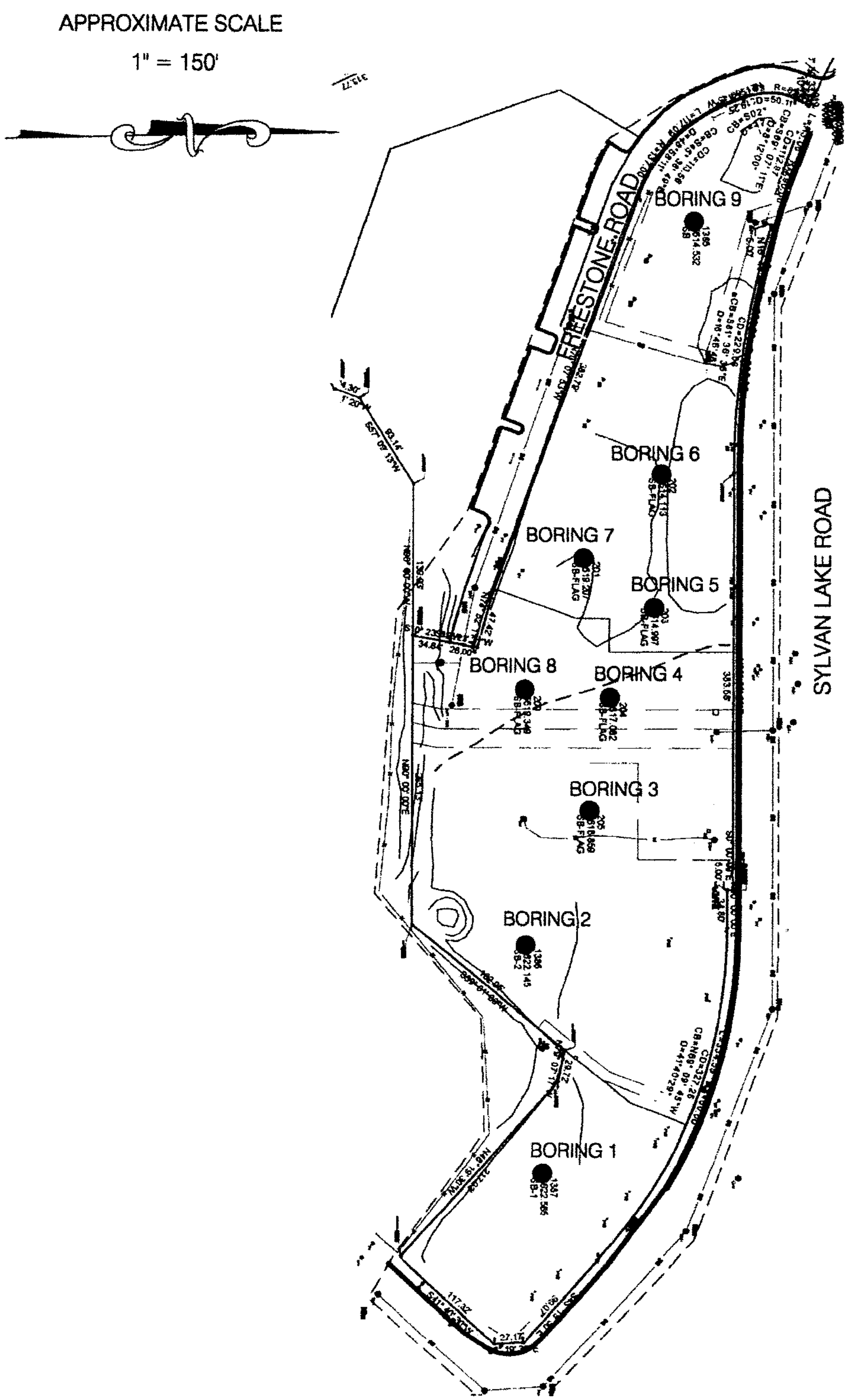
Reviewed by:

Daniel E. Hardin, P.E.

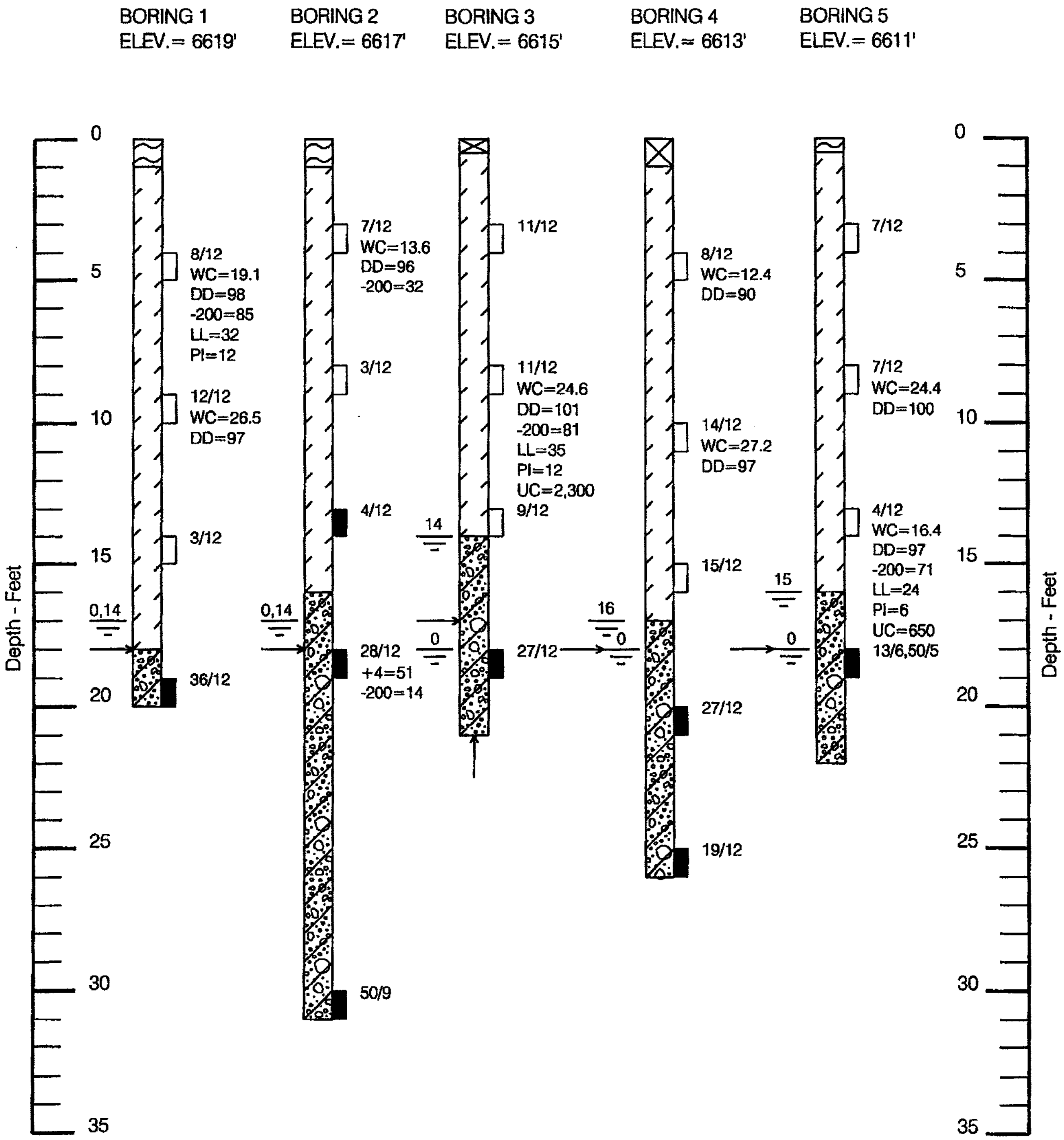
SLP/ksw



112 269B	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	LOCATION OF EXPLORATORY BORINGS	Figure 1

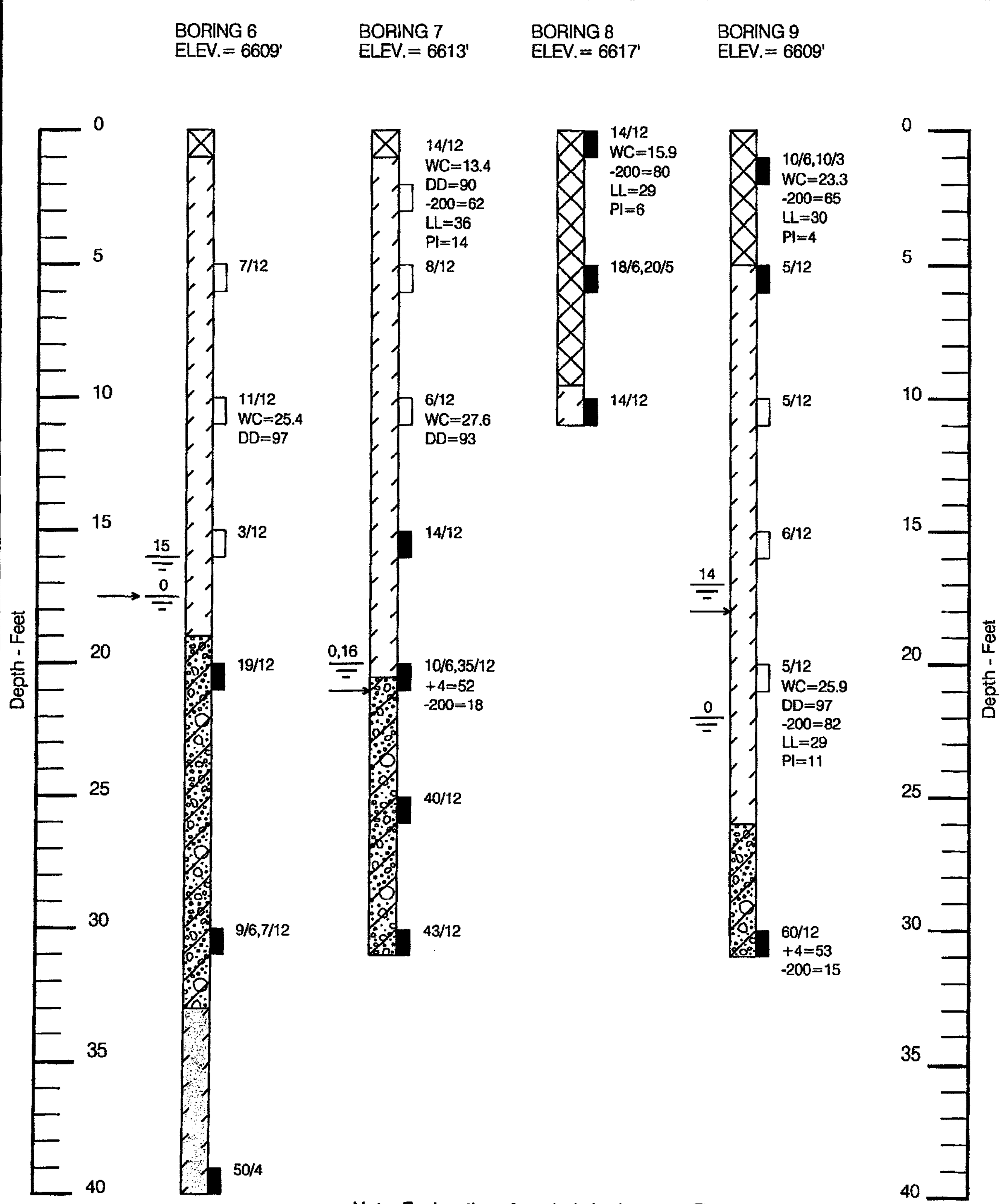


112 269B	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	SITE IMPROVEMENT SURVEY	Figure 1A



Note: Explanation of symbols is shown on Figure 4.

112 269B	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	LOGS OF EXPLORATORY BORINGS	Figure 2



Note: Explanation of symbols is shown on Figure 4.

112 269B	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	LOGS OF EXPLORATORY BORINGS	Figure 3

LEGEND:

FILL; mixed clay, silt and gravel with scattered cobbles, stiff/medium dense, gravel from old parking lot at Boring 3, slightly moist to moist, mixed brown.

TOPSOIL; organic silty clay, dark brown.

CLAY (CL); silty, sandy to sandy silt (Volcanic Ash) in typically upper 3 to 10 feet, stiff and moist to soft and very moist with depth, mixed grey-brown to brown with depth, low plasticity.

GRAVEL (GM); silty, sandy, cobbles, possible small boulders medium dense, wet, red-brown.

CLAYSTONE BEDROCK; silty, gypsum, medium hard to hard, grey. Eagle Valley Evaporite.

Relatively undisturbed drive sample; 2-inch I.D. California liner sample.

Drive sample; standard penetration test (SPT), 1 3/8 inch I.D. split spoon sample, ASTM D-1586.

8/12 Drive sample blow count; indicates that 8 blows of a 140 pound hammer falling 30 inches were required to drive the California or SPT sampler 12 inches.

0,2 Free water level in boring and number of days following drilling measurement was taken.

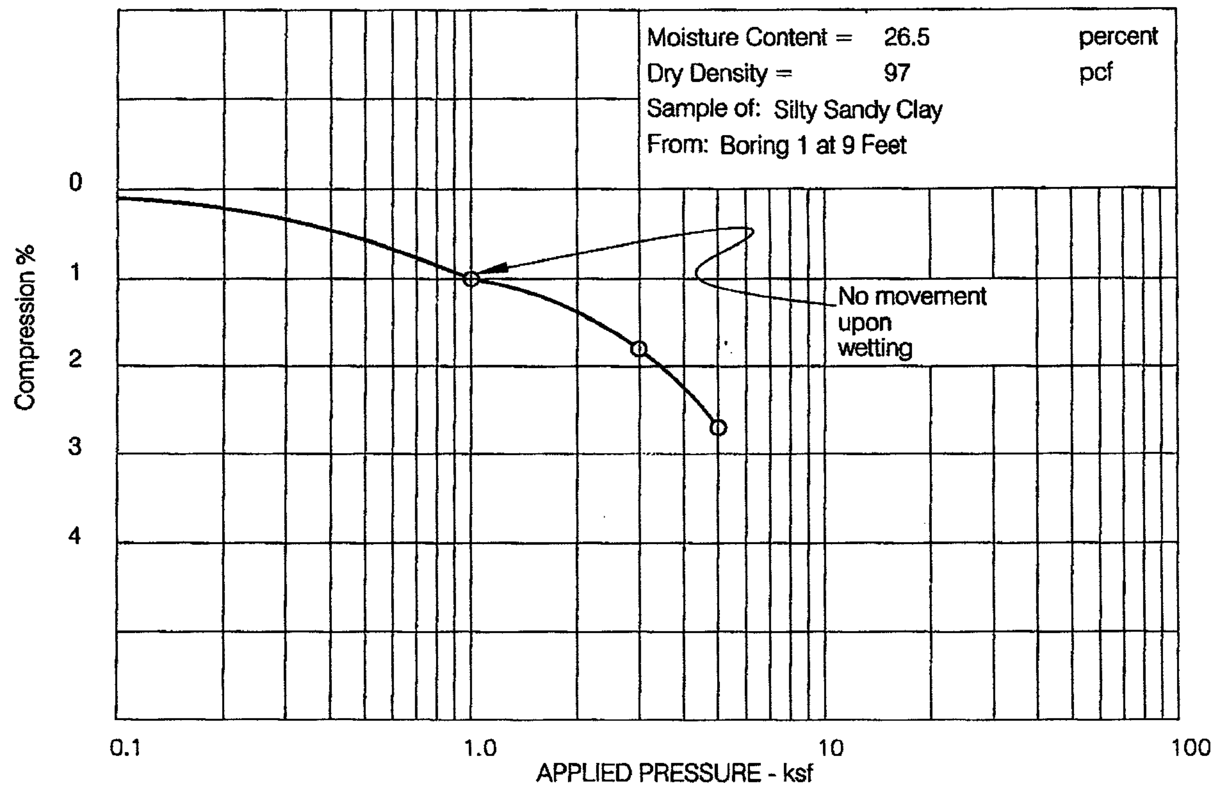
→ Depth at which boring had caved when checked on August 27, 2012.

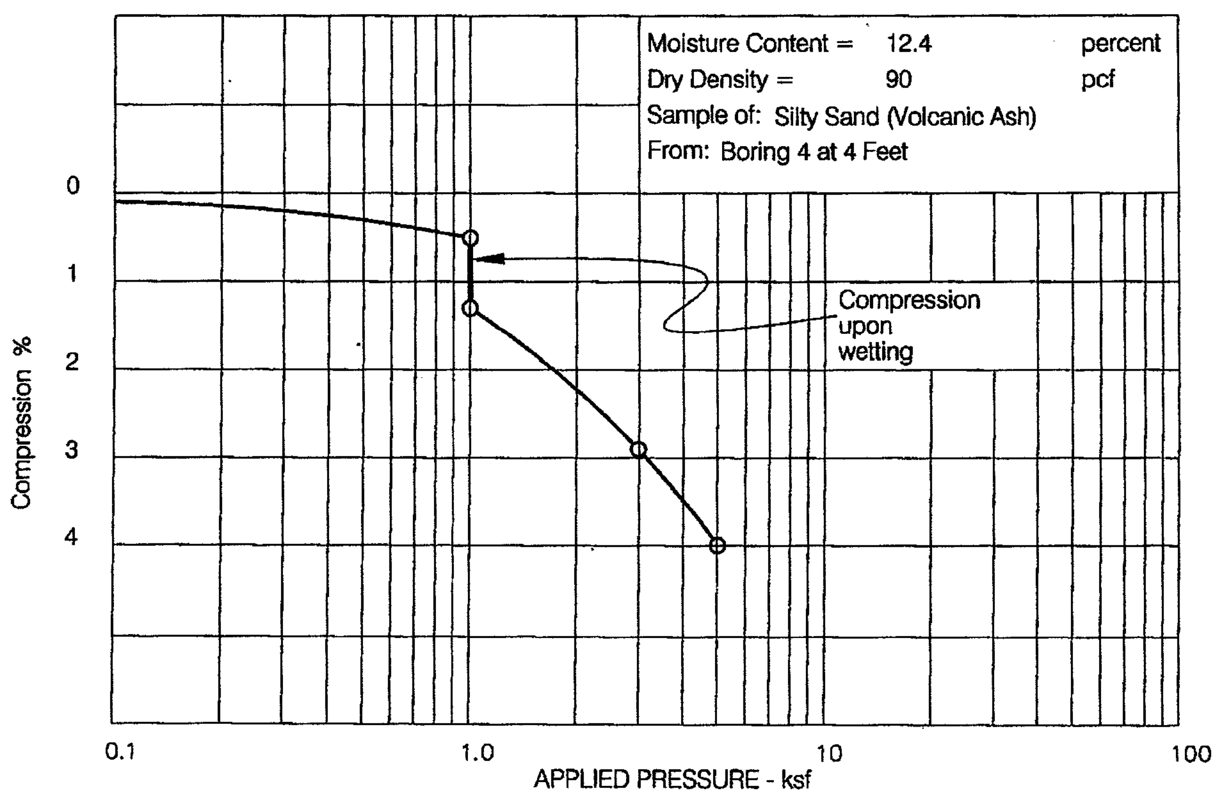
Practical drilling refusal.

NOTES:

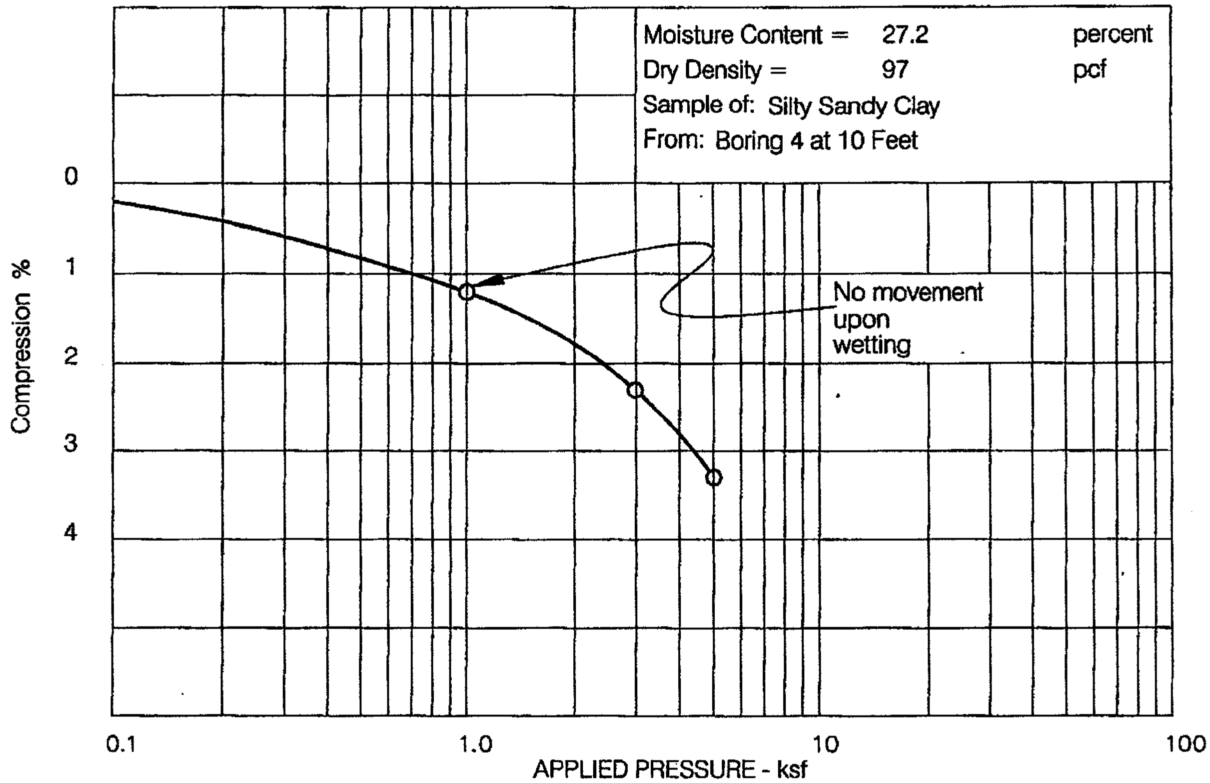
1. Exploratory borings were drilled on August 20, 21 and 22, 2012 with 4-inch diameter continuous flight power auger.
2. The exploratory boring locations were survey located by Eagle County.
3. Elevations of exploratory borings were interpolated from contours on the plan provided.
4. The exploratory boring locations and elevations should be considered accurate only to the degree implied by the method used.
5. The lines between materials shown on the exploratory boring logs represent the approximate boundaries between material types and transitions may be gradual.
6. Water level readings shown on the logs were made at the time and under the conditions indicated. Fluctuations in water level may occur with time.
7. Laboratory Testing Results:
   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)

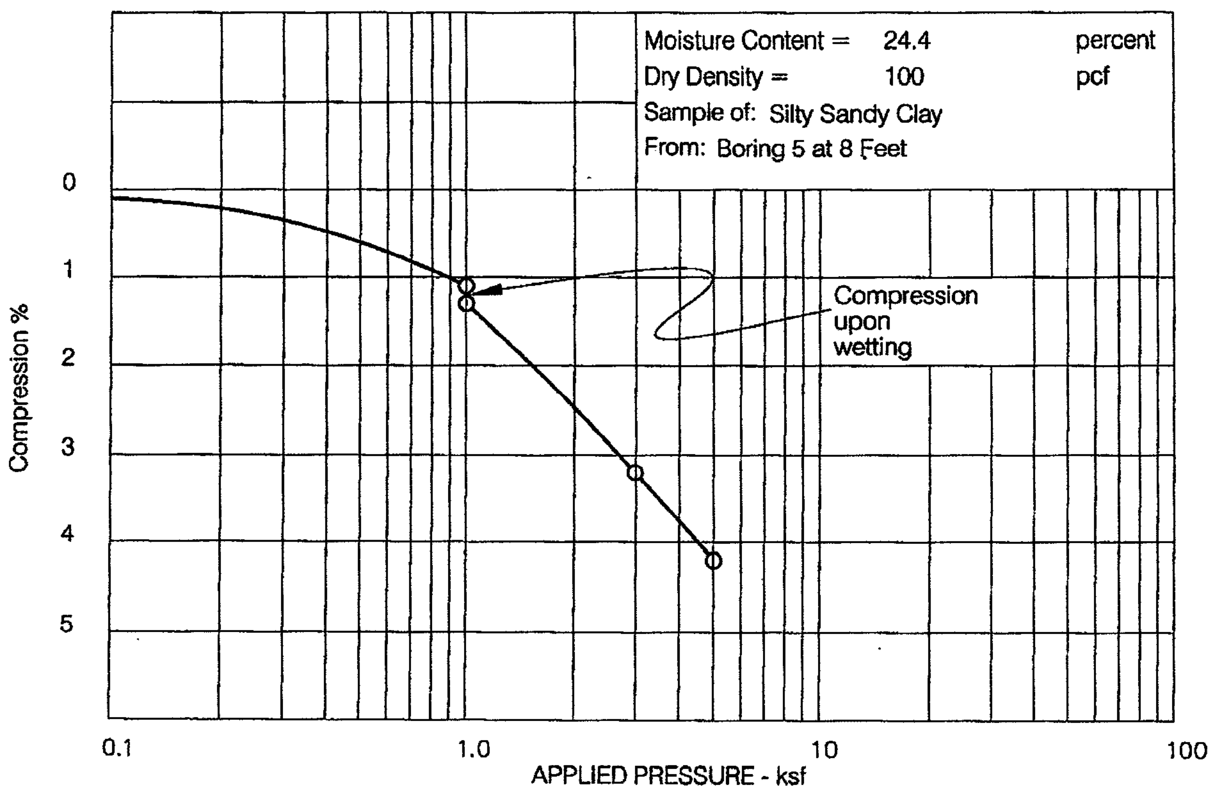
112 269B	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	LEGEND AND NOTES	Figure 4



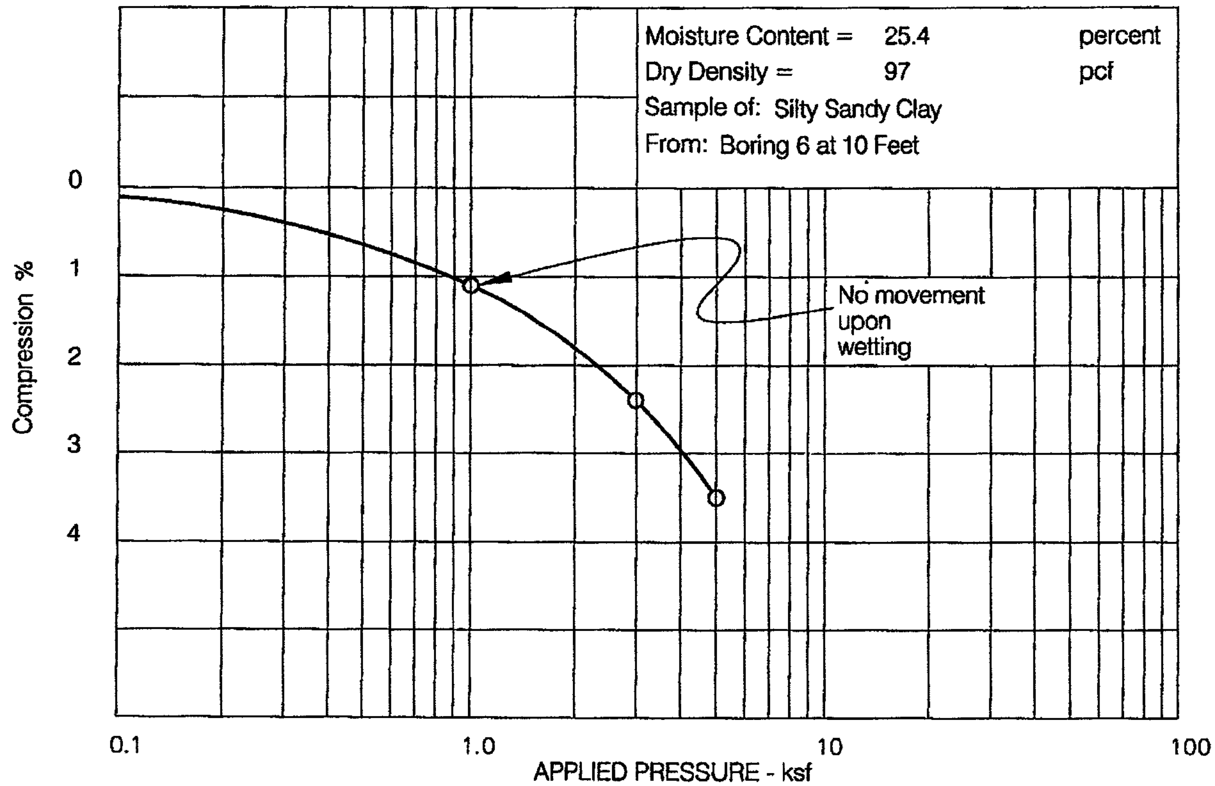


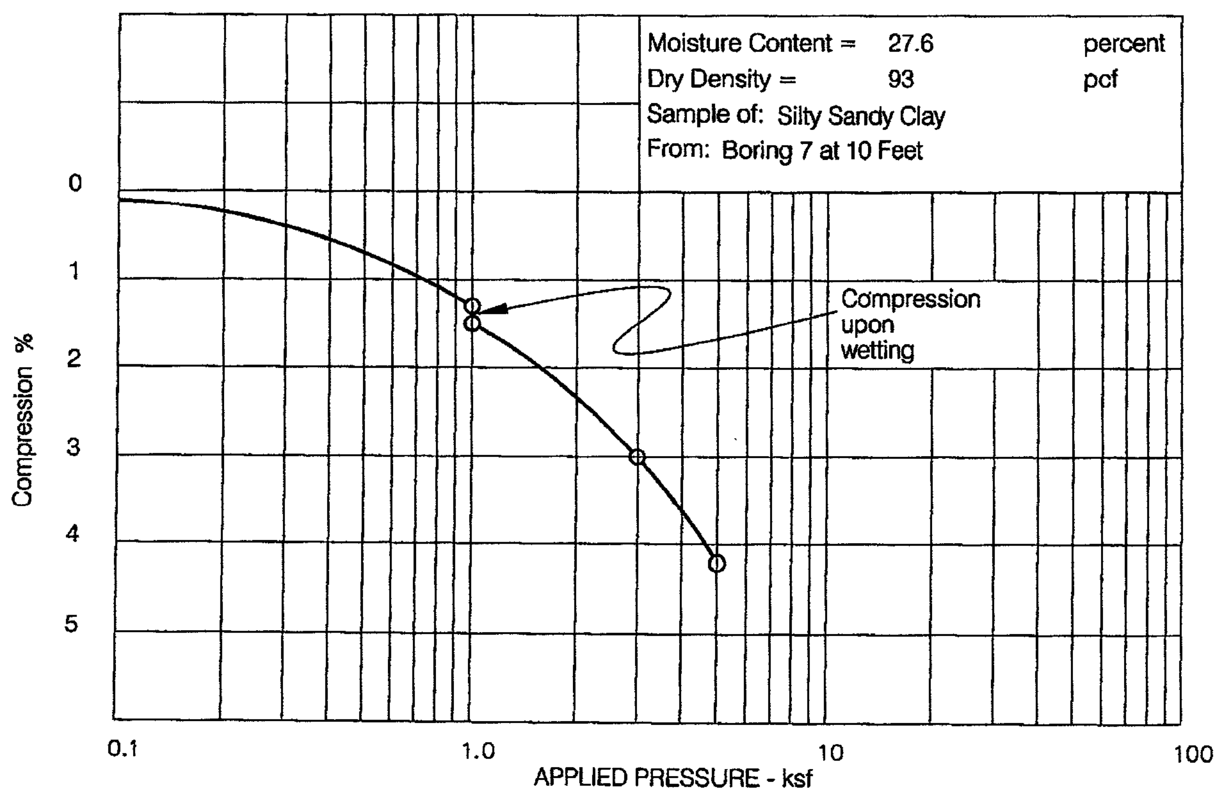
112 269B	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	SWELL-CONSOLIDATION TEST RESULTS	Figure 5



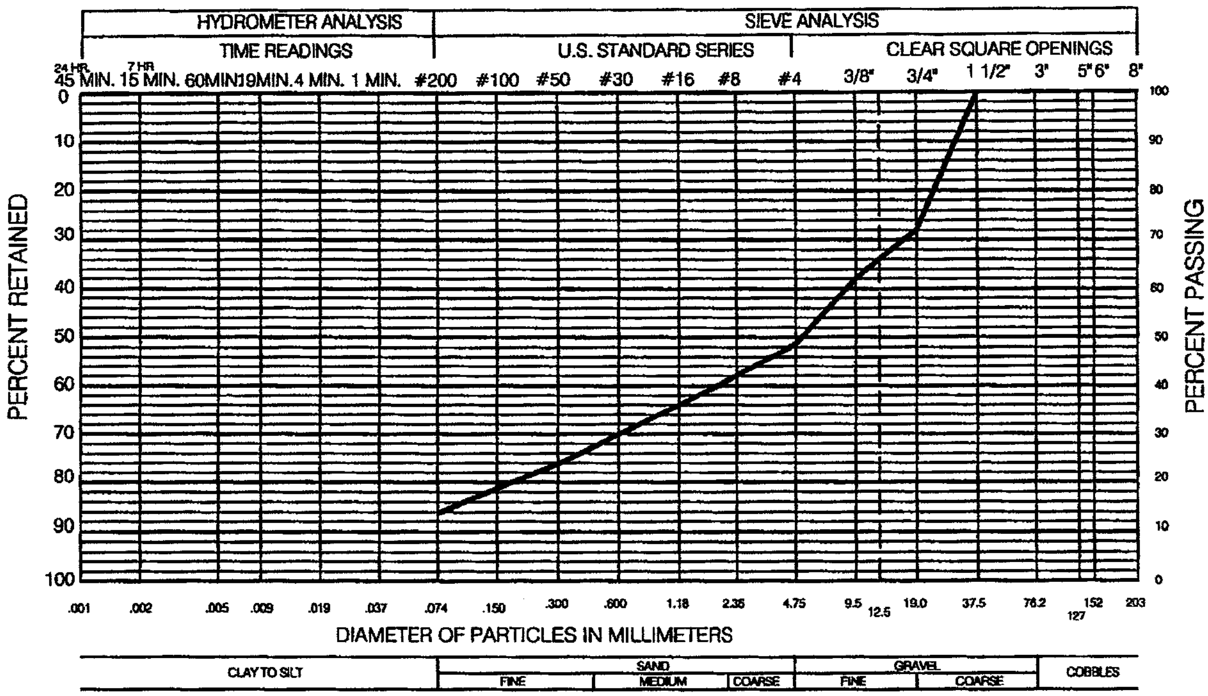


112 269B	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	SWELL-CONSOLIDATION TEST RESULTS	Figure 6





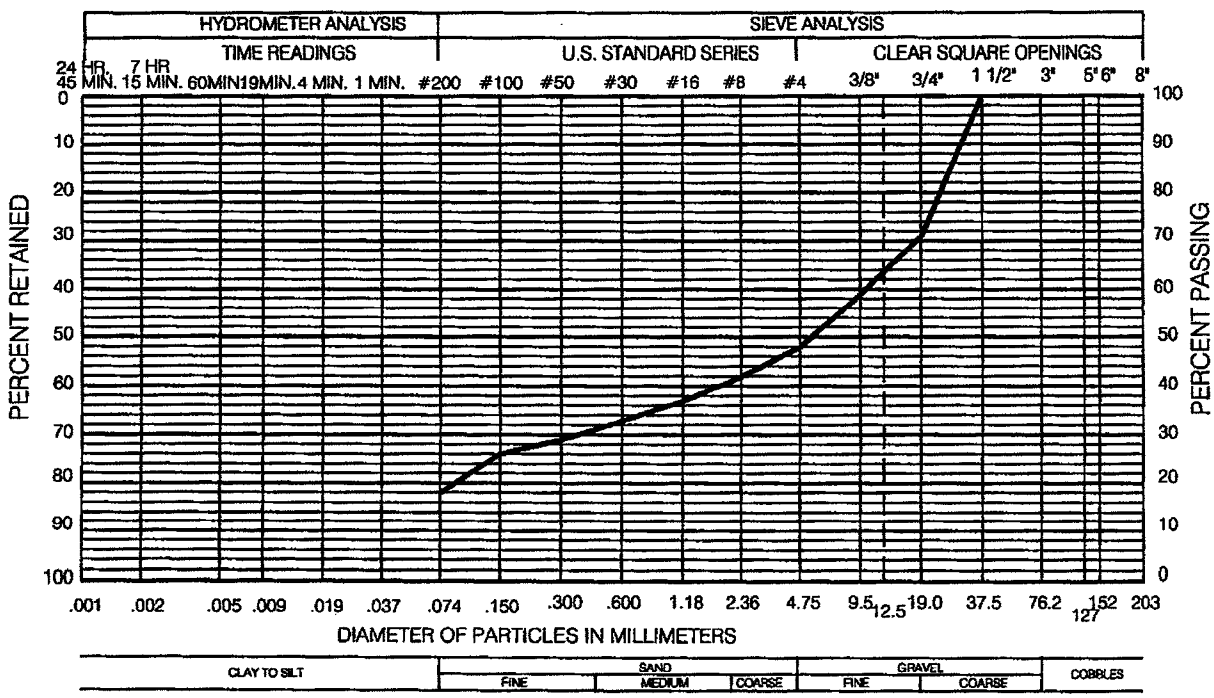
112 269B	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	SWELL-CONSOLIDATION TEST RESULTS	Figure 7



GRAVEL 51 % SAND 35 % SILT AND CLAY 14 %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF: Silty Sandy Gravel FROM: Boring 2 at 18 Feet

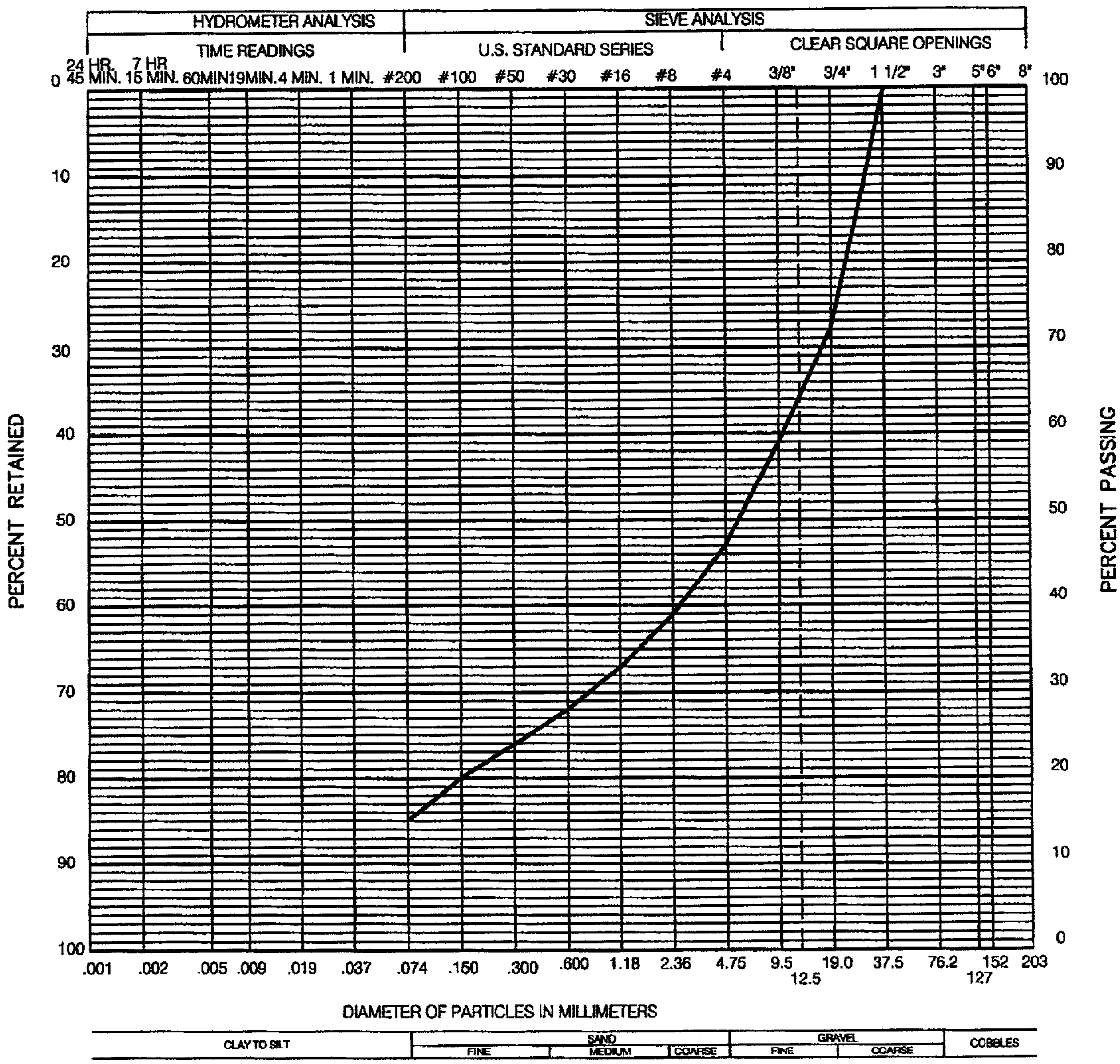


GRAVEL 52 % SAND 30 % SILT AND CLAY 18 %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF: Silty Sandy Gravel FROM: Boring 7 at 20 Feet

112 000A	HP Geotech Hepworth–Pawlak Geotechnical	GRADATION TEST RESULTS	Figure 8



GRAVEL 53 % SAND 32 % SILT AND CLAY 15 %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF: Silty Sandy Gravel FROM: Boring 9 at 30 Feet

112 269B	HP Geotech Hepworth–Pawlak Geotechnical	GRADATION TEST RESULTS	Figure 9

HEPWORTH-PAWLAK GEOTECHNICAL, INC.
TABLE 1
SUMMARY OF LABORATORY TEST RESULTS

Job No. 112 269B

SAMPLE LOCATION		NATURAL MOISTURE CONTENT	NATURAL DRY DENSITY	GRADATION		PERCENT PASSING NO. 200 SIEVE	ATTERBERG LIMITS		UNCONFINED COMPRESSIVE STRENGTH	AASHTO CLASSIFICATION	SOIL OR BEDROCK TYPE
BORING	DEPTH (ft)	(%)	(pcf)	GRAVEL (%)	SAND (%)		LIQUID LIMIT (%)	PLASTIC INDEX (%)			
1	4	19.1	98			85	32	12		A-6 (9)	Silty Sandy Clay
	9	26.5	97								Silty Sandy Clay
2	3	13.6	96			32					Silty Sand (Volcanic Ash)
	18			51	35	14					Silty Sandy Gravel
3	8	24.6	101			81	35	12	2,300	A-6 (9)	Silty Sandy Clay
4	4	12.4	90								Silty Sand (Volcanic Ash)
	10	27.2	97								Silty Sandy Clay
5	8	24.4	100								Silty Sandy Clay
	13	16.4	97			71	24	6	650	A-4(2)	Sandy Silt & Clay
6	10	25.4	97								Silty Sandy Clay
7	2	13.4	90			62	36	14		A-6 (7)	Silty Sandy Clay
	10	27.6	93								Silty Sandy Clay
	20			52	30	18					Silty Sandy Gravel

HEPWORTH-PAWLAK GEOTECHNICAL, INC.
TABLE 1
SUMMARY OF LABORATORY TEST RESULTS

SAMPLE LOCATION		NATURAL MOISTURE CONTENT	NATURAL DRY DENSITY	GRADATION		PERCENT PASSING NO. 200 SIEVE	ATTERBERG LIMITS		UNCONFINED COMPRESSIVE STRENGTH	AASHTO CLASSIFICATION	SOIL OR BEDROCK TYPE
BORING	DEPTH (ft)	(%)	(pcf)	GRAVEL (%)	SAND (%)		LIQUID LIMIT (%)	PLASTIC INDEX (%)			
8	0	15.9				80	29	6		A-4 (4)	Sandy Silt and Clay (Fill)
9	5	23.3				65	30	4		A-4 (2)	Sandy Silt & Clay (Fill)
	20	25.9	97			82	29	11		A-6 (8)	Silty Sandy Clay
	30			53	32	15					Silty Sandy Gravel