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PRELIMINARY GEOTECHNICAL STUDY
PROPOSED HAYMEADOW PHASE A1 DEVELOPMENT
AND WATER STORAGE TANK
BRUSH CREEK ROAD
EAGLE, COLORADO

JOB NO. 113 097A

AUGUST 14, 2013

PREPARED FOR:

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PURPOSE AND SCOPE OF STUDY

This report presents the results of a preliminary geotechnical study for the proposed Haymeadow Development Phase A1 and water storage tank, Brush Creek Road, Eagle, Colorado. The project site is shown on Figure 1. The purpose of the study was to develop recommendations for site grading, foundation and pavement section design. The study was conducted as part of our proposal for geotechnical engineering services to Abrika Properties, LLC dated April 10, 2013.

A field exploration program consisting of exploratory borings was conducted to obtain information on the subsurface conditions. Samples of the subsoils and bedrock 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 recommendations for site grading, pavement section and infrastructure including the water storage tank, as well as preliminary recommendations for foundation types, depths and allowable pressures for the proposed building and bridge foundations. This report summarizes the data obtained during this study and presents our conclusions, design recommendations and other geotechnical engineering considerations based on the proposed construction and the subsurface conditions encountered.

BACKGROUND INFORMATION

Hepworth-Pawlak Geotechnical performed a geologic site assessment of the Haymeadow property, and submitted our findings in a report dated August 14, 2013. We also recently reviewed the debris flow hazards at the site identified in previous geologic studies and provided recommendations for the design of the debris flow mitigation in reports dated June 12 and July 11, 2013, submitted under our Job No. 113 097A. Additionally, we performed a preliminary geotechnical study for the Adam's Rib Ranch PUD, which included the Haymeadow property, and submitted our findings in a report dated July 30, 1998, Job No. 298 141.

PROPOSED CONSTRUCTION

The site is planned to be developed as residential subdivisions and associated infrastructure in phases. The current development will consist of Phase A1, located in the western and southwestern portions of the property, and the water storage tank located in the extreme northeastern party of the property, see Figures 1 and 1A.

The subdivision residences will include single family and multi-family structures. Some of the buildings will have basement levels where feasible. There will also be a community park area with a gazebo and other outbuildings. There will be two shallow ponds in the lower part of the site that are planned to be lined. As part of the Phase A1 infrastructure, there will be a roadway crossing the large drainage ditch through the middle portion of the site along the proposed Sylvan Lake Road. In general, grading for the subdivision roadways will consist of minor cuts and fills. There will be a pedestrian path along the north side of the property constructed on a deeper fill berm as part of the debris flow mitigation.

The water storage tank will be an above ground steel structure about 90 feet in diameter. Considerable grading will be need for the tank site and the access road to the tank site. The grading for the access road will include cuts and fills from about 10 to 30 feet deep. Considerable grading consisting of relatively deep cuts up to about 50 to 60 feet will be needed to level the tank site. The cuts and fills are planned to be graded at 1½ horizontal to 1 vertical.

If locations or grading plans change significantly from those described above, we should be notified to re-evaluate the recommendations contained in this report.

SITE CONDITIONS

The subdivision areas of the site are primarily irrigated hay fields vegetated with grass and weeds, and are vacant property. The ground surface in Phase A1 is primarily strongly sloping down to the south-southwest toward Brush Creek which borders the southwestern side of the property. There is a large ditch trending north to south along the east side of the Phase A1 development. The water storage tank site is located in steep hilly terrain.

GEOLOGIC CONDIITONS

The geologic conditions at the site were addressed in our geologic site assessment report dated August 14, 2013, Job No. 113 097A. Major potential geologic hazards that could affect the development include the potential for sinkhole development, debris flows and floods, and the potential for construction induced slope instability. The debris flow hazard and mitigation measures have been discussed in our previous reports dated June 12 and July 11, 2013. Recommendations for site grading, including the tank site and access road, are discussed in this report. Soft foundation and subgrade soils are also discussed in this report.

FIELD EXPLORATION

The field exploration for the project was conducted between April 29 and May 21, 2013. Fifteen exploratory borings were drilled at the locations shown on Figures 1 and 1A to evaluate the subsurface conditions. Borings 1 through 12 were drilled in the A1 Phase subdivision area and Borings 13 through 15 were drilled at the water storage tank site. The borings were advanced with 4 inch diameter continuous flight augers powered by a track-mounted CME 45 drill rig. The track rig was needed due to the soft and wet field areas, and the steep terrain at the tank site. The borings were logged by a representative of Hepworth-Pawlak Geotechnical, Inc.

Samples of the subsoils were taken with 1⅜ inch and 2 inch I.D. spoon samplers. The samplers were driven into the subsurface materials 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 through 5. The samples were returned to our laboratory for review by the project engineer and testing.

Slotted PVC pipe was installed in most of the borings drilled in the subdivision area to allow monitoring of the groundwater levels. Borings that were finished with slotted pipe and depths the pipe were installed are shown on the boring logs.

SUBSURFACE CONDITIONS

Graphic logs of the subsurface conditions encountered at the site are shown on Figures 2 through 5. The subsoils encountered in the Phase A1 subdivision, Borings 1 through 12, encountered 1 to 1½ feet of organic topsoil overlying soft to medium stiff, sandy clay and silt underlain at depths from about 7 to 36 feet by relatively dense, silty sandy gravel and cobbles with small boulders. Between depths of about 4 to 5 feet in Borings 4 and 5, a layer of very silty sand (Volcanic Ash) was encountered. The borings at the tank site, Borings 13 through 15, below about ½ foot of topsoil, encountered about 1½ to 3 feet of medium dense, silty clayey sandy gravel with cobbles (consisting primarily of angular shale fragments) overlying claystone/siltstone bedrock of the Eagle Valley Formation. The claystone/siltstone was weathered in the upper portion becoming hard and less weathered with depth. Drilling in the dense granular soils (at Borings 1 through 12) with auger equipment was difficult due to the cobbles and boulders and drilling refusal was encountered in the deposit. Drilling refusal was also encountered with depth in the claystone/siltstone bedrock (Borings 13 through 15) due to its hardness and possible cemented layers.

Laboratory testing performed on samples obtained from the borings included natural moisture content and density, gradation analyses, and Atterberg limits. Results of swell-consolidation testing performed on relatively undisturbed drive samples of the clay and silt soils, presented on Figures 7 through 14, indicate moderate to relatively high compressibility under conditions of loading and wetting. Results of swell-consolidation testing performed on relatively undisturbed drive samples of the claystone/siltstone bedrock, presented on Figures 15 and 16, generally indicate low to moderate compressibility under conditions of loading and wetting. One of the claystone/siltstone samples (Boring 15 at 18 feet) showed a low swell potential when wetted. Results of gradation analyses performed on small diameter drive samples (minus 1½ inch fraction) of the coarse granular subsoils are shown on Figure 17 and 18. The laboratory testing is summarized in Table 1. We also performed chemical testing on samples from the borings at the tank site with the results provided in Table 2.

Free water was encountered in Borings 1, 2, 5 and 7 through 12 during our field exploration with the water level measurements shown on the boring logs. No free water was encountered in Borings 3, 4 and 6, and 13 through 15 drilled at the tank site. Subsequent water level readings were made by the client in the borings with PVC pipe as irrigation season progressed with the results provided to us shown in Appendix A. The subsoils were moist to very moist and wet with depth in some of the borings. The claystone/siltstone bedrock was typically slightly moist.

GEOTECHNICAL RECOMMENDATIONS

Development of the site as generally proposed is feasible based on geotechnical considerations. Several aspects of the project will require special attention such as soft subgrade soils for roadway and foundation construction, shallow groundwater in subdivision areas of the project, and potential instability of deep cuts for site grading and at the tank site.

SITE GRADING

The risk of construction-induced slope instability at the site appears low provided the cut and fill depths are limited to about 12 to 15 feet and sloped back to a stable grade or retained. In general, cut and fill slopes can be graded no steeper than 1½ horizontal to 1 vertical. At the tank site, there is a risk of instability for the proposed deep cuts primarily relating to possible adverse bedding and joint conditions of the bedrock.

All structural fills such as for roadway construction, and overlot grading should be compacted to at least 95% of the maximum standard Proctor density (SPD) at a moisture content within about 2% of optimum. Fills deeper than about 10 feet will have increased settlement potential even when properly placed and compacted. In the deeper fill areas, increasing compaction to 100% SPD at a moisture content within about 2% of optimum could be done to help mitigate the settlement potential. Use of select granular materials in the deeper fill areas would also help limit settlements. 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% SPD. The fill should be benched horizontally into the slopes that exceed 20% grade. The onsite soils when properly processed can be used as fill for the roadway and overlot grading. Most of the on-site soils are above optimum moisture content and will require drying prior to their placement as structural fill. The claystone/siltstone bedrock can also be used as structural provided it is well broken into a soil like material.

Soft subgrade conditions should be expected in most areas and may require stabilization prior to fill placement or pavement construction. The stabilization can probably be done by subexcavation of 2 to 3 feet and replacement with imported coarse granular soils such as “single pass” pit-run sand and gravel or CDOT Class 2 road base. Use of a triaxial geo-grid, such as Tensar TX 140 or TX 160 can be used and should act to reduce the depth of subexcavation and replacement with coarse granular soils. In some areas it may be feasible to scarify the subgrade and allow it to dry to reduce the need for stabilization.

PAVEMENT SECTION THICKNESS

We understand that asphalt pavement is proposed for the streets. Traffic loadings for the collector and subdivision streets have not been provided but are expected to be typical of the area. The subgrade soils encountered at the site are typically low to medium plasticity sandy silty clay and sand clay and silt with AASHTO classifications of A-6 and A-4 with Group Indices of 0 to 14 on the samples tested, see Table 1. The soils are considered a relatively poor support for pavement sections. We estimate a Hveem stabilometer 'R' value of about 5 for these soils.

Based on our experience in the area, an 18 kip equivalent daily load application (EDLA) of about 20 was assumed for the collector streets (Sylvan Lake Road and the main subdivision road) and an EDLA of about 10 was assumed for the residential streets. Construction traffic could increase the assumed EDLA. Using a Regional Factor of 2.0, a serviceability index of 2.5 for the collector streets and 2.0 for the residential streets, an 'R' value of about 5 and the above estimated EDLA's, we recommend the following alternate minimum pavement sections for the collector and residential streets.

Street Location	Alternative Number	Asphalt (inches)	Base Course (inches)	Sub-base (inches)
Collectors	1	5	8	-
Collectors	2	4	12	-
Collectors	3	4	4	12
Collectors	4	4	6	8
Residential	1	4	8	-
Residential	2	4	4	8

The silty clay and clayey silt soils are considered moderately to highly susceptible to frost heave. Providing 8 to 12 inches or more of granular sub-base material below the pavement section would help limit the frost heave potential.

The asphalt should be a batched hot mix, approved by the engineer and placed and compacted to the project specifications. The base course and sub-base should meet CDOT Class 6 and Class 2 specifications, respectively. All base course, sub-base and required subgrade fill should be compacted to at least 95% of the maximum standard Proctor density at a moisture content within about 2% of optimum. Concrete pavement sections should consist of a minimum 6 inches of concrete on 4 inches of Class 5 or 6 base course on stable subgrade soils. The concrete should meet CDOT Class P or D specifications, and be air entrained.

The pavement subgrade areas should be proof-rolled. 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 are better known, we should review our pavement section recommendations.

BUILDING FOUNDATIONS AND BASEMENTS

The fine-grained soils at the site possess low bearing capacity and moderate to moderately high settlement potential. Lightly loaded spread footings bearing on the natural subsoils may be feasible for support of lightly loaded residential buildings provided some settlement is tolerable. Providing 3 feet of imported granular structural fill below footings would act to increase the bearing capacity and reduce the settlement potential. Placing the building foundations on the natural coarse granular soils would provide a relatively low risk of settlement and building distress. Helical piers or screw piles could also be used to achieve foundation bearing on the underlying coarse granular soils.

We expect that footings bearing on the natural fine grained soils can be sized for an allowable bearing pressure in the range of 1,000 to 1,500 psf with some risk of settlement. For footings bearing on a minimum 3 feet of structural fill on the fine grained soils, an allowable soil bearing pressure of 2,000 psf should be feasible. For footings bearing entirely on the natural coarse granular soils an allowable soil bearing pressure in the range of 2,500 to 3,500 psf should be feasible. The footings should have a minimum depth of 48 inches for frost protection. Foundation walls should be heavily reinforced to withstand the effects of some differential movement and to resist lateral earth loadings when acting as retaining structures. Foundation drains should be provided around below grade construction (and behind retaining walls) to prevent buildup of hydrostatic pressures and wetting of the below grade areas.

Groundwater was encountered in most of the borings in the A1 Phase with considerable rise during summer irrigation season. In some areas, typical depth basement levels should be feasible. Basement finish floor elevations should be at least 3 feet above high water level to minimize the risk of wetting of the basement level construction. Subsoil studies should be done for the individual buildings to evaluate the site specific foundation soil bearing and groundwater level conditions.

BRIDGE FOUNDATION

The proposed Sylvan Lake Road bridge crossing of the large ditch is located in the area of Boring 9. The subsoils encountered in Boring 9 consisted of about 22 feet of medium stiff, sandy clay and silt underlain by dense, coarse granular soils. Recent groundwater level readings indicate free water level in Boring 9 at a depth of 9½ feet depth on July 18. The upper soils are compressible and the bridge should be founded on steel H-piles driven to refusal in the dense, coarse granular soils. The piles should develop their structural capacity when driven to refusal. HP 12x53 or HP 12x74 piles should be feasible at this site. The piles should be provided with factory manufactured tip protection. Based on our experience in the area, some long term corrosion of the steel piling should be expected and some reduction in load should be considered due to the corrosion potential. Once plans for the bridge have been developed we can provide additional recommendations as needed for the design.

WATER STORAGE TANK

Based on the relatively deep proposed cuts at the tank site, very hard claystone/siltstone bedrock of the Eagle Valley Formation is expected at excavation subgrade. An allowable bearing pressure of 4,000 psf can be used for the foundation design for bearing on the bedrock. The weight of water in the tank can be neglected in the foundation design. A minimum 6 inches of CDOT Class 5 or 6 base course compacted to at least 98% of standard Proctor density should be provided below the tank and extend at least 10 feet beyond the perimeter of the tank. There could be some differential movement due to possible variable bearing conditions and if the claystone/siltstone subgrade becomes wetted. The surface grading around the tank should divert surface water runoff well away from the tank foundation materials to reduce the potential for wetting.

There is a risk of construction induced slope instability at the site due to the proposed deep cuts up to about 60 feet. The risk is primarily if the bedrock has adverse bedding and joints to the cuts. It should be feasible to grade the cut slopes no steeper than 1½ horizontal to 1 vertical with this risk. We should observe the cut slopes as they progress during construction to determine if adverse bedrock conditions exists and if mitigation, such as shotcreteing, rock bolting or flatter slope grades are needed.

Chemical testing was performed on samples from the borings at the tank site with the results provided in Table 2. Based on the test results and our experience in the area, the soils and bedrock are expected to be at least moderately aggressive toward buried steel. Concrete containing Type I/II cement with fly ash should be used for concrete exposed to the on-site soils and bedrock. The concrete should be a relatively rich mix and be air entrained.

PONDS

The ponds will be located in the area of Boring 8. Boring 8 encountered about 13 feet of sandy clay and silt soils over dense, coarse granular soils. Recent groundwater level readings indicate free water at a depth of 6½ feet in Boring 8 on July 18. We expect the pond bottoms will be excavated near to below the high seasonal groundwater and within the clay and silt soils. The fine grained soils are expected to have low permeability and treatment with a bentonite or other similar material may be adequate for the liner. The clay and silt soils will probably need to be dry of optimum moisture content for the bentonite mixing.

A drain system may be feasible at the pond sites to intercept and lower the groundwater level and allow use of a synthetic liner for the pond bottoms and sides. Otherwise a synthetic liner could be damaged under shallow groundwater conditions. Typically 1 to 2 feet of cobbley material is placed on synthetic liners to help hold the liner in place.

Any fill for construction of the ponds, should consist of the on-site clay and silt soils compacted to at least 95% of the maximum standard Proctor density near optimum moisture content. Prior to fill placement, the subgrade should be carefully prepared by removing all vegetation and topsoil and compacting to at least 95% of the maximum standard Proctor density. Cut and fill slopes for the ponds should be no steeper than 2½ horizontal to 1 vertical and protected against erosion by revegetation or other means.

We should review the plans for the ponds and perform additional analyses as needed prior to construction.

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 Figures 1 and 1A, 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.

COLORADO REGISTERED DAVID A. YOUNG 32-216 PROFESSIONAL ENGINEER 8-16-13

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Reviewed by:

Steven L. Pawlak

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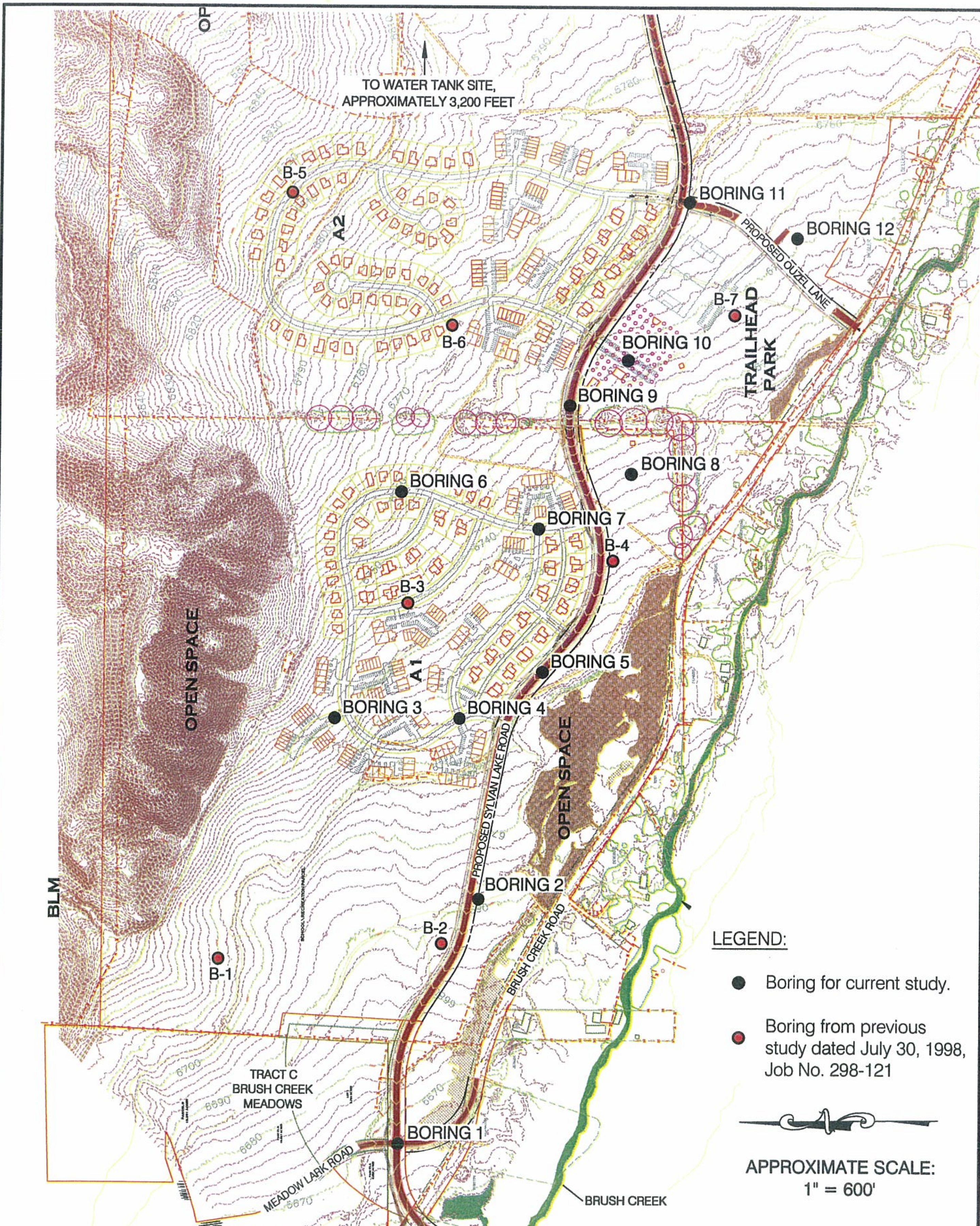
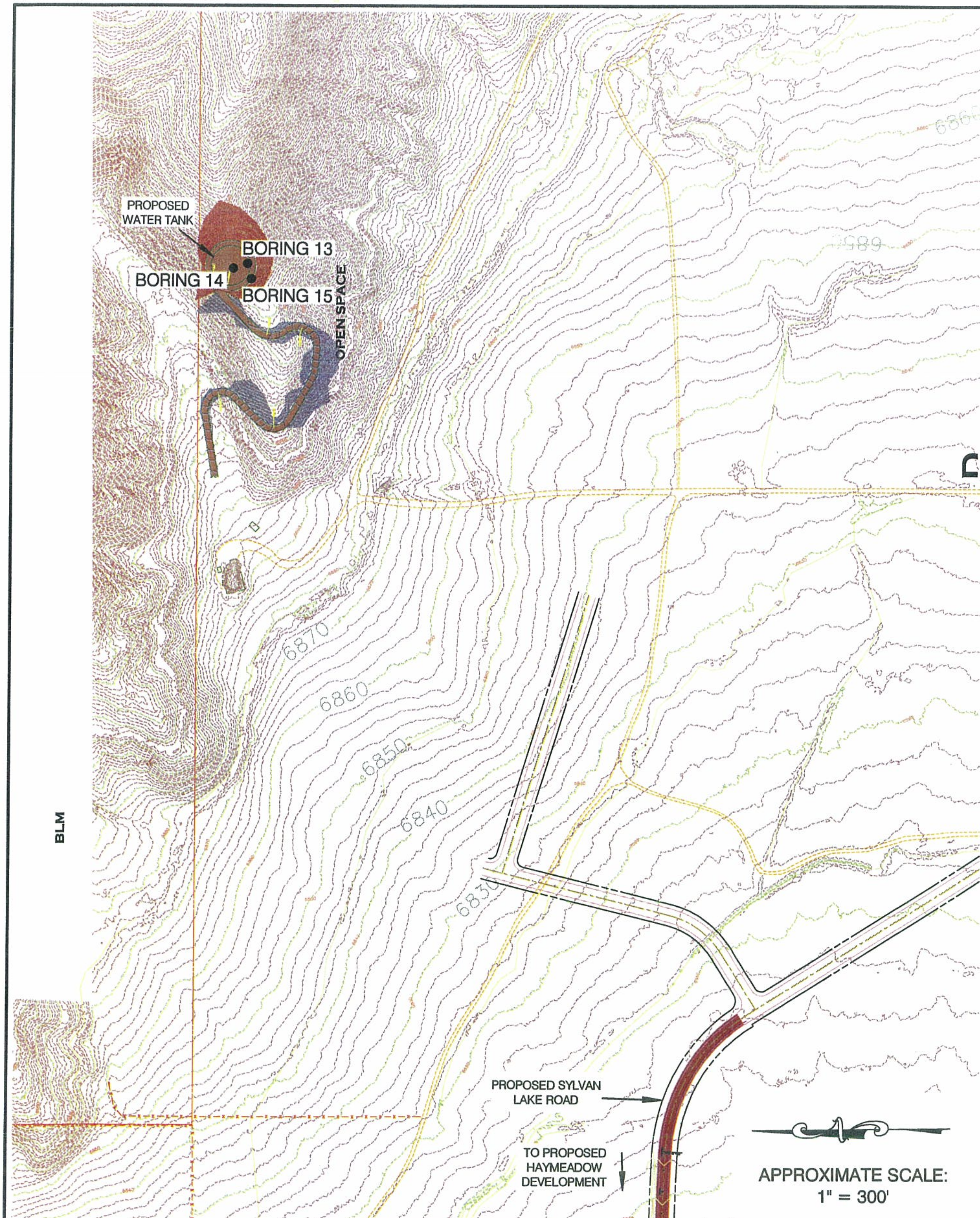
DAY/ksw

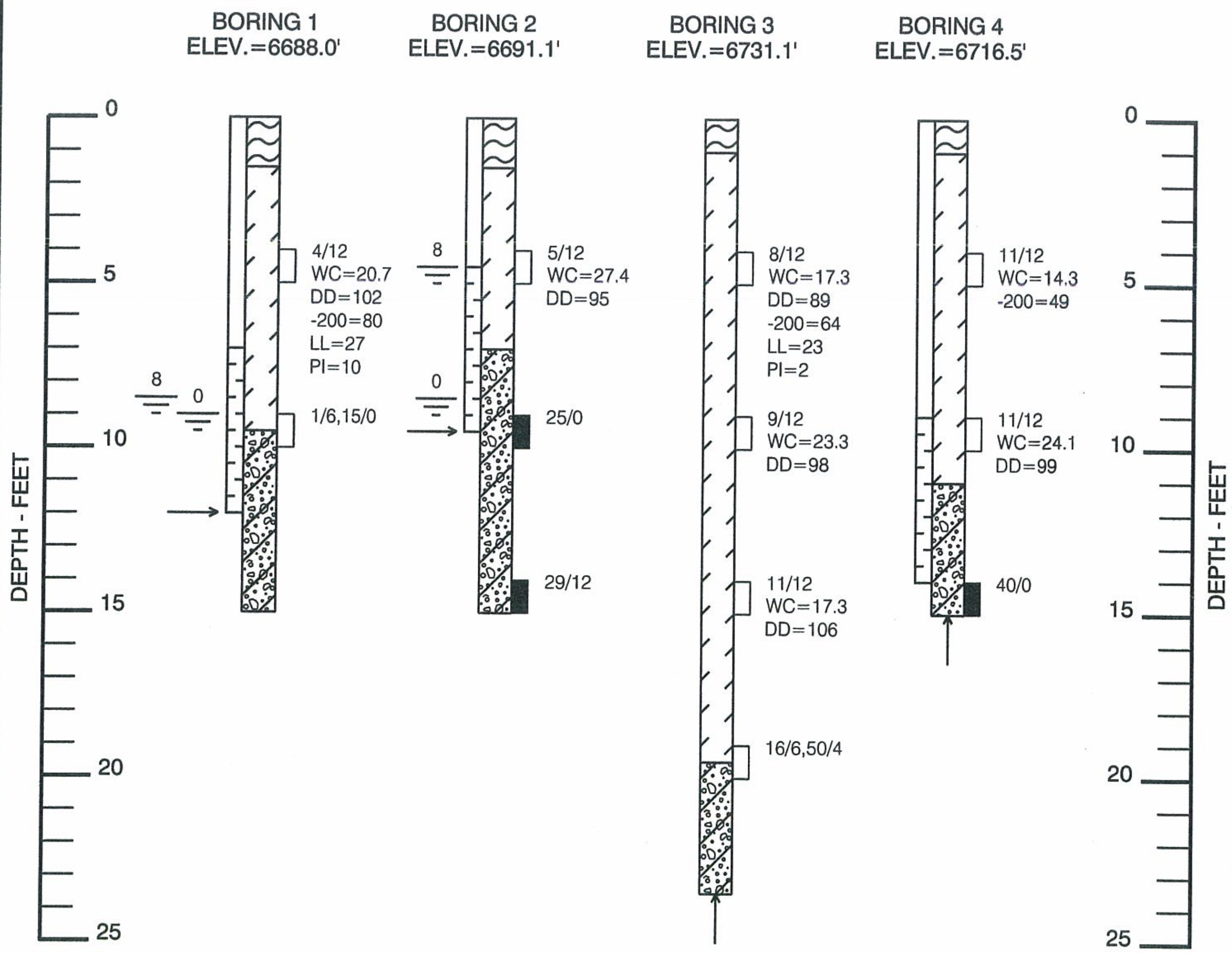
cc: Alpine Engineering – Gary Brooks (brooks@alpinecivil.com)



113 097A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	LOCATIONS OF EXPLORATORY BORINGS HAYMEADOW DEVELOPMENT PHASE A1	FIGURE 1

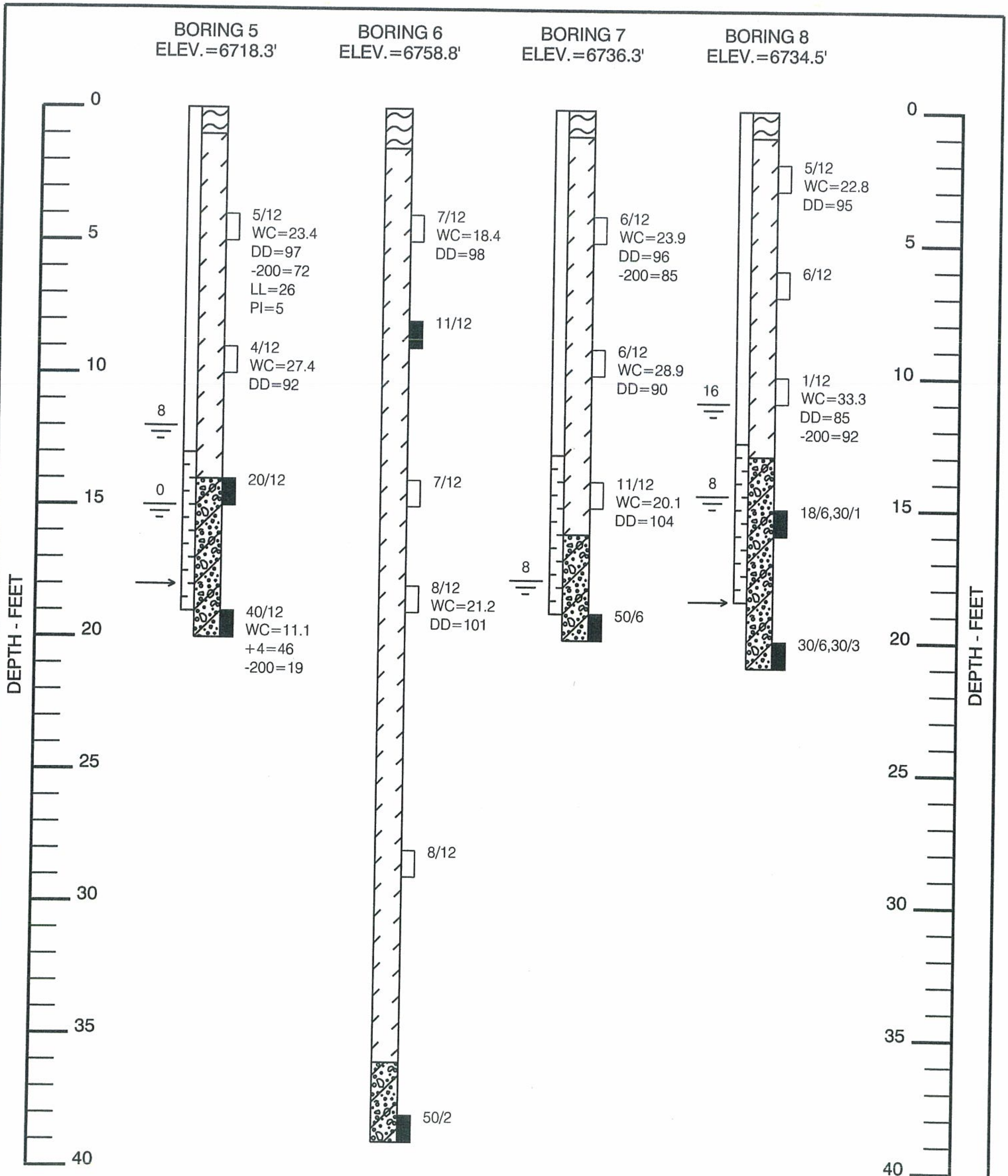


113 097A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	LOCATIONS OF EXPLORATORY BORINGS HAYMEADOW WATER STORAGE TANK	FIGURE 1A



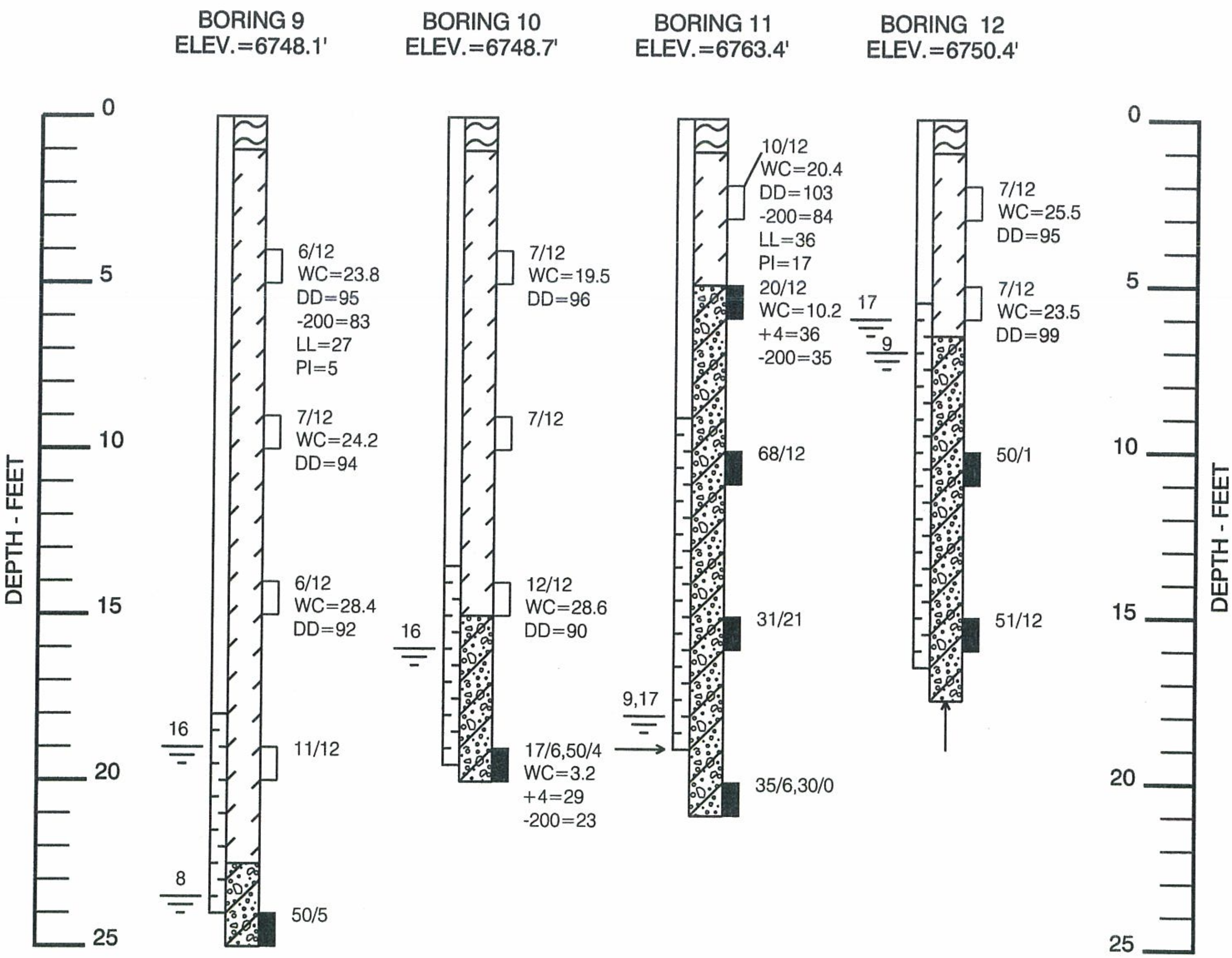
NOTE: Explanation of symbols is shown on Figure 6.

113 097A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	LOGS OF EXPLORATORY BORINGS	FIGURE 2



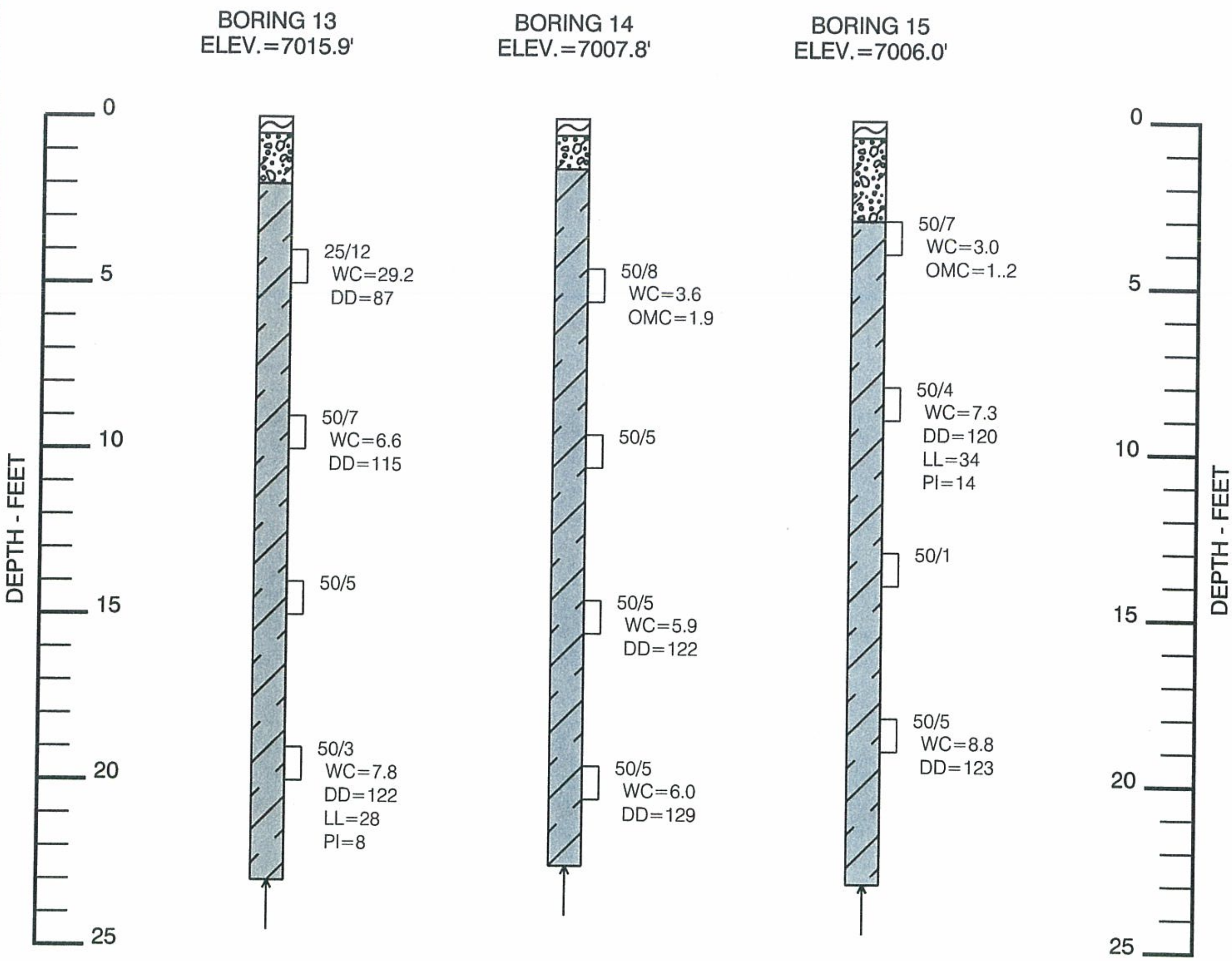
NOTE: Explanation of symbols is shown on Figure 6.

113 097A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	LOGS OF EXPLORATORY BORINGS	FIGURE 3



NOTE: Explanation of symbols is shown on Figure 6.

113 097A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	LOGS OF EXPLORATORY BORINGS	FIGURE 4



NOTE: Explanation of symbols is shown on Figure 6.

113 097A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	LOGS OF EXPLORATORY BORINGS	FIGURE 5

LEGEND:

TOPSOIL; organic sandy silt and clay, moist, dark brown.

CLAY AND SILT(CL-ML); sandy to occasionally very sandy, soft to medium stiff, slightly moist to very moist, brown, slightly calcareous and/or gypsiferous, low to medium plasticity. Silty Sand (Volcanic Ash) encountered in Borings 4 and 5 at about 4 to 5 feet depth.

GRAVEL AND COBBLES (GM); with boulders, silty, sandy, occasionally clayey, some sand layers or pockets, medium dense to dense, slightly moist to moist and becoming wet below water table, brown to red-brown, typically non-plastic fines, rocks are primarily sub-rounded to rounded.

GRAVEL (GC-GM); with cobbles, sandy, clayey, silty, medium dense, slightly moist, red-brown, low to non-plastic fines, rocks are primarily angular shale fragments.

CLAYSTONE/SILTSTONE BEDROCK; weathered to very hard with depth, slightly moist, red-brown to grey, occasionally gypsiferous. Eagle Valley Formation.

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.

4/12 Drive sample blow count; indicates that 4 blows of 140 pound hammer falling 30 inches were required to drive the California or SPT sampler 12 inches.

Practical drilling refusal.

0,8 Free water depth measured in boring and number of days following drilling measurement was taken.

Depth boring caved following drilling.

Indicates 2-inch diameter hand slotted PVC pipe installed in borehole to the depth shown on the log.

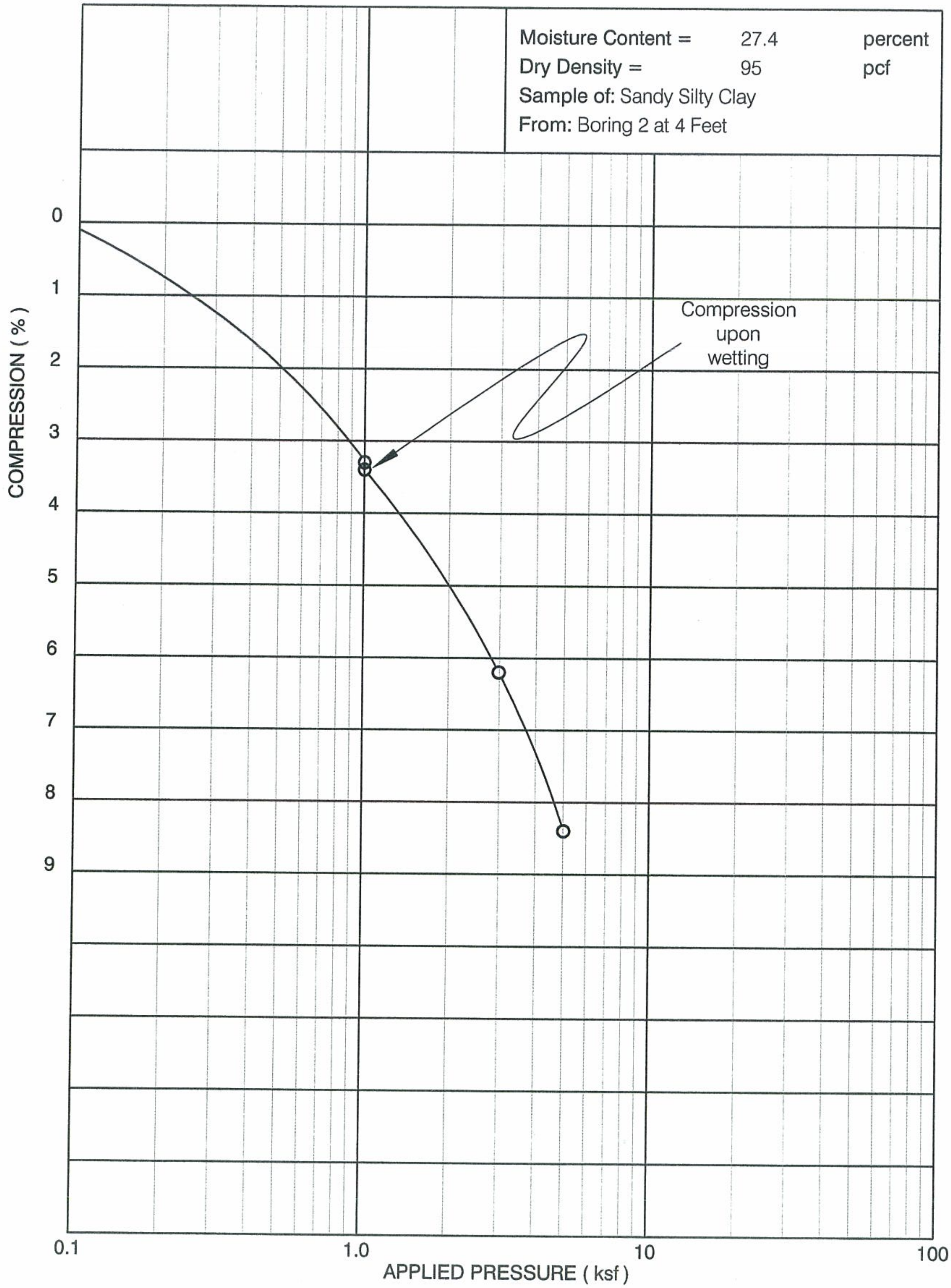
NOTES:

1. Exploratory borings were drilled between April 29 and May 21, 2013 with 4-inch diameter continuous flight power auger.
2. Locations and elevations of exploratory borings were provided by Archibeque Land Consulting, Ltd.
3. The lines between materials shown on the exploratory boring logs represent the approximate boundaries between material types and transitions may be gradual.
4. Water level readings shown on the logs were made at the time and under the conditions indicated. See Appendix A for summary of water level readings provided by others. No free water was encountered in Borings 13-15, located at the tank site. Fluctuations in water level may occur with time.
5. Laboratory Testing Results:

WC = Water Content (%)
DD = Dry Density (pcf)
+4 = Percent retained on the No. 4 sieve
-200 = Percent passing No. 200 sieve

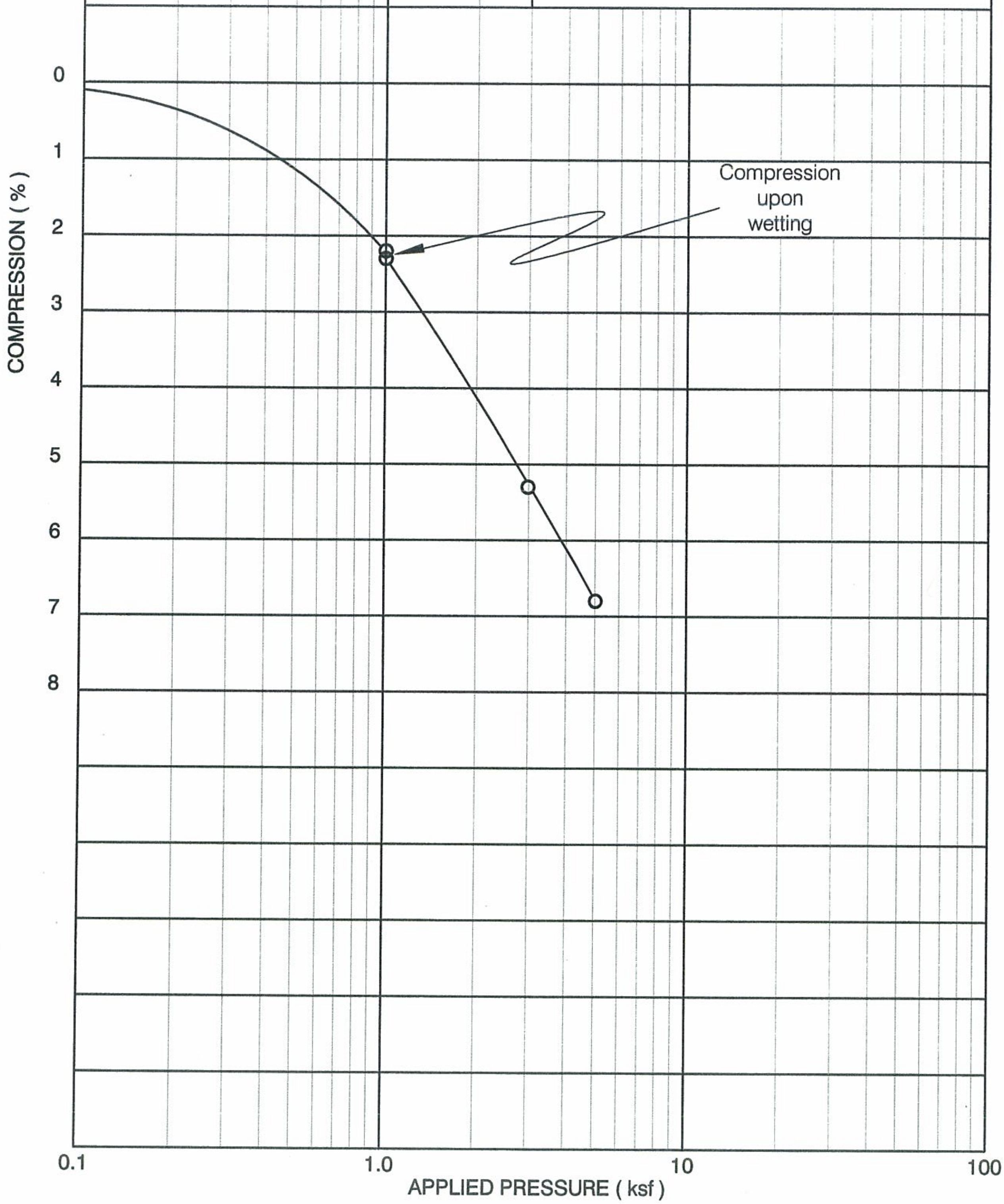
LL = Liquid Limit (%)
PI = Plasticity Index (%)
OMC = Organic Matter Content (%)

113 097A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	LEGEND AND NOTES	FIGURE 6

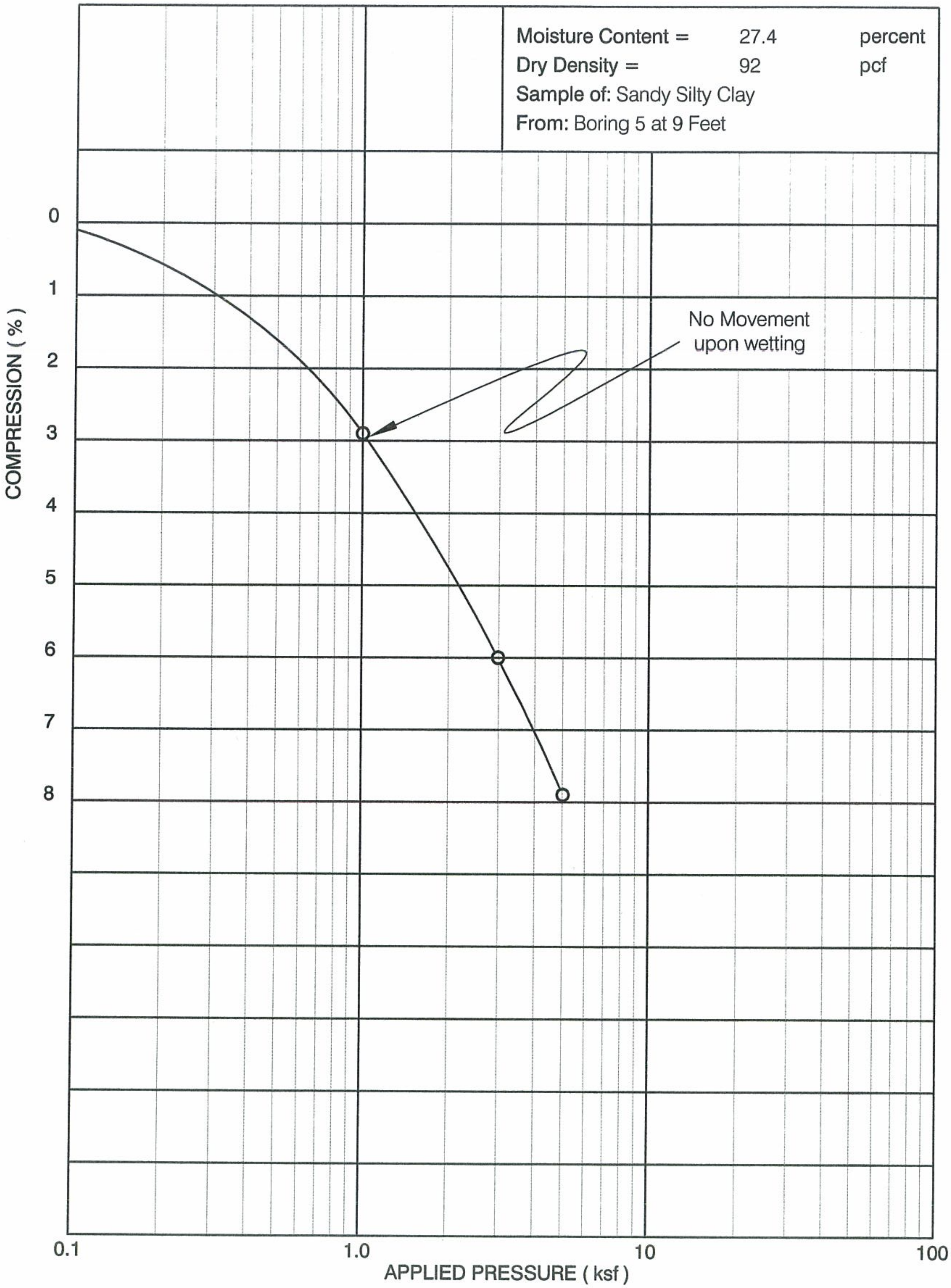


113 097A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	SWELL-CONSOLIDATION TEST RESULTS	FIGURE 7

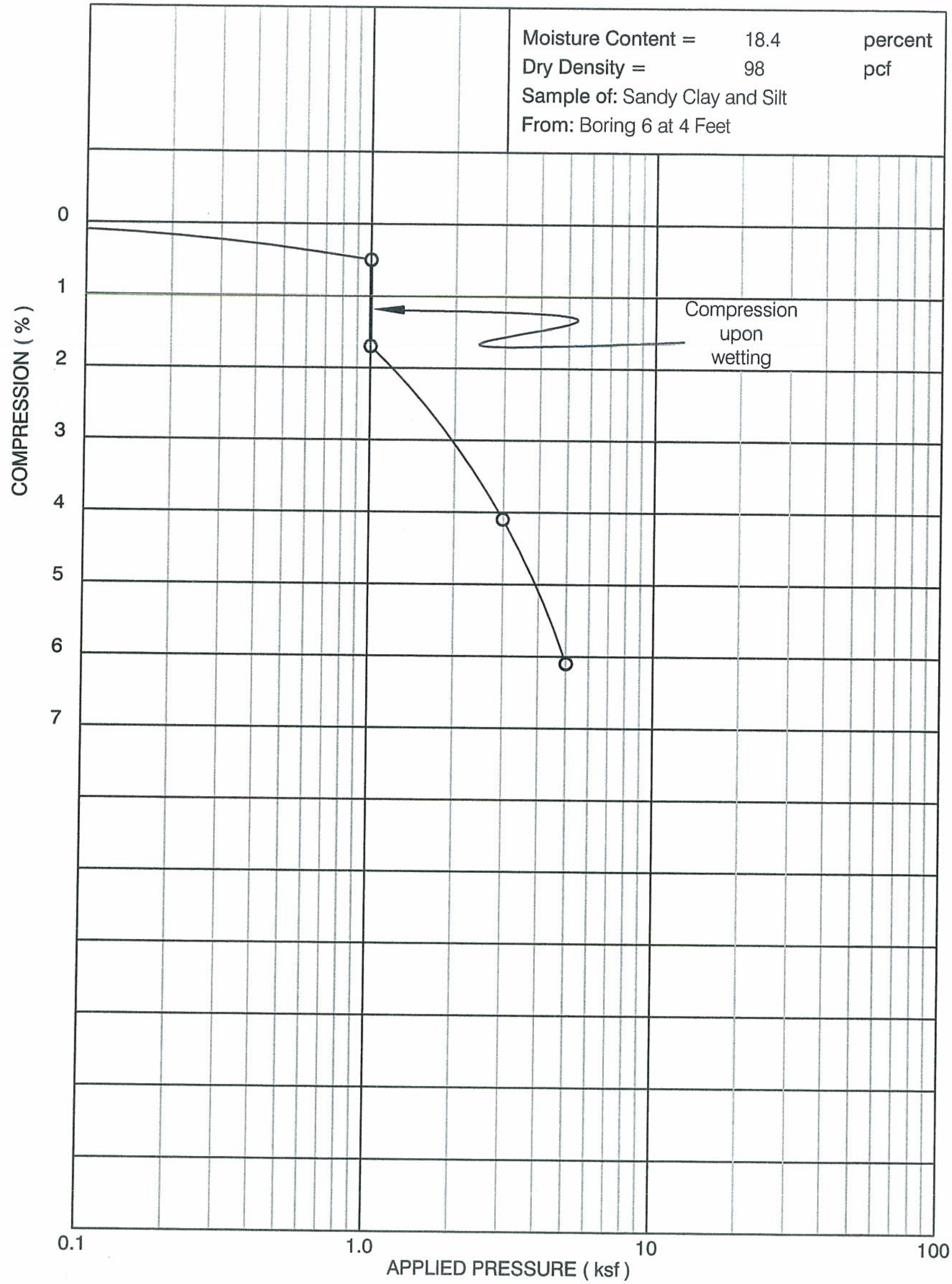
Moisture Content = 23.3 percent
Dry Density = 98 pcf
Sample of: Sandy Clay and Silt
From: Boring 3 at 9 Feet



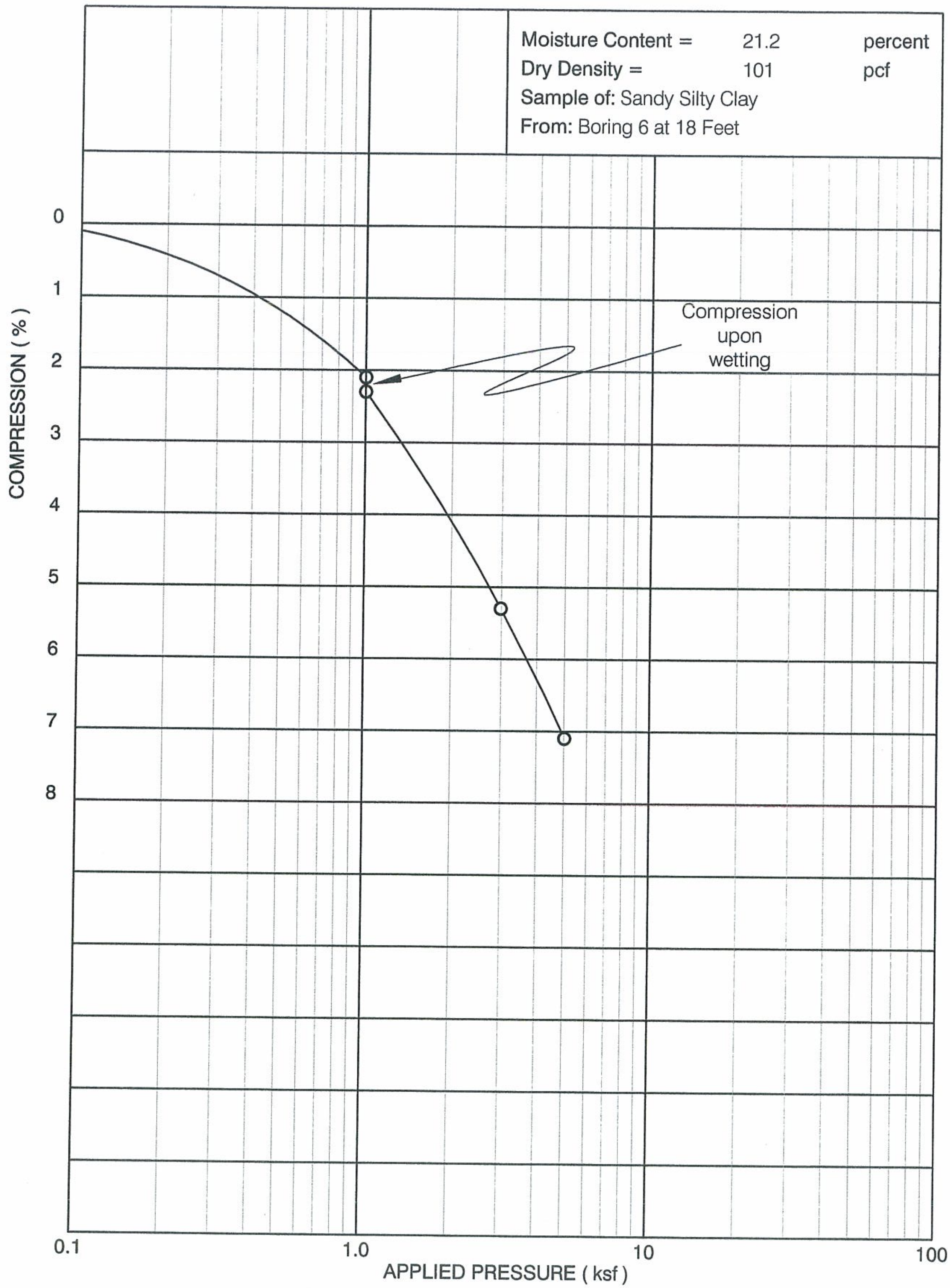
113 097A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	SWELL-CONSOLIDATION TEST RESULTS	FIGURE 8



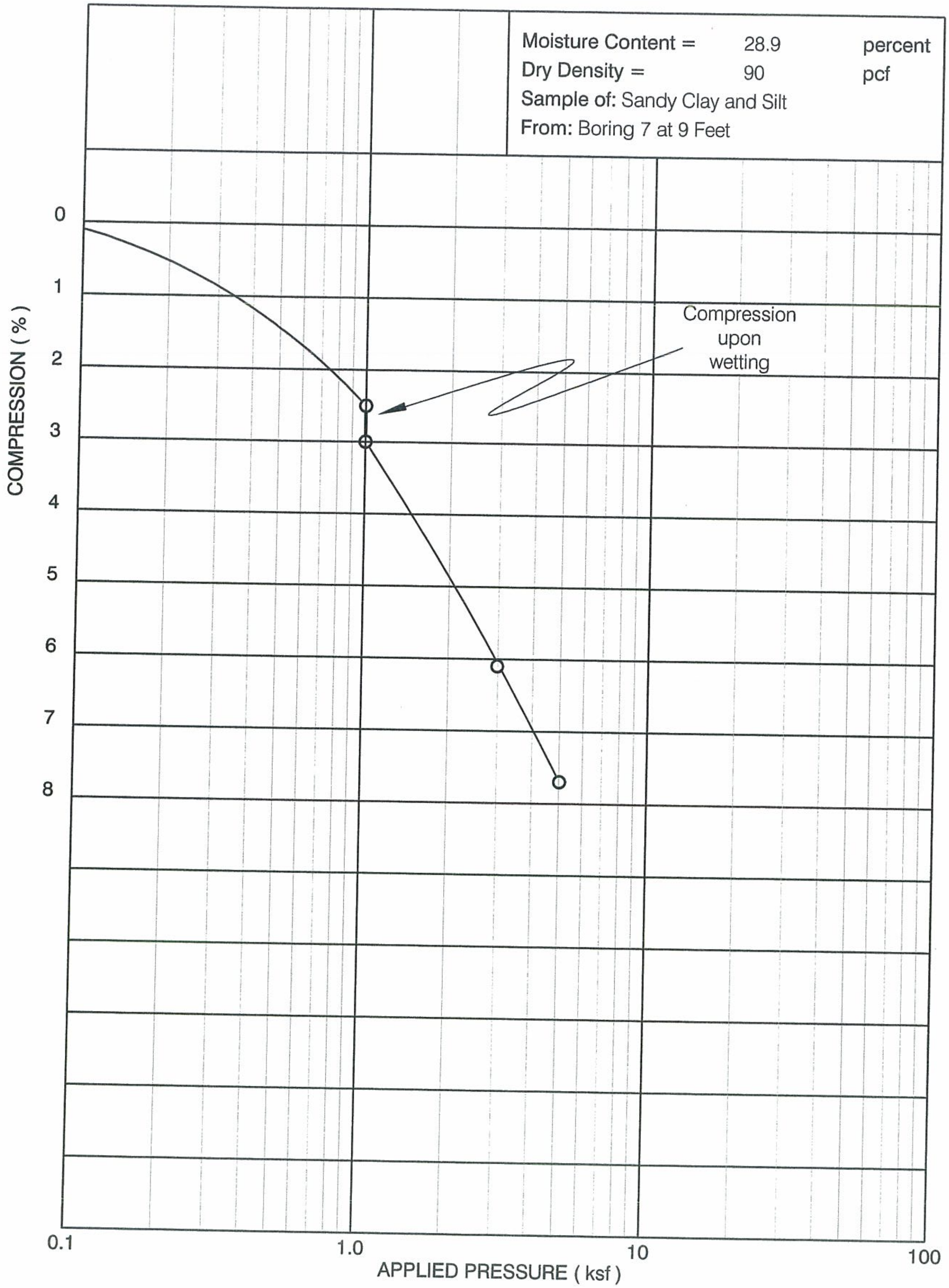
113 097A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	SWELL-CONSOLIDATION TEST RESULTS	FIGURE 9



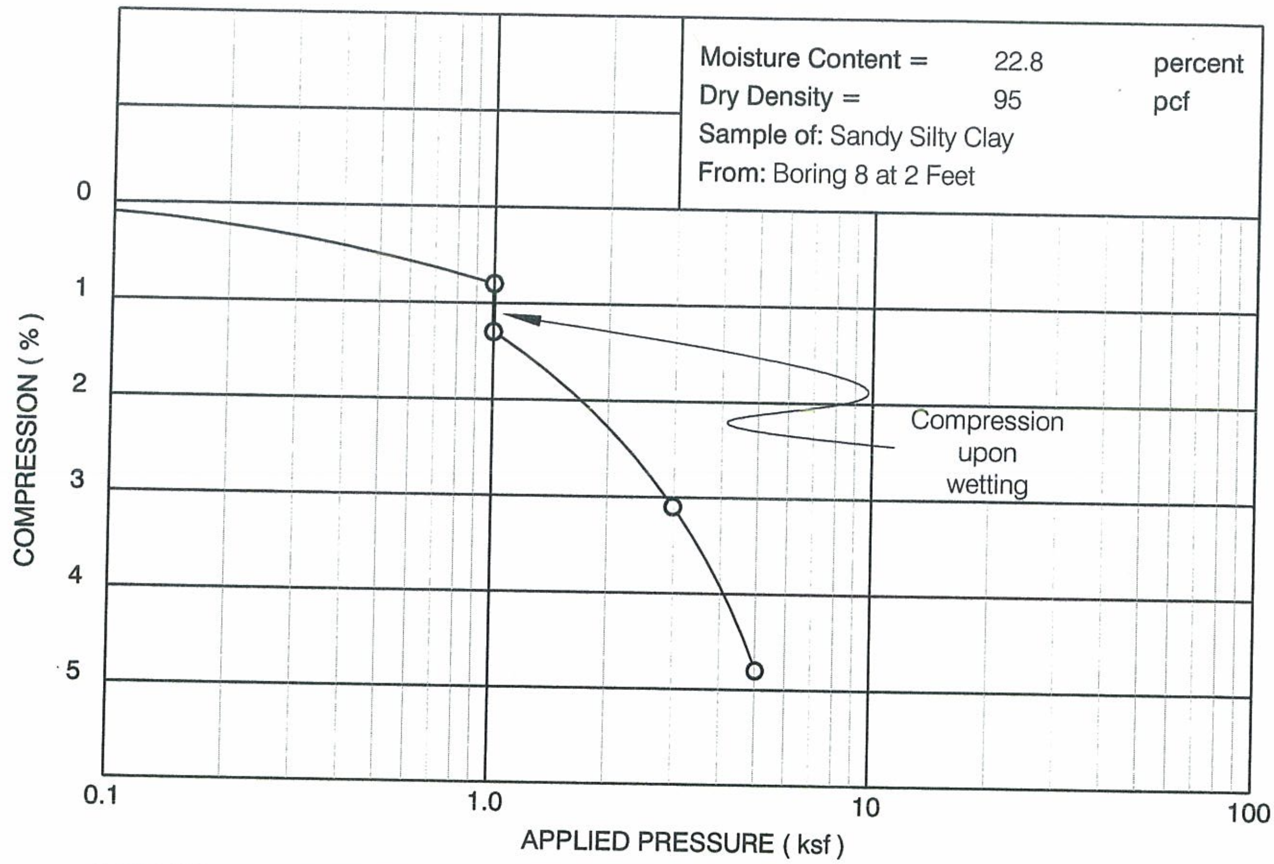
113 097A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	SWELL-CONSOLIDATION TEST RESULTS	FIGURE 10

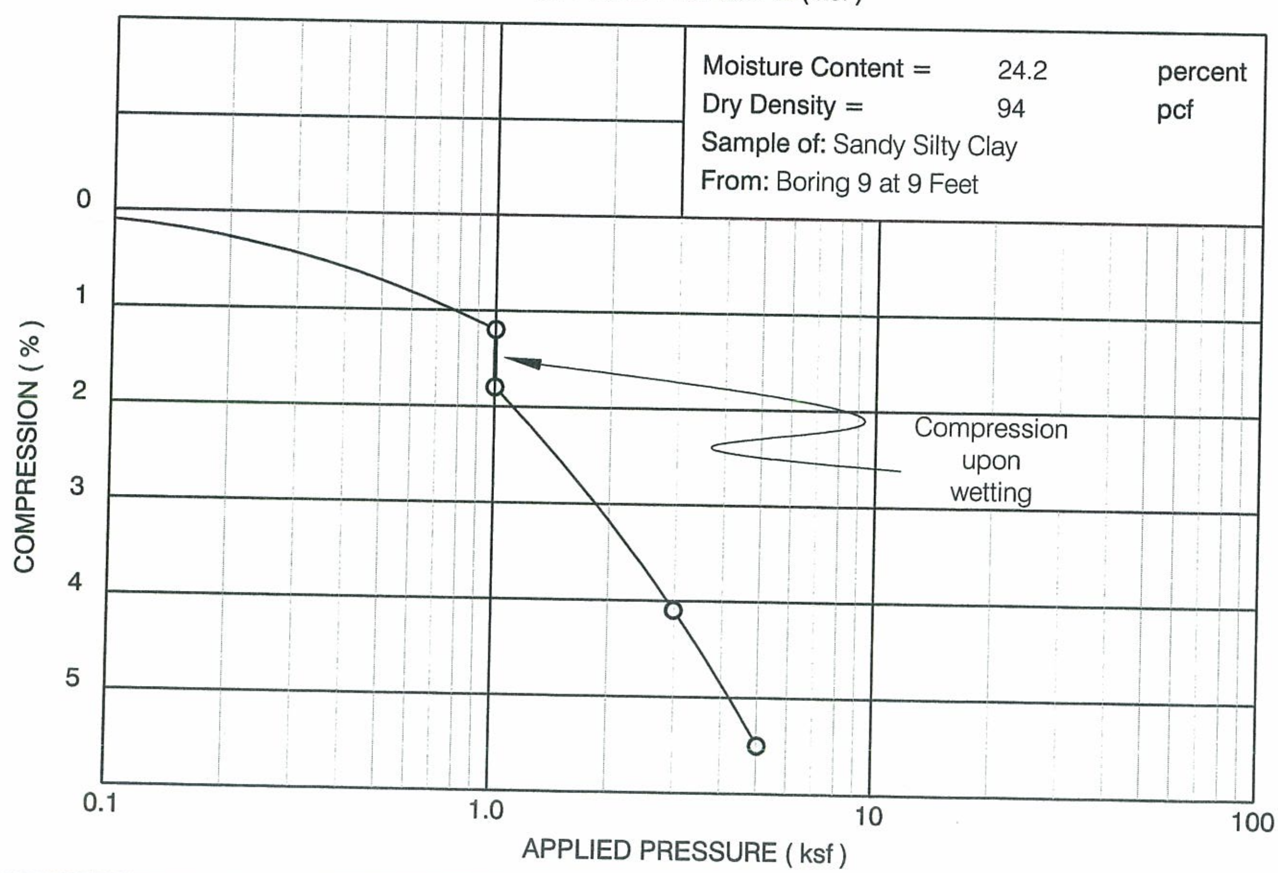


113 097A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	SWELL-CONSOLIDATION TEST RESULTS	FIGURE 11

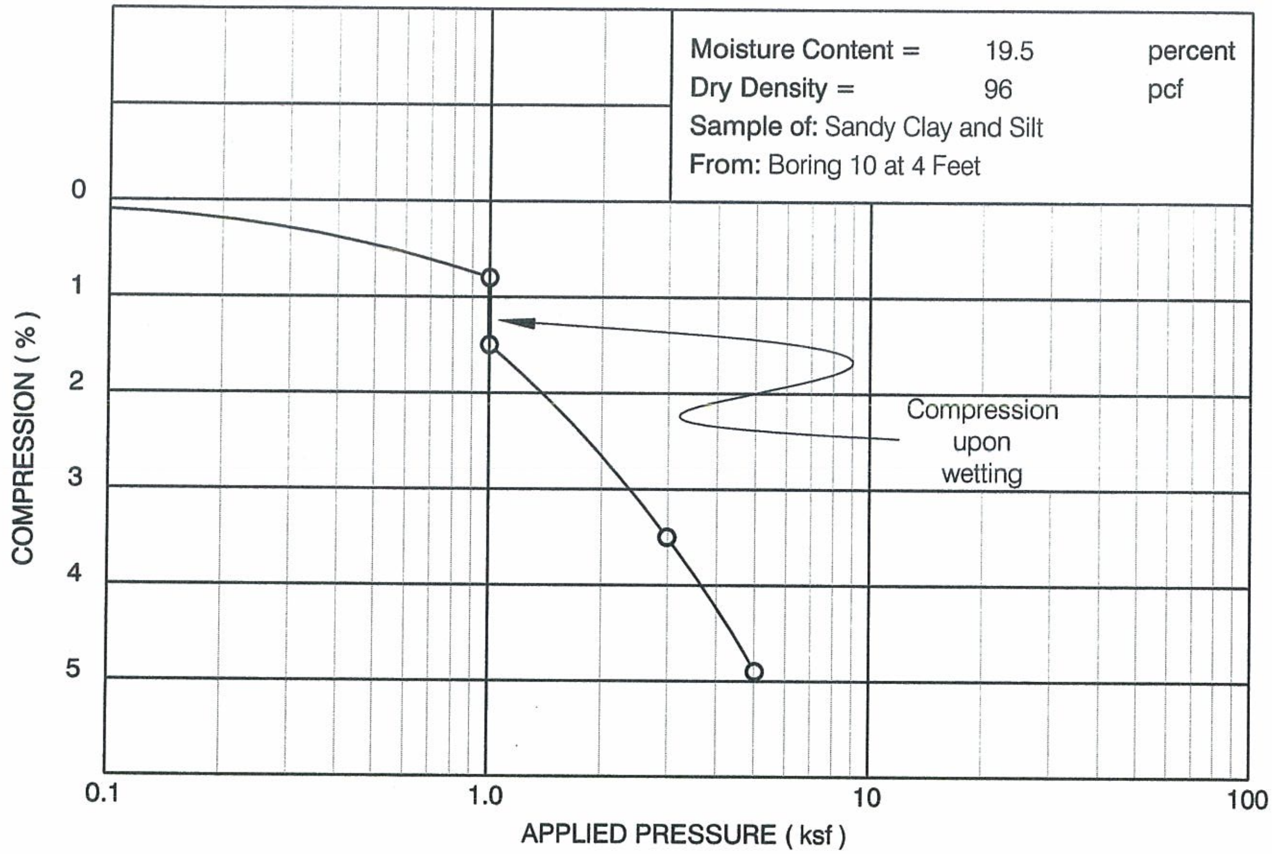


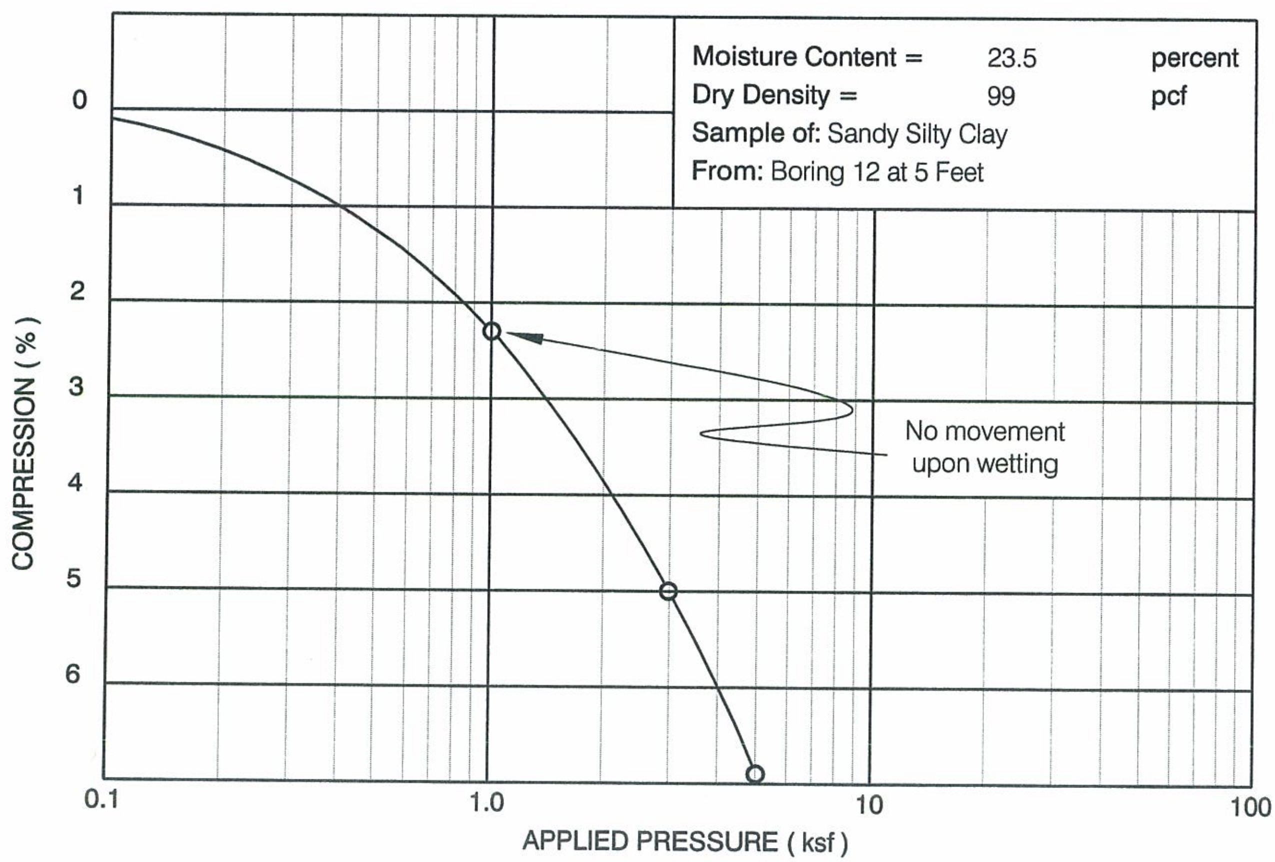
113 097A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	SWELL-CONSOLIDATION TEST RESULTS	FIGURE 12



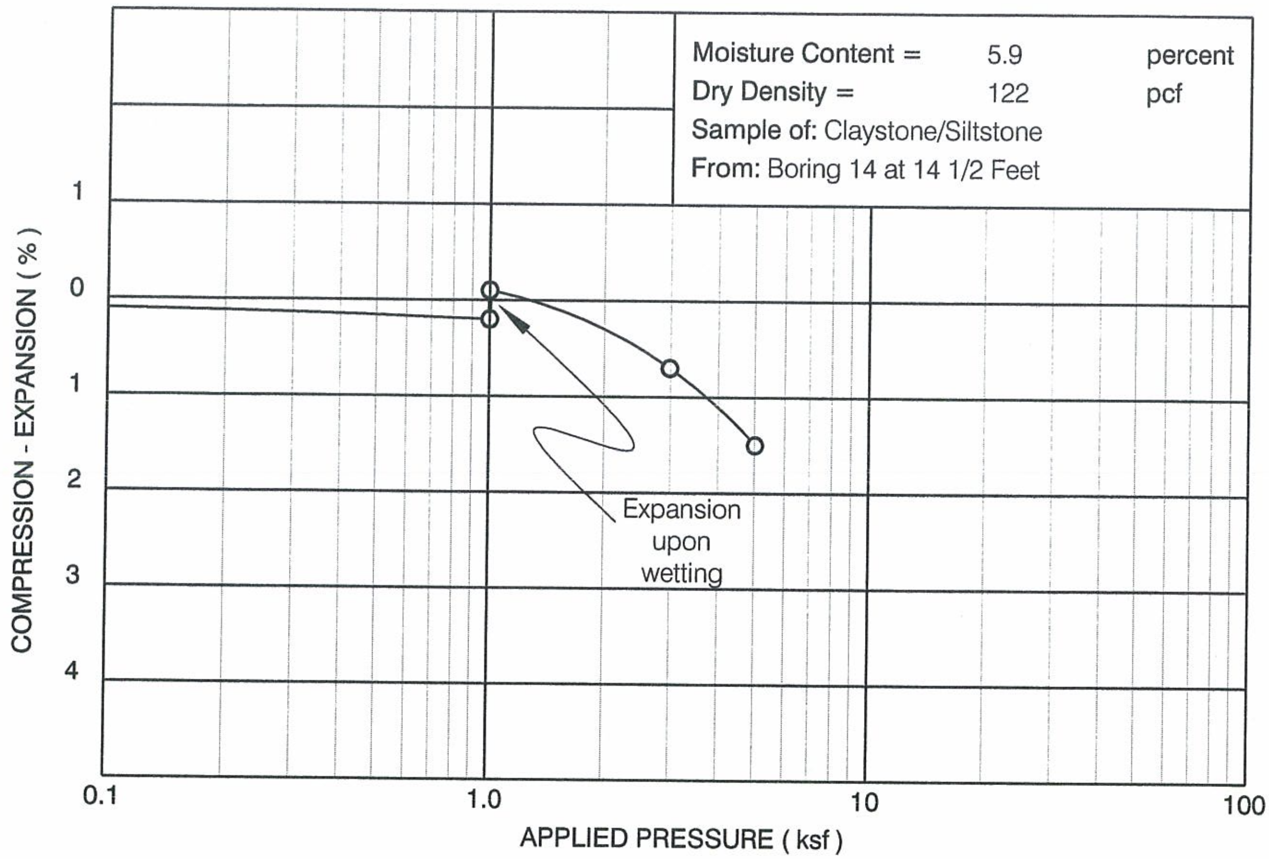


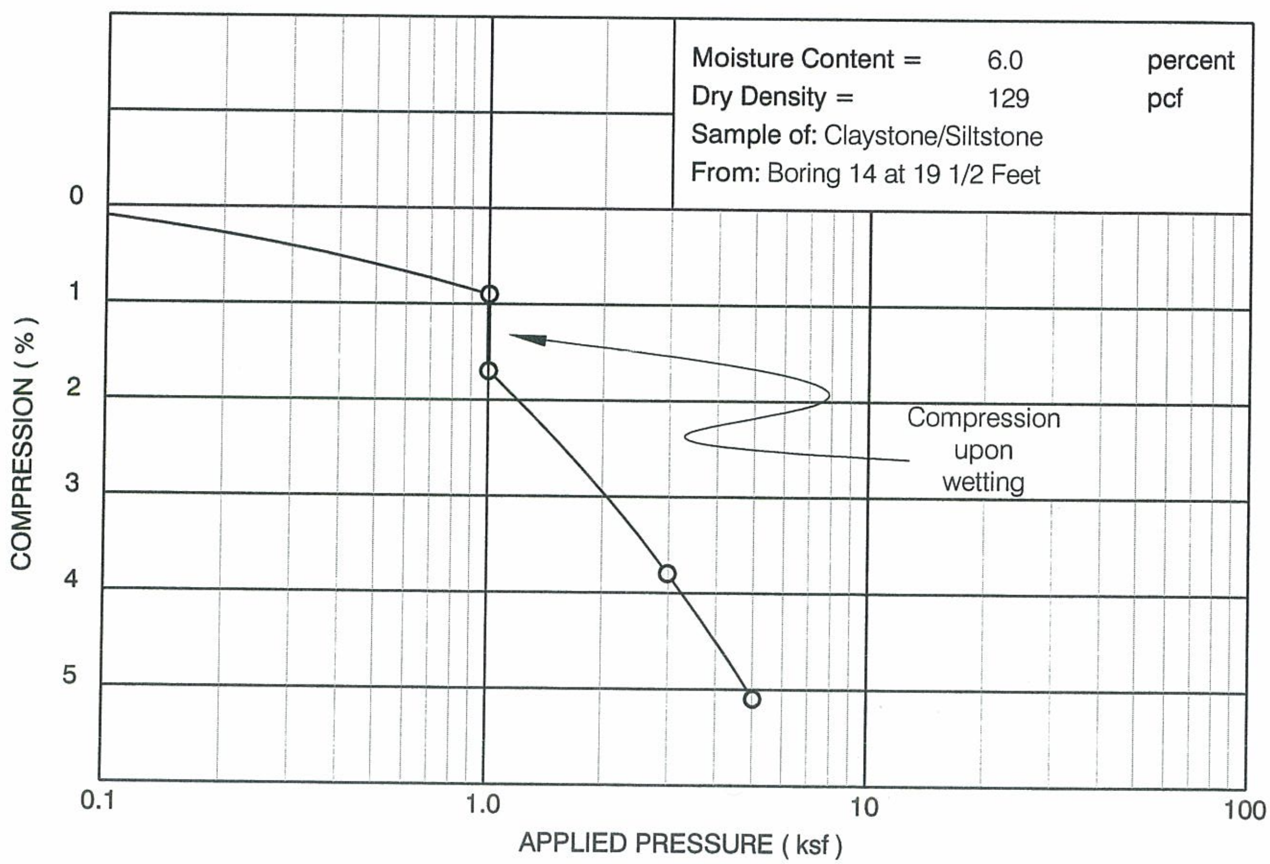
113 097A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	SWELL-CONSOLIDATION TEST RESULTS	FIGURE 13



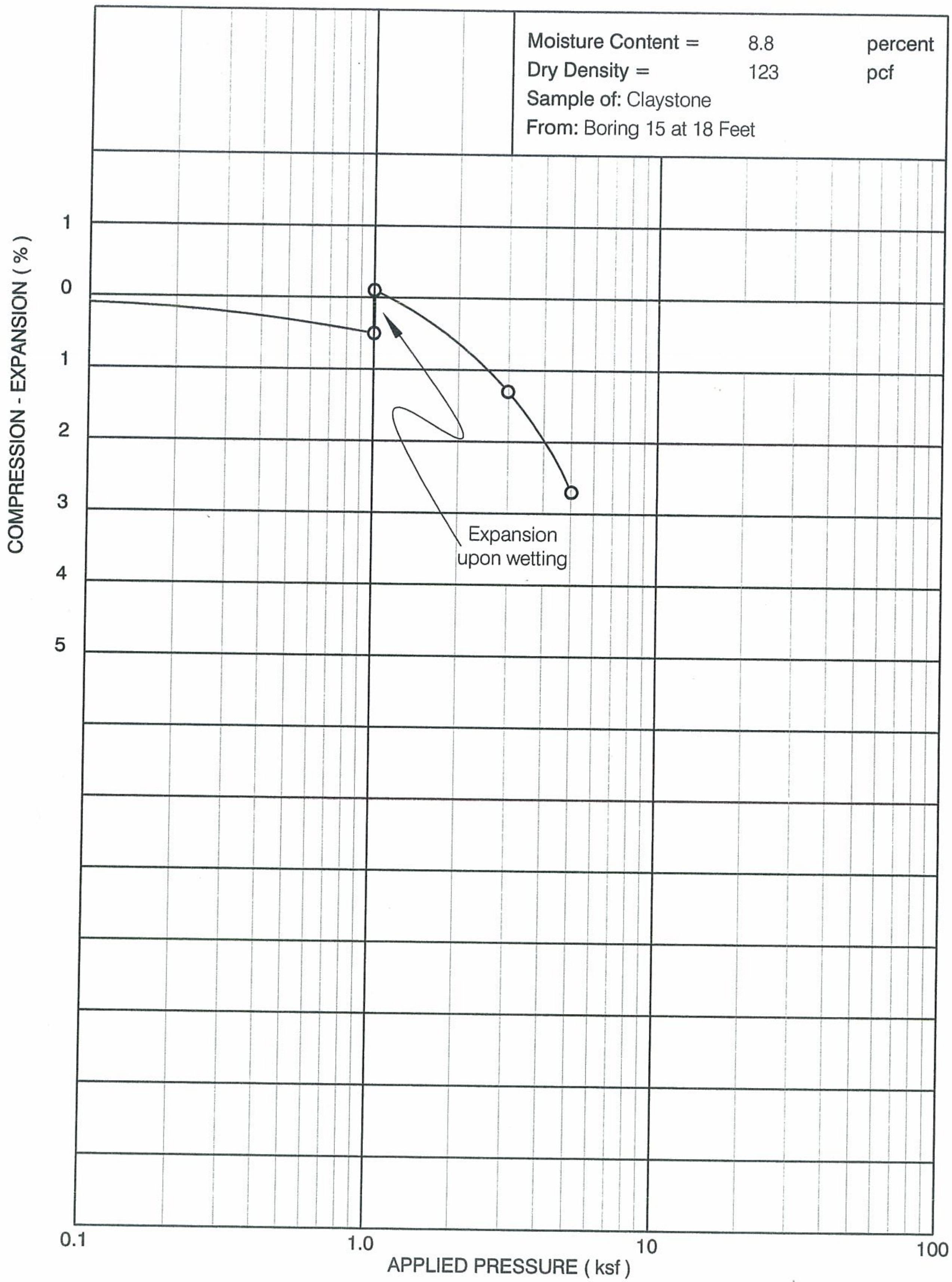


113 097A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	SWELL-CONSOLIDATION TEST RESULTS	FIGURE 14

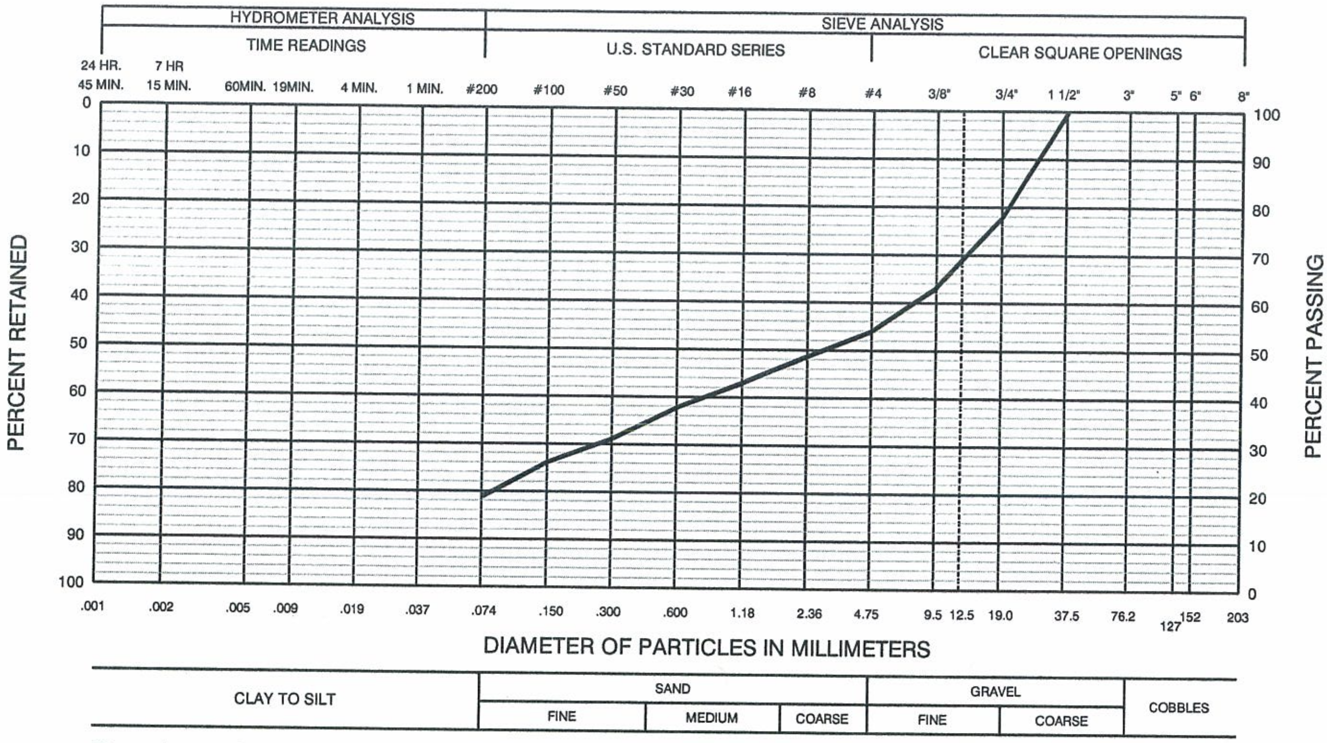




113 097A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	SWELL-CONSOLIDATION TEST RESULTS	FIGURE 15



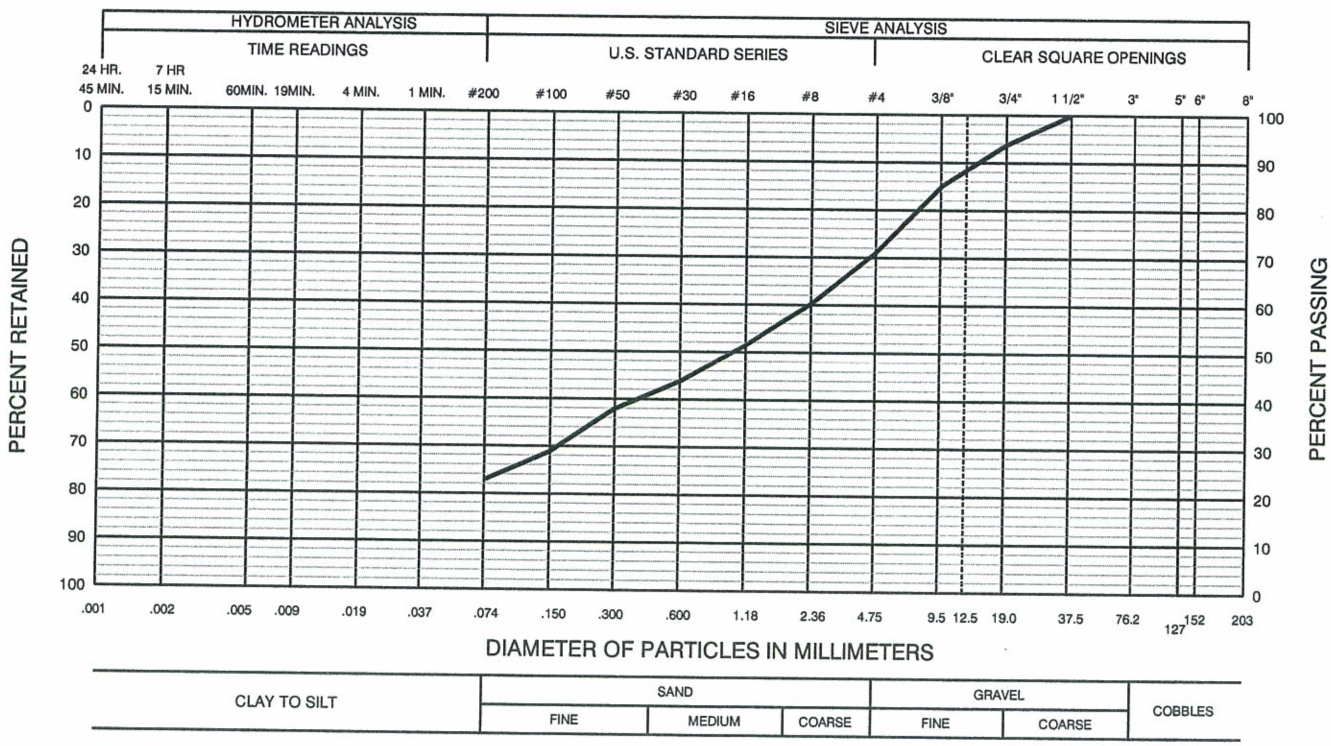
113 097A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	SWELL-CONSOLIDATION TEST RESULTS	FIGURE 16



Gravel 46 % Sand 35 % Silt and Clay 19 %

Sample of: Silty Sandy Gravel

From: Boring 5 at 19 Feet

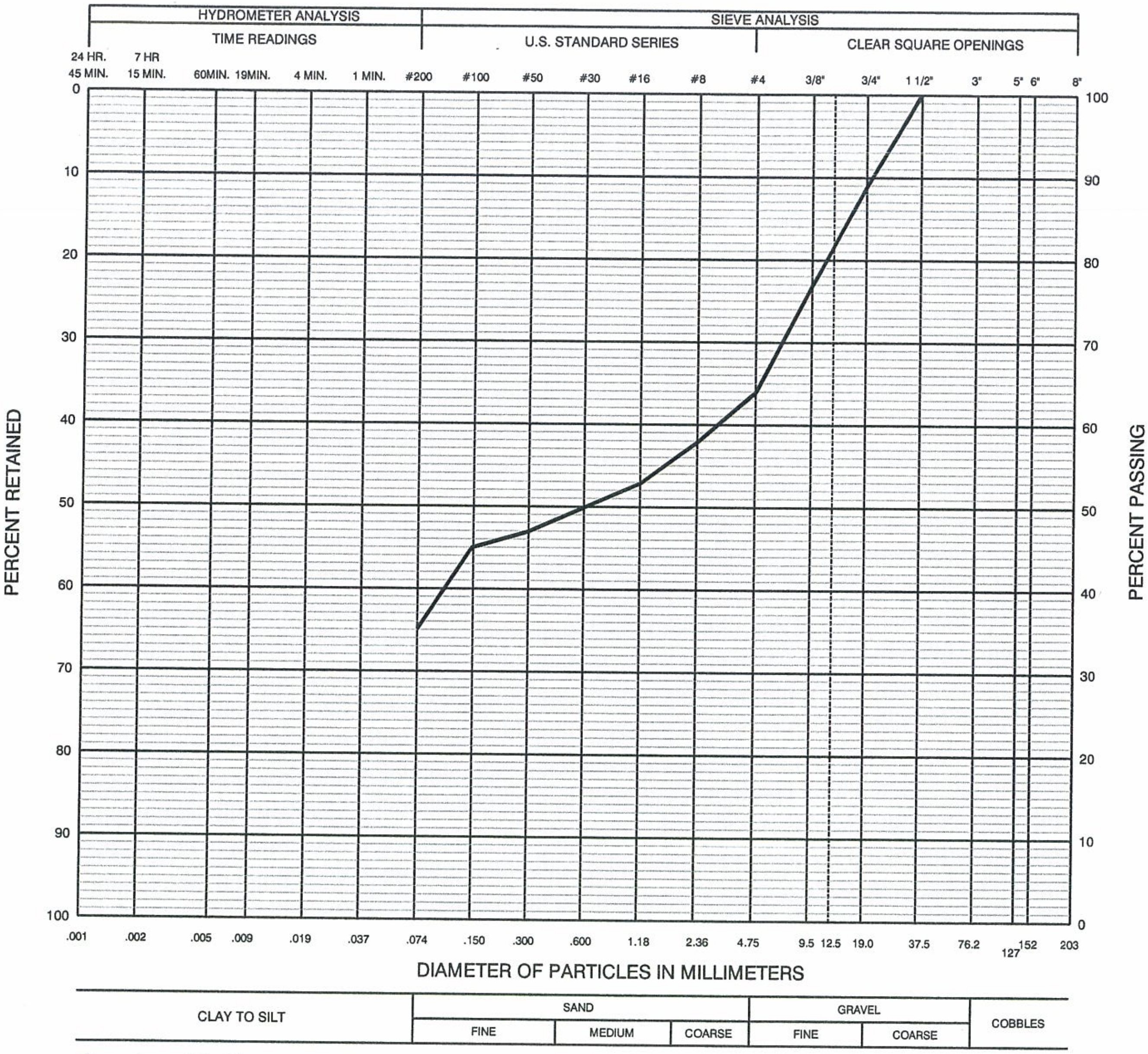


Gravel 29 % Sand 48 % Silt and Clay 23 %

Sample of: Silty Gravelly Sand

From: Boring 10 at 19 Feet

113 097A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	GRADATION TEST RESULTS	FIGURE 17



Gravel 36 % Sand 29 % Silt and Clay 35 %

Sample of: Silty Sand and Gravel

From: Boring 11 at 5 Feet

113 097A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	GRADATION TEST RESULTS	FIGURE 18

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		AASHTO CLASSIFICATION	ORGANIC MATTER CONTENT	SOIL OR BEDROCK TYPE
BORING	DEPTH (ft)	(%)	(pcf)	GRAVEL (%)	SAND (%)		LIQUID LIMIT (%)	PLASTIC INDEX (%)		(%)	
1	4	20.7	102			80	27	10	A-6(6)		Sandy Silty Clay
2	4	27.4	95								Sandy Silty Clay
3	4	17.3	89			64	23	2	A-4(0)		Sandy Clay and Silt
	9	23.3	98								Sandy Clay and Silt
	14	17.3	106								Sandy Clay and Silt
4	4	14.3				49					Very Silty Sand (Volcanic Ash)
	9	24.1	99								Sandy Silty Clay
5	4	23.4	97			72	26	5	A-4(2)		Sandy Clay and Silt
	9	27.4	92								Sandy Silty Clay
	19	11.1		46	35	19					Silty Sandy Gravel
6	4	18.4	98								Sandy Clay and Silt
	18	21.2	101								Sandy Silty Clay

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		AASHTO CLASSIFICATION	ORGANIC MATTER CONTENT	SOIL OR BEDROCK TYPE
BORING	DEPTH (ft)	(%)	(pcf)	GRAVEL (%)	SAND (%)		LIQUID LIMIT (%)	PLASTIC INDEX (%)		(%)	
7	4	23.9	96			85					Sandy Silty Clay
	9	28.9	90								Sandy Clay and Silt
	14	20.1	104								Sandy Silty Clay
8	2	22.8	95								Sandy Silty Clay
	10	33.3	85			92					Sandy Silty Clay
9	4	23.8	95			83	27	5	A-4(3)		Sandy Silty Clay
	9	24.2	94								Sandy Silty Clay
	14	28.4	92								Sandy Silty Clay
10	4	19.5	96								Sandy Clay and Silt
	14	28.6	90								Sandy Silty Clay
	19	3.2		29	48	23					Silty Gravelly Sand
11	2	20.4	103			84	36	17	A-6(14)		Sandy Silty Clay
	5	10.2		36	29	35					Silty Sand and Gravel
12	2	25.5	95								Sandy Silty Clay
	5	23.5	99								Sandy Silty Clay

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		AASHTO CLASSIFICATION	ORGANIC MATTER CONTENT	SOIL OR BEDROCK TYPE
BORING	DEPTH (ft)	(%)	(pcf)	GRAVEL (%)	SAND (%)		LIQUID LIMIT (%)	PLASTIC INDEX (%)		(%)	
13	4	29.2	87								Weathered Claystone/Siltstone
	9	6.6	115								Claystone/Siltstone
	19	7.8	122				28	8			Claystone/Siltstone
14	4½	3.6								1.9	Claystone/Siltstone
	14½	5.9	122								Claystone/Siltstone
	19½	6.0	129								Claystone/Siltstone
15	3	3.0								1.2	Claystone
	8	7.3	120				34	14			Claystone/Siltstone
	18	8.8	123								Claystone

HEPWORTH-PAWLAK GEOTECHNICAL, INC.
TABLE 2
SUMMARY OF LABORATORY CHEMICAL TEST RESULTS

Job No. 113 097A

Boring No.	Depth (ft)	Moisture Content (%)	Organic Matter Content (%)	pH	Total Soluble Salts (%)	Water Soluble Sulfates (%)	Redox Potential (mV)	Apparent Resistivity (ohm-cm)	Soil Type
14	4½	3.6	1.9	7.8	0.094	0.004	320	1970	Claystone/Siltstone
15	3	3.0	1.2	7.5	0.135	0.018	322	900	Claystone

APPENDIX A

GROUNDWATER LEVEL

MEASUREMENTS BY OTHERS

Haymeadow

Piezometer	Water Level Bleow Ground Surface (ft)									
	5/21/2013	6/5/2013	6/6/2013	6/13/2013	6/21/2013	6/26/2013	7/4/2013	7/11/2013	7/18/2013	7/25/2013
Boring 1			5'7"	7'6"	8'2"	8'	8'3"	7'6"	7'	7'
Boring 2			4'6"		5'	6'	6'	6'	4'6"	6'
Boring 4			Dry	13'6"	13'6"	15'	14'	13'6"	13'	13'
Boring 5			12'	10'6"	10'6"	10'6"	10'9"	10'6"	9'8"	10
Boring 7			17'8"	13'6"	13'	14'	14'6"	13'4	12'4"	13'2"
Boring 8	Dry		11'	7'4"	8'	9'2"	9'	8'3"	6'6"	7'4"
Boring 9		21'	17'	11'6"	12'6"	13'	14'3"	11'6"	9'8"	12'
Boring 10			16'	10'6"	10'6"	12'	13'4"	10'6"	8'4"	10'6"
Boring 11	Dry		18'3"	10'6"		16'3"	16'	11'6"	12'	14'
Boring 12	Dry		6'	3'6"	4'	5'	6'4"	4'4"	4'	2'