PRELIMINARY GEOTECHNICAL STUDY
PROPOSED LAKOTA RIDGE SENIOR HOUSING
LOT 2A, LAKOTA CANYON RANCH, PHASE 7
CASTLE VALLEY BOULEVARD
NEW CASTLE, COLORADO

JOB NO. 115 014A

FEBRUARY 10, 2015

PREPARED FOR:

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PURPOSE AND SCOPE OF STUDY

This report presents the results of a preliminary geotechnical study for the proposed senior housing development to be located on Lot 2A, Lakota Canyon Ranch, Phase 7, Castle Valley Boulevard, New Castle, Colorado. The project site is shown on Figure 1. The purpose of the study was to develop recommendations for preliminary foundation and grading designs. The study was conducted in accordance with our proposal for geotechnical engineering services to Community Resources & Housing Development Corporation, dated January 5, 2015. Hepworth-Pawlak Geotechnical previously conducted a preliminary geotechnical study for Eagle Ridge Ranch (now known as Lakota Canyon Ranch) which included the current proposed development site and presented our findings in a report dated April 26, 2002 which has been considered as background information for the current study.

A field exploration program consisting of exploratory borings was conducted to obtain information on the general subsurface conditions at the project site. Samples of the subsurface materials 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 types, depths and allowable pressures for the proposed building foundation and grading of the site. This report summarizes the data obtained during this study and presents our conclusions, design recommendations and other geotechnical engineering considerations based on the general proposed development plan and the subsurface conditions encountered.

PROPOSED CONSTRUCTION

Development plans for the project site were conceptual at the time of our study. In general, the proposed development on the 2.8 acre lot will be a 40 unit affordable senior housing and ancillary facilities project. Ground floor of the structures could be slab-on-grade or structural above crawlspace. The development will include interior driveway

and parking areas. Grading for the structures is assumed to be relatively minor with cut and fill depths up to about 10 feet. We assume relatively light to moderate foundation loadings for the general proposed type of construction.

When building locations, grading and loading information have been developed, we should be notified to conduct additional subsurface evaluation to develop design level recommendations for the project.

SITE CONDITIONS

The lot was vacant and covered with patches of snow at the time of our field exploration. The ground surface slopes fairly uniformly down to the northeast at grades between about 8% in the downhill part and 15% in the uphill part. The grade transitions to a minor drainage swale at the uphill, south side of the lot which slopes down to the east and off of the project site. A steep mountain side is located on the order of 100 feet directly uphill of the site. Vegetation consists of field grass and weeds. Underground utilities and a broad drainage swale are located at the downhill side of the lot.

GEOLOGIC CONDITIONS

The project site is located on a gently to strongly sloping alluvial bench underlain by the upper member of the Mancos Shale Formation. The Mancos makes up the steep hillside located immediately south of the site and has a gentle to moderate bedding dip down to the south into the hillside. The bedding dip is favorable to the stability of the hillside adjacent to the project site. The alluvial soils consist mainly of clay which can have variable compressibility and expansion potential.

FIELD EXPLORATION

The field exploration for the project was conducted on January 26, 2015. Three 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 auger powered by a truck-mounted CME-45B drill rig. The borings were logged by a representative of Hepworth-Pawlak Geotechnical, Inc.

Samples of the subsoils were taken with a 2-inch I.D. spoon sampler. The sampler was 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, Figure 2. 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 Figure 2. The subsoils, below about ½ foot of topsoil, consist of very stiff to hard with depth, sandy silty clay to depths of 14 to 19 feet overlying medium hard and weathered to very hard with depth siltstone/claystone-shale bedrock to the drilled depth of 26 feet.

Laboratory testing performed on samples obtained from the borings included natural moisture content and density, finer than sand size gradation analyses and liquid and plastic limits. Results of swell-consolidation testing performed on relatively undisturbed drive samples of the clay and weathered siltstone/claystone, presented on Figures 4-7, indicate low compressibility under relatively light loading and natural low moisture conditions. The clay soil samples showed low to moderately high collapse (settlement under constant load) or expansion potential when wetted and moderate to high compressibility under additional loaded after wetting. Results of liquid and plastic limits testing indicate the clay soils have low plasticity. The laboratory testing is summarized in Table 1.

No free water was encountered in the borings at the time of drilling and the subsoils and bedrock were slightly moist.

FOUNDATION BEARING CONDITIONS

The clay soils encountered to depths of about 14 to 19 feet at the borings have variable settlement/heave potential, mainly when wetted, which could result in differential movement of a shallow foundation and building distress. The underlying weathered siltstone/claystone bedrock generally has an expansion potential when wetted and becomes very hard with depth and not likely to have an expansion potential. At residential projects there are several sources of subsurface wetting, such as irrigation, surface water runoff and utility line leaks. A relatively low risk foundation system with regard to potential building movement caused by subsurface wetting is straight-shaft drilled piers that extend down into the underlying very hard bedrock. In addition to their ability to reduce the foundation movement risk, the piers have the advantage of providing relatively high load capacity with a relatively small settlement potential. Presented below are preliminary recommendations for design of the buildings and site grading. When specific building types, locations and grading plans have been developed, we should be contacted to provide additional evaluations and design level recommendations.

PRELIMINARY DESIGN RECOMMENDATIONS

FOUNDATIONS

Drilled Piers: Considering the subsurface conditions encountered in the exploratory borings and the nature of the proposed construction, we recommend straight shaft piers drilled into the underlying bedrock for support of movement sensitive structures. The design and construction criteria presented below should be observed for a straight-shaft drilled pier foundation system.

1) The piers should be designed for an allowable end bearing pressure of 30,000 psf and a skin friction of 3,000 psf for that portion of the pier embedded in hard bedrock. Pier penetration through the upper clay soils

and weathered bedrock should be neglected in the skin friction calculations.

2) Piers should also be designed for a minimum dead load pressure of 10,000 psf based on pier end area only. If the minimum dead load requirement cannot be achieved, the pier length should be extended beyond the minimum penetration to make up the dead load deficit. This can be accomplished by assuming one-half the allowable skin friction value given above acts in the direction to resist uplift.

3) Uplift on the piers from structural loading can be resisted by utilizing 75% of the allowable skin friction value plus an allowance for the weight of the pier.

4) Piers should penetrate at least three pier diameters into the hard bedrock. A minimum penetration of 8 feet into the bedrock (including the weathered depth) and a minimum pier length of 20 feet are recommended.

5) Piers should be designed to resist lateral loads assuming a modulus of horizontal subgrade reaction of 50 tcf in the clay soils and a modulus of horizontal subgrade reaction of 250 tcf in the bedrock. The modulus values given are for a long, 1 foot wide pier and must be corrected for pier size.

6) Piers should be reinforced their full length with one #5 reinforcing rod for each 16 inches of pier perimeter to resist tension created by the swelling materials.

7) A 4-inch void form should be provided beneath grade beams to prevent the swelling soil and rock from exerting uplift forces on the grade beams and to concentrate pier loadings. A void form should also be provided beneath pier caps.

8) Concrete utilized in the piers should be a fluid mix with sufficient slump so that concrete will fill the void between the reinforcing steel and the pier hole.

9) Pier holes should be properly cleaned prior to the placement of concrete. The drilling contractor should mobilize equipment of sufficient size to effectively drill through possible coarse soils and cemented bedrock zones.

10) Although free water was not encountered in the borings drilled at the site, some seepage in the pier holes may be encountered during drilling. If water cannot be removed or sealed off prior to placement of concrete, the tremie method should be used after the hole has been cleaned of spoil. In no case should concrete free fall into more than 3 inches of water.

11) Care should be taken to prevent the forming of mushroom-shaped tops of the piers which can increase uplift force on the piers from swelling soils.

12) A representative of the geotechnical engineer should observe pier drilling operations on a full-time basis.

Foundation Alternative: In low settlement or expansive clay soil areas, it may be feasible to support the structures on lightly loaded spread footings placed on a minimum 3 foot depth of compacted structural fill with some risk of long term foundation movement and distress. The design and construction criteria presented below should be observed for a spread footing foundation system.

1) Footings placed on a minimum 3 foot depth of compacted structural fill can be designed for an allowable bearing pressure of 2,000 psf. Based on experience, we expect settlement/heave of footings designed and constructed as discussed in this section will be about 1 to 2 inches which could occur over a long time period. Heavily reinforced continuous wall foundations rather than isolated pads should be used to limit the effects of differential settlement.

2) Prior to placing structural fill for the foundation, the area should be stripped of the vegetation and topsoil. Structural fill should be placed in uniform lifts not to exceed 8 inches and compacted to at least 98% of the maximum standard Proctor density at within 2% of optimum moisture content. The structural fill should extend laterally beyond the edges of the footing a distance at least equal to one-half the depth of fill below the

footing. The structural fill should have sufficient fines content to restrict subsurface water flow and can consist of the on-site clay.

3) The footings should have a minimum width of 16 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 36 inches below exterior grade is typically used in this area.

5) Continuous foundation walls should be heavily reinforced top and bottom to span an unsupported length of at least 12 feet.

6) A representative of the geotechnical engineer should evaluate fill placement for compaction and observe all footing excavations 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 55 pcf for backfill consisting of the on-site clay soils and 45 pcf for backfill consisting of imported granular materials. Cantilevered retaining structures which are separate from structures 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 45 pcf for backfill consisting of the on-site soils and 40 pcf for backfill consisting of imported granular materials.

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 near optimum moisture content. Backfill placed 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.

FLOOR SLABS

The upper clay soils encountered in the borings possess variable settlement/heave potential and slab movement could occur if the subgrade soils were to become wet. Slab-on-grade construction may be used provided precautions are taken to limit potential movements and the risk of distress to the building is accepted by the owner. Removal and replacement of the natural soils to provide at least 3 feet of compacted structural fill below slabs can be done to reduce the risk of slab movement. The structural fill should be constructed similar to that recommended above in the *Foundation Alternative* section. A structural floor above crawlspace could also be used to achieve a low movement risk.

To reduce the effects of some differential settlement, nonstructural 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. Slab reinforcement and control joints should be established by the designer based on experience and the intended slab use.

A minimum 4 inch layer of base course gravel should be placed immediately beneath slabs-on-grade. This material should consist of minus 2 inch aggregate with less than 50% passing the No. 4 sieve and less than 12% passing the No. 200 sieve. The gravel will provide slab support and help break capillary moisture rise.

Required fill beneath slabs can consist of the on-site soils or a suitable imported granular material, excluding topsoil and oversized rocks. The fill should be spread in thin horizontal lifts, adjusted to at or above optimum moisture content, and compacted to at least 95% of the maximum standard Proctor density. All topsoil and loose or disturbed soil should be removed and the subgrade moistened and compacted prior to fill placement.

UNDERDRAIN SYSTEM

Although free water was not encountered during our exploration, it has been our experience in the area and where bedrock is shallow that local perched groundwater can develop during times of heavy precipitation or seasonal runoff. Frozen ground during spring runoff can 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 2 feet deep. An impervious liner such as a 20 mil PVC membrane should be placed beneath the drain gravel in a trough shape and attached to the foundation wall with mastic to prevent wetting of the bearing soils and potential flow of water to below the structure.

PAVEMENT SECTION

We expect that asphalt pavement will be used for the driveways and parking areas. Traffic loadings for the pavement areas have not been provided. The near surface soils encountered at the site are generally low plasticity sandy silty clays which are considered a relatively poor support for pavement sections. Based on our experience, an assumed 18 kip EDLA of 5, a Regional Factor of 1.75 and a serviceability index of 2.0, the minimum pavement section thickness should consist of 4 inches of asphalt on 8 inches of base course in driveway areas and 3 inches of asphalt on 8 inches of base course in parking areas. A granular subbase aggregate could be used below the base course layer to improve the long term pavement performance.

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 at a moisture content within 2% of optimum.

Required fill to establish design subgrade level can consist of the on-site soils or suitable imported granular soils. Prior to fill placement the subgrade should be stripped of vegetation and 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 soils 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 developed, we should re-evaluate the pavement section recommendations.

SURFACE DRAINAGE

The following drainage precautions should be observed during construction and maintained at all times after the structures have 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 about 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 10 feet from foundation walls. Consideration should be given to the use of xeriscape to limit potential wetting of soils below the building caused by irrigation.

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 to be different from those described in this report, we should be notified at once so 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 of the recommendations presented herein. We recommend on-site observation of pier drilling, excavations and foundation bearing strata and testing of structural fill by a representative of the geotechnical engineer.

Respectfully Submitted,

HEPWORTH - PAWLAK GEOTECHNICAL, INC.

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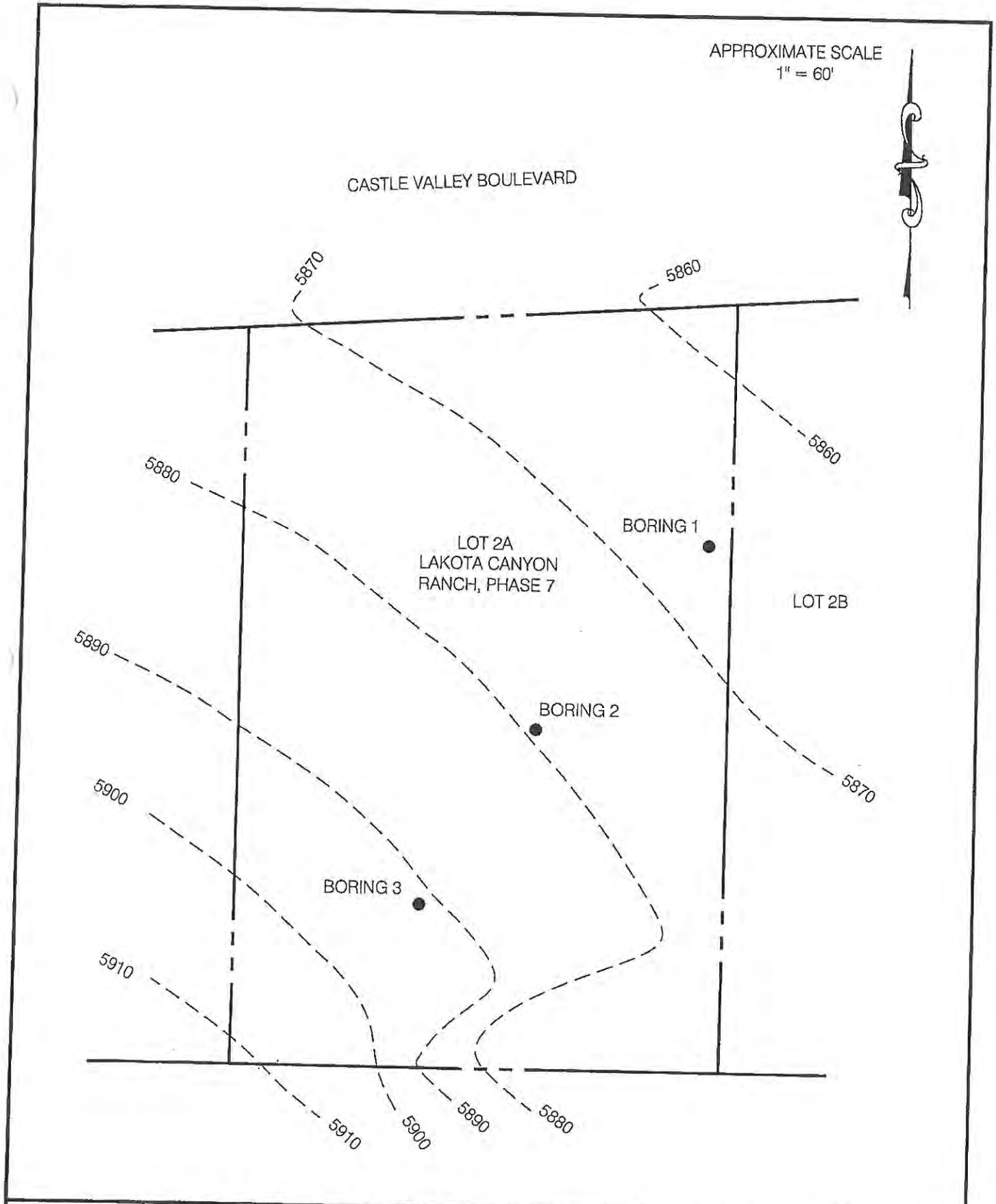
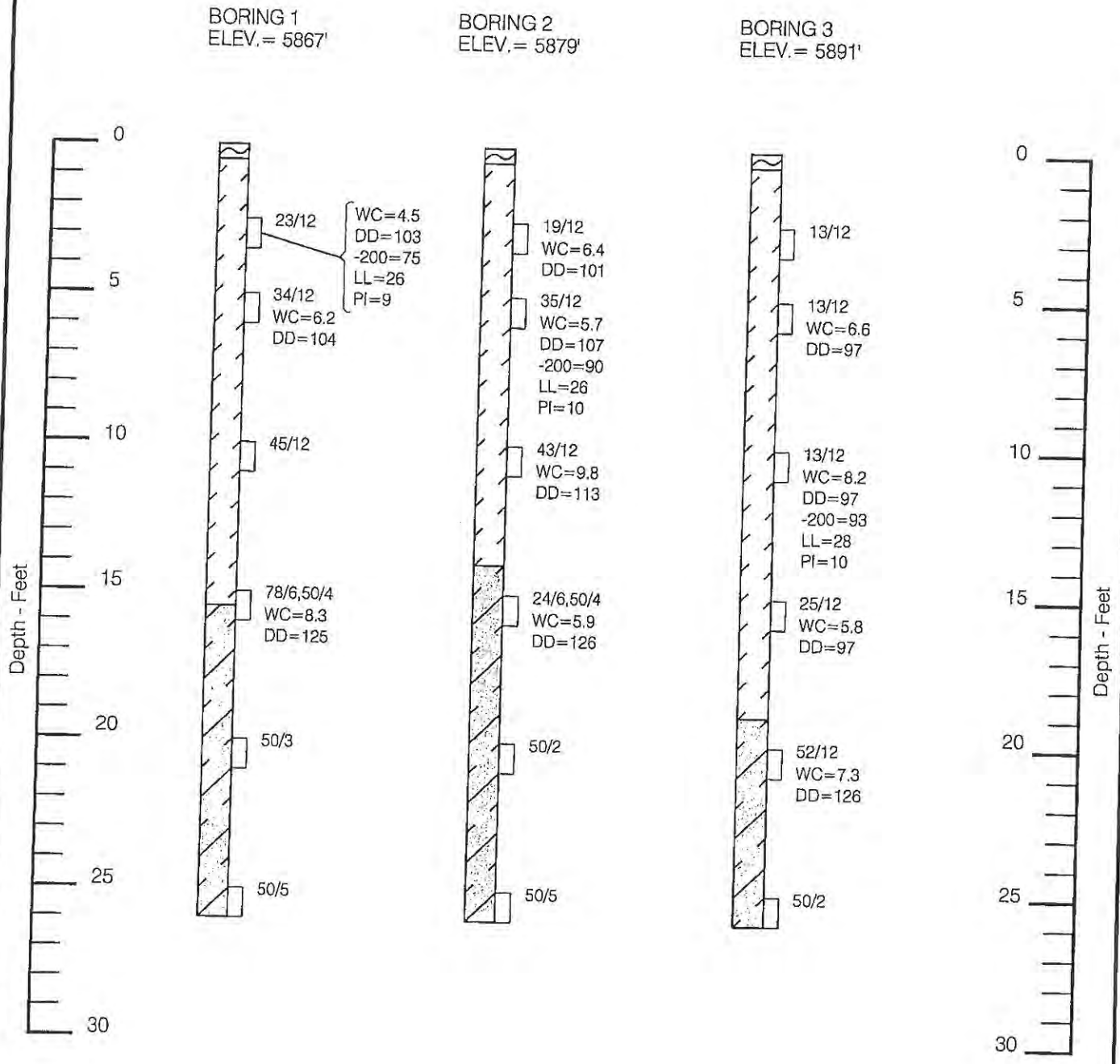


115 014A	HP Geotech Hepworth–Pawlak Geotechnical	LOCATION OF EXPLORATORY BORINGS	Figure 1



Note: Explanation of symbols is shown on Figure 3.

115 014A	HP Geotech Hepworth–Pawlak Geotechnical	LOGS OF EXPLORATORY BORINGS	Figure 2

LEGEND:

TOPSOIL; organic sandy silt and clay, brown.

CLAY (CL); silty, slightly sandy to sandy, scattered gravel, very stiff to hard with depth, slightly moist, light brown to medium brown, low plasticity, slightly porous and calcareous.

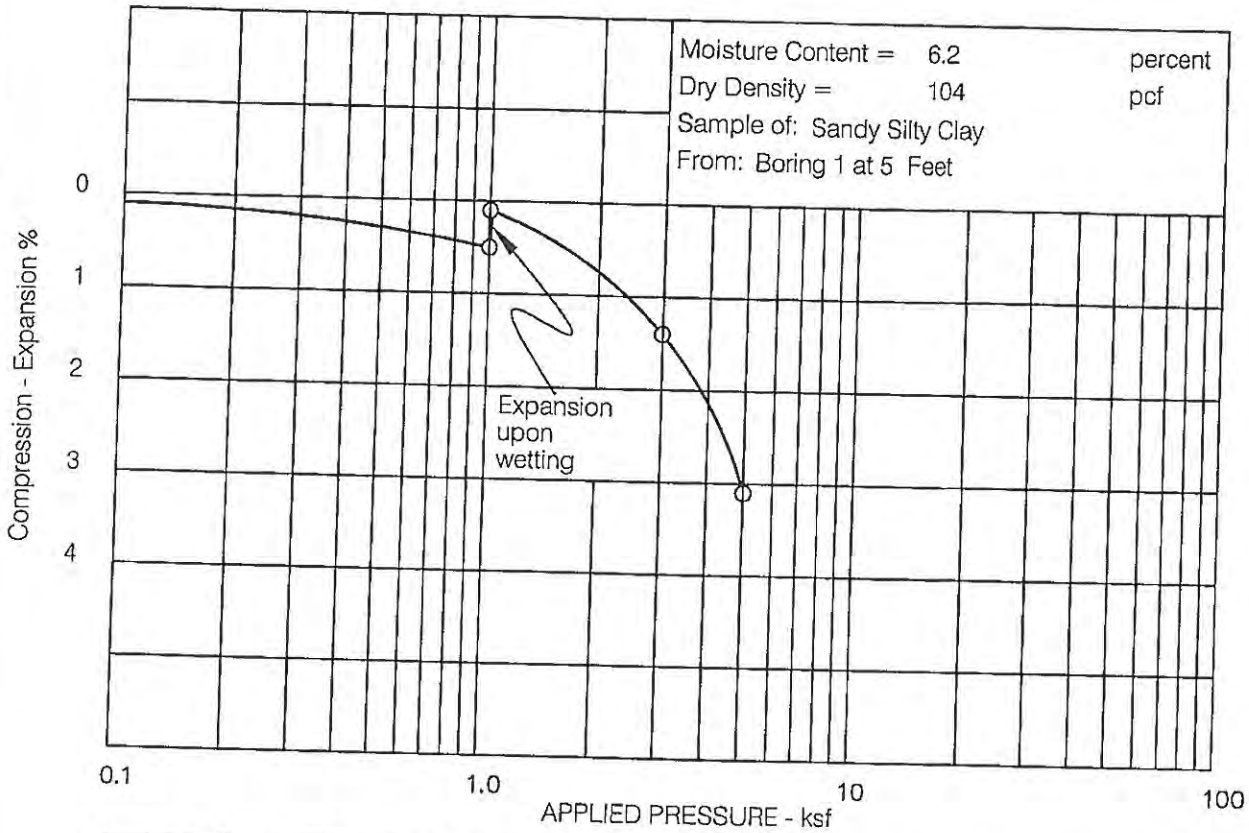
SILTSTONE/CLAYSTONE BEDROCK; weathered and medium hard to very hard with depth, thinly bedded, slightly moist, grey-brown, moderate bedding dip. Mancos Shale.

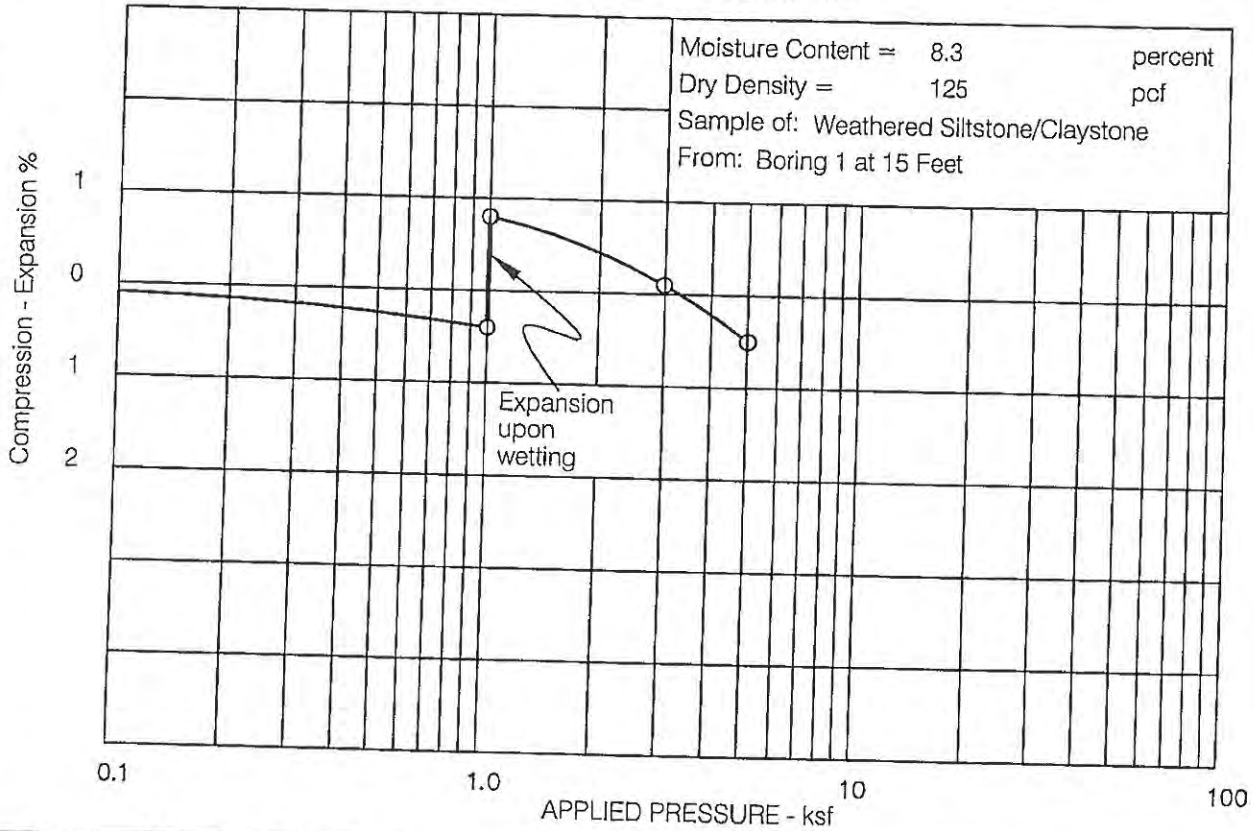
Relatively undisturbed drive sample; 2-inch I.D. California liner sample.

25/12 Drive sample blow count; indicates that 25 blows of a 140 pound hammer falling 30 inches were required to drive the California sampler 12 inches.

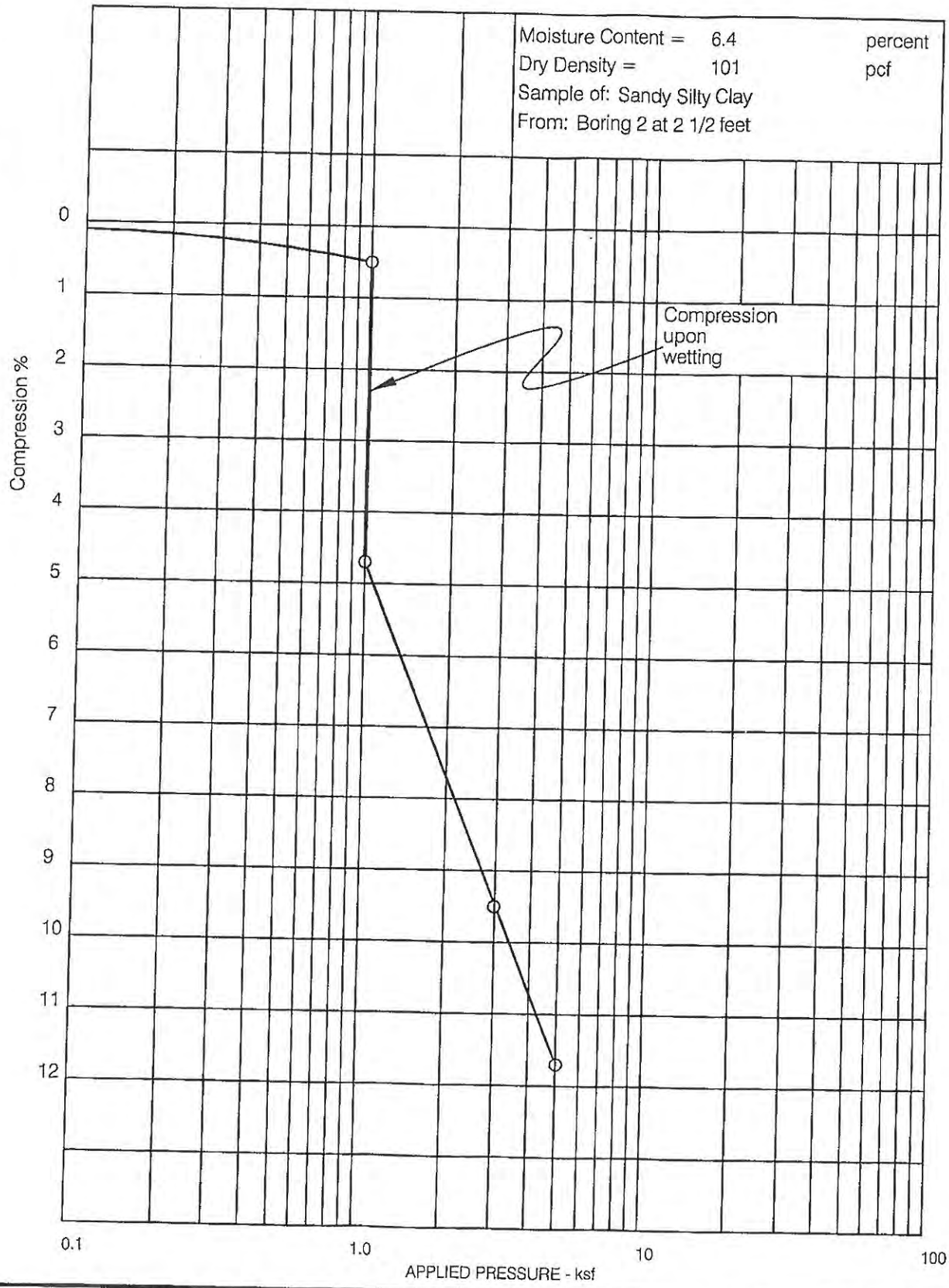
NOTES:

1. Exploratory borings were drilled on January 26, 2015 with 4-inch diameter continuous flight power auger.
2. Locations of exploratory borings were measured approximately by pacing from features shown on the site plan provided.
3. Elevations of exploratory borings were obtained by interpolation between contours shown on the site plan provided and the relative elevations checked by instrument level.
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. No free water was encountered in the borings at the time of drilling. Fluctuation in water level may occur with time.
7. Laboratory Testing Results:
   WC = Water Content (%)
   DD = Dry Density (pcf)
   -200 = Percent passing No. 200 sieve
   LL = Liquid Limit (%)
   PI = Plasticity Index (%)

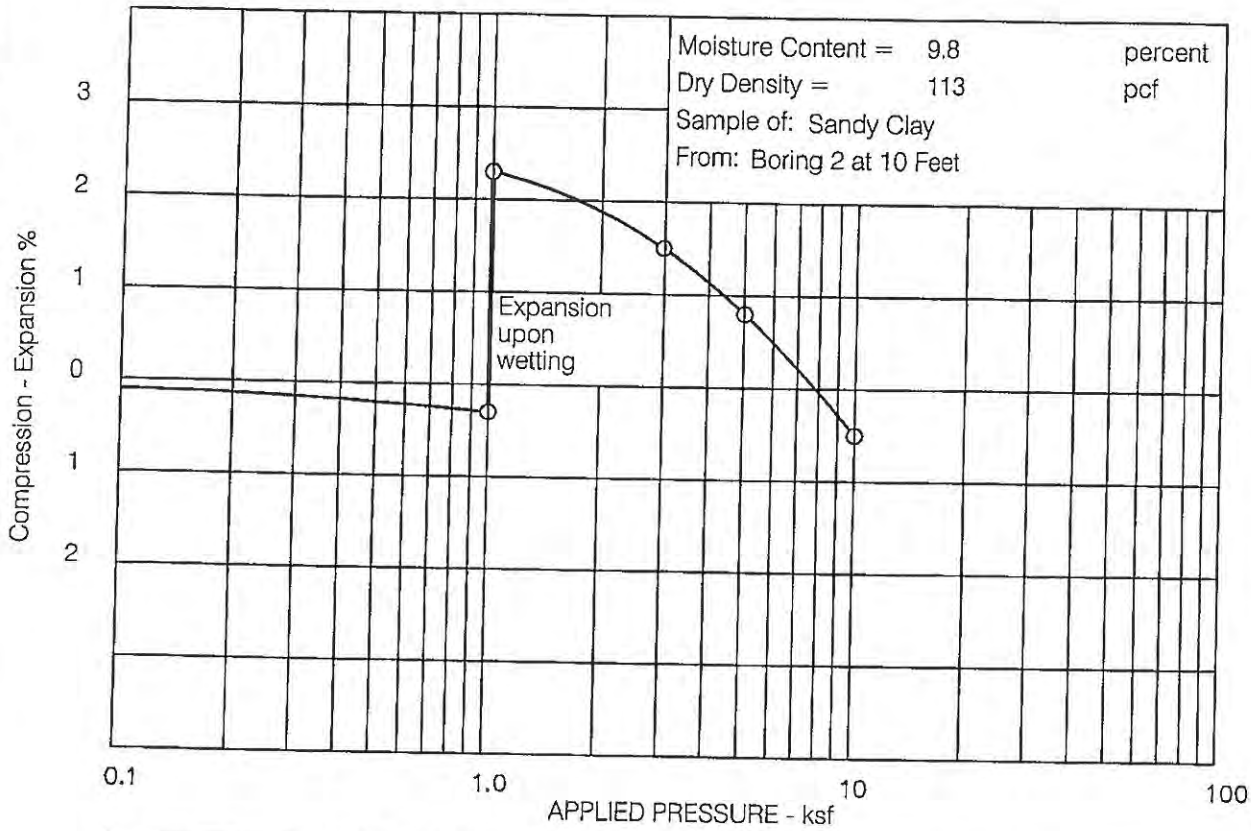


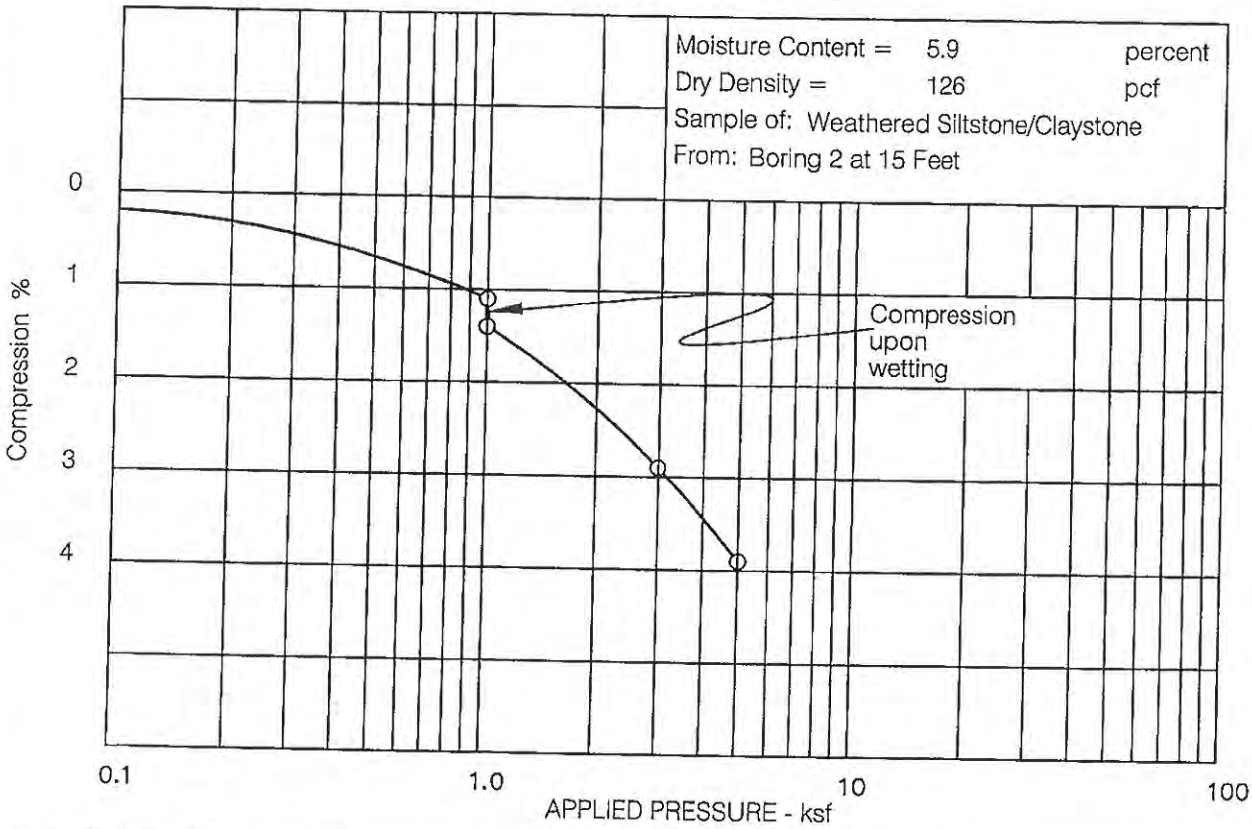


115 014A	HP Geotech Hepworth–Pawlak Geotechnical	SWELL-CONSOLIDATION TEST RESULTS	Figure 4

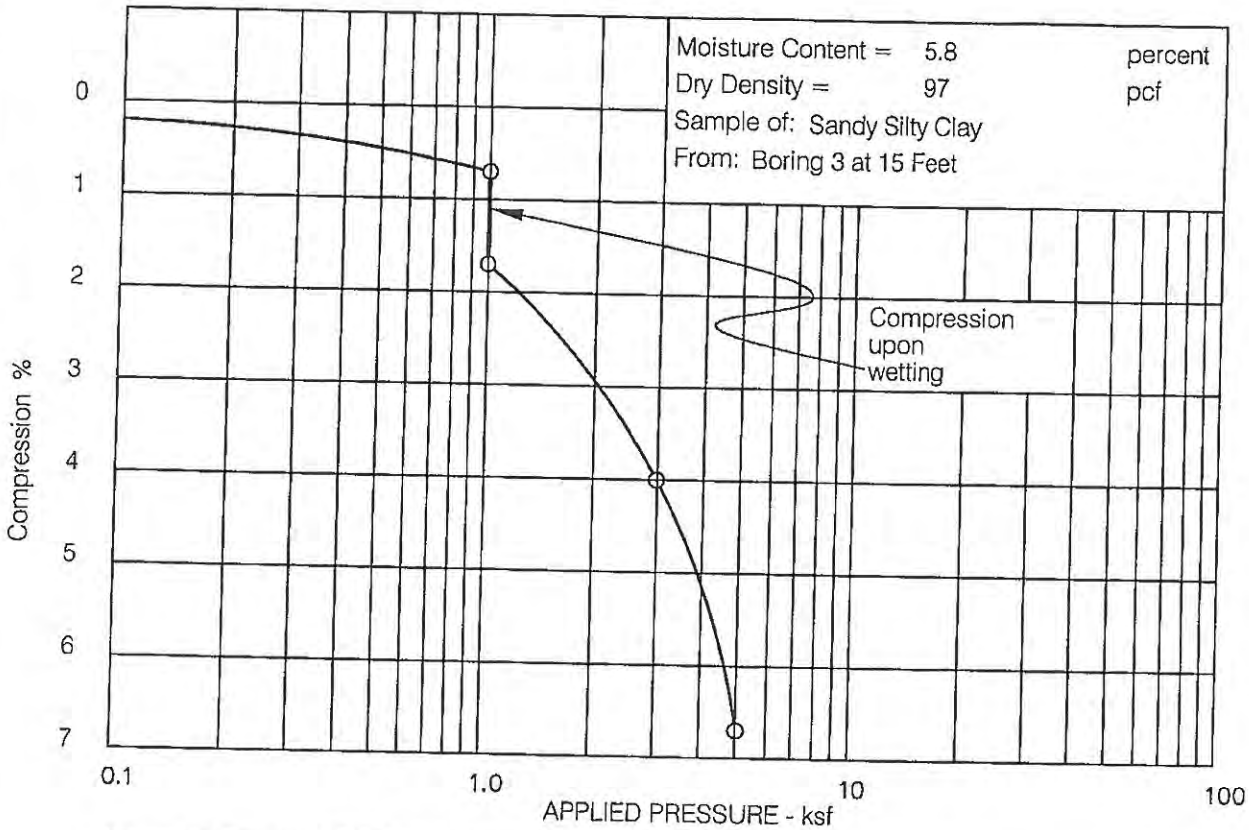


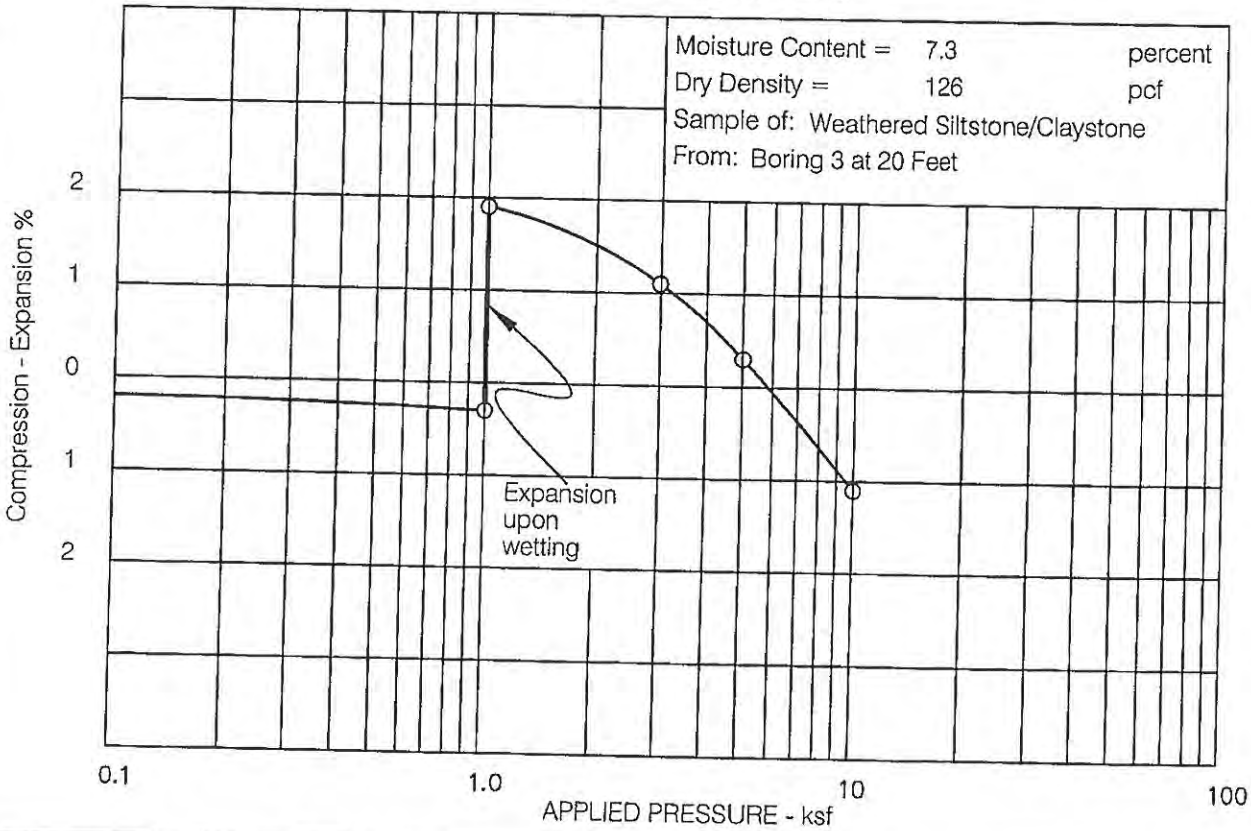
115 014A	HP Geotech Hepworth–Pawlak Geotechnical	SWELL-CONSOLIDATION TEST RESULTS	Figure 5





115 014A	HP Geotech Hepworth-Pawlak Geotechnical	SWELL-CONSOLIDATION TEST RESULTS	Figure 6





115 014A	HP Geotech Hepworth–Pawlak Geotechnical	SWELL-CONSOLIDATION TEST RESULTS	Figure 7

HEPWORTH-PAWLAK GEOTECHNICAL, INC.

TABLE 1

SUMMARY OF LABORATORY TEST RESULTS

Job No. 115 014A

SAMPLE LOCATION		NATURAL MOISTURE CONTENT	NATURAL DRY DENSITY	GRADATION		PERCENT PASSING NO. 200 SIEVE	ATTERBERG LIMITS		AASHTO CLASSIFICATION		SOIL OR BEDROCK TYPE
BORING	DEPTH (ft)	(%)	(pcf)	GRAVEL (%)	SAND (%)		LIQUID LIMIT (%)	PLASTIC INDEX (%)			
1	2½	4.5	103			75	26	9	A-4 (4)		Sandy Silty Clay
	5	6.2	104								Sandy Silty Clay
	15	8.3	125								Weathered Siltsone/Claystone
2	2½	6.4	101								Sandy Silty Clay
	5	5.7	107			90	26	10	A-4 (7)		Slightly Sandy Silty Clay
	10	9.8	113								Sandy Clay
	15	5.9	126								Weathered Siltsone/Claystone
3	5	6.6	97								Sandy Silty Clay
	10	8.2	97			93	28	10	A-4 (8)		Slightly Sandy Silty Clay
	15	5.8	97								Sandy Silty Clay
	20	7.3	126								Weathered Siltsone/Claystone