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SUBSOIL STUDY
FOR FOUNDATION AND PAVEMENT DESIGNS
PROPOSED RIFLE FIRE STATION #3
LOT 18, GRAND RIVER PLAZA
LAST CHANCE DRIVE
RIFLE, COLORADO

JOB NO. 109 001A

JANUARY 30, 2009

PREPARED FOR:

RIFLE FIRE PROTECTION DISTRICT
ATTN: MIKE MORGAN, FIRE CHIEF
1850 RAILROAD AVENUE
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PURPOSE AND SCOPE OF STUDY

This report presents the results of a subsoil study for the proposed Rifle Fire Station #3 to be located on Lot 18, Grand River Plaza, Last Chance Drive, Rifle, Colorado. The project site is shown on Figure 1. The purpose of the study was to develop recommendations for the foundation and pavement designs. The study was conducted in accordance with our proposal for geotechnical engineering services to the Rifle Fire Protection District dated January 8, 2009. The geologic and subsurface conditions and their potential impacts to the Grand River Plaza subdivision (formerly known as the Gould subdivision) were presented in a preliminary geotechnical study report prepared by this office (Hepworth-Pawlak Geotechnical, 2003).

A field exploration program consisting of exploratory borings was conducted at the subject lot 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 recommendations for foundation types, depths and allowable pressures for the proposed building foundation and pavement section thicknesses. 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.

PROPOSED CONSTRUCTION

The Rifle Fire Station #3 facility will consist of a new building on the west half of the lot with asphalt and concrete drive areas to the east as shown on Figure 1. The proposed building will be a single story structure consisting of masonry construction for the Apparatus Bay (south half) and wood frame construction with masonry veneer and stucco finish for the remainder. Ground floors will be slab-on-grade. Grading is assumed to be relatively minor for the building with cut depths between about 3 to 8 feet and relatively extensive for the south end of the concrete drive area with cuts up to 12 feet and retained by a cast-in-place concrete retaining wall. We assume relatively light to moderate foundation loadings, typical of the proposed type of construction. We understand that the pavement areas will be subjected to a maximum vehicle loading of 80,000 lbs.

If the proposed development plan, structure loadings 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 lot was vacant and is located on the south (uphill) side of Last Chance Drive in the southwest corner of the Grand River Plaza subdivision as shown on Figure 1. Lot 17 to the east was undeveloped and there was residential development to the west. The Last Chance Ditch is located to the north (downhill) of the lot. The ground surface was covered with between 12 to 18 inches of snow at the time of our field exploration. Previous site grading consisted of minor filling at the north part of the lot and an unretained cut slope at the south and west sides of the lot to construct a relatively flat/level building area during the subdivision development. The cut slope appeared to be graded at about 1½ horizontal to 1 vertical with a maximum height of 30 feet at the southwest corner of the lot and is similar to that proposed at the time of our previous study (Hepworth-Pawlak Geotechnical, 2003). Steep natural valley side slopes continue up above the cut slope. Vegetation visible above the snow cover consisted of scattered weeds on the lot. Cobbles, boulders and outcrops of the Wasatch Formation were visible on the natural valley side slopes above the lot.

FIELD EXPLORATION

The field exploration for the project was conducted on January 12, 2009. Three exploratory borings were drilled at the locations shown on Figure 1 to evaluate the 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 and monitored for groundwater level by a representative of Hepworth-Pawlak Geotechnical, Inc.

Samples of the subsoils and bedrock were taken with a 2 inch I.D. spoon sampler. The sampler was driven into the subsoils and bedrock 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 generally consist of relatively dense, clayey to silty sandy gravel and cobbles with boulders overlying silty gravelly sand and clay with scattered cobbles. The sand and clay layer is typically medium dense /stiff to very stiff and extends down to the maximum explored depth of 50 feet at Boring 1. Siltstone bedrock (Wasatch Formation) was encountered below the sand and clay at a depth of 18½ feet in Boring 2 down to the bottom depth of 21 feet. Drilling in the upper, coarse granular soils with auger equipment was difficult due to the cobbles and boulders and drilling refusal was encountered in the deposit at Borings 2 and 3.

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 finer graded soils, presented on Figures 4 and 5, generally indicate low to moderate compressibility under conditions of loading and wetting. Two of the samples showed a minor to low collapse potential (settlement under constant load) after wetting. Results of gradation analyses performed on a small diameter drive sample (minus 1½ inch fraction) of the upper, coarse granular soils are shown on Figure 6. The results of an Atterberg limits test indicate that the matrix of the upper coarse granular soils has low plasticity. The laboratory testing is summarized in Table 1.

Free water was measured in Boring 1 at a depth of 25 feet when checked on January 15, 2009. The remaining borings were dry down to their respective cave depths. The upper soils were typically slightly moist to moist.

FOUNDATION BEARING CONDITIONS

The natural subsoils encountered in the borings typically consist of matrix supported alluvial fan deposits which vary from clayey to silty sandy gravel and cobbles with boulders to silty gravelly sand and clay with scattered cobbles. These soils are typically compressible under load, and possess a minor to low collapse potential if subjected to post-construction wetting. The rock content of these soils tends to lower the collapse potential. Considering the subsurface conditions encountered in the exploratory borings and the nature of the proposed construction, it appears suitable to support the proposed building on a shallow spread footing foundation bearing on the natural soils with some risk of long-term foundation settlement and building distress. Settlement would be differential across the building and depend on the specific foundation loading and bearing conditions. Additional post-construction differential foundation settlement could occur if the bearing soils are subjected to wetting, especially between continuous foundation walls and isolated column footing pads. In general, a conventionally reinforced mat foundation would help reduce the potential for differential foundation settlement and building distress. Recommendations for spread footing and mat foundations are presented in the *Design Recommendations* section of this report.

As an alternative to achieve a low settlement risk, it may be feasible to support the building on a deep foundation system, such as driven piles or drilled piers, which extends the bearing level down to bedrock. A deep foundation system would provide moderate to high load capacity with a relatively low settlement potential. The depth to a suitable bearing strata required for a deep foundation at the proposed building site is variable. Specifically, at least 50 feet of alluvial fan deposits were encountered at Boring 1 and hard siltstone bedrock was encountered at a depth of 18½ feet at Boring 2. Additional subsurface exploration would be needed to verify the feasibility of a deep foundation system at the site and provide foundation design recommendations. Based on our experience in the area, bedrock or relatively dense river gravel alluvium could underlie the alluvial fan deposits at the north part of the lot on the order of 55 to 60 feet below the current ground surface elevation. If a deep foundation system is proposed, we should be contacted to provided addition exploration and recommendations.

DESIGN RECOMMENDATIONS

SPREAD FOOTING AND MAT FOUNDATIONS

The design and construction criteria presented below should be observed for a spread footing or conventionally reinforced mat foundation systems.

1) Footings or a structural mat placed on the undisturbed natural soils should be designed for an allowable bearing pressure of 2,000 psf. Based on experience, we expect initial settlement of foundations designed and constructed as discussed in this section will be about 1 inch or less. Settlement would be differential across the building and depend on the specific loading and bearing conditions. Additional post-construction differential foundation settlement is estimated to be up to 1½ inches assuming a wetted depth of about 10 feet below the building foundation. Actual settlements could be more if the wetting is more extensive than that described in this report. Precautions should be taken to prevent post-construction wetting of the bearing soils.
2) Footings should have a minimum width of 18 inches for continuous walls and 2 feet for isolated pads.
3) Exterior footings, edges of mats 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.

4) Continuous foundation walls should be reinforced top and bottom to span local anomalies such as by assuming an unsupported length of at least 12 feet. 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.

5) All existing fill, topsoil and any loose or disturbed soils should be removed and the footing bearing level extended down to the undisturbed natural soils. The exposed soils in foundation bearing areas should be moistened and compacted prior to concrete placement.

6) A representative of the geotechnical engineer should observe all foundation excavations prior to concrete placement to evaluate bearing conditions.

FOUNDATION AND RETAINING WALLS

Foundation walls and retaining structures, up to about 15 feet tall, 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 50 pcf for backfill consisting of the on-site predominantly granular 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 40 pcf for backfill consisting of the on-site predominantly granular soils. Backfill should not contain topsoil, vegetation, debris or rock larger than about 6 inches.

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 equivalent fluid unit weight of at least 20 pcf should be added to the earth pressure loadings given above for a 1½ horizontal to 1 vertical backslope above the walls. An underdrain system 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 in pavement and walkway areas should be compacted to at least 95% of the maximum standard Proctor density. Care should be taken not to over compact 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 0.40. Passive pressure of compacted backfill against the sides of the footings can be calculated using an equivalent fluid unit weight of 350 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 a compacted to at least 95% of the maximum standard Proctor density at near optimum moisture content.

NON-STRUCTURAL FLOOR SLABS

The natural on-site soils, exclusive of topsoil and existing fill, are suitable to support lightly to moderately loaded slab-on-grade construction. We understand that the concrete floor slab in the Apparatus Bay will support a maximum vehicle load on 80,000 pounds. Based on pavement section design analysis, the concrete floor slab in the Apparatus Bay should consist of at least 6 inches of concrete on 4 inches of road base. The concrete should have a minimum design compressive strength of 4,000 psi. The slab sections could also be analyzed as a structural beam supported by a subgrade modulus of 230 pci. Slabs-on-grade in other parts of the building should be designed according to their respective loading.

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 relatively well graded sand and gravel should be placed beneath interior slabs. This material should consist of minus 2 inch aggregate with at least 50% retained on the No. 4 sieve and less than 12% 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 near optimum moisture content. Required fill can consist of the on-site soils devoid of vegetation, topsoil, debris and oversized rock.

UNDERDRAIN SYSTEM

Free water was encountered during our exploration in Boring 1 below the expected excavation depth for the building, but it has been our experience in mountainous areas 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 uphill foundation walls cut into the hillside and cantilevered retaining walls be protected from wetting and hydrostatic pressure buildup by an underdrain system.

The drains should consist of drainpipe 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

The risk of construction-induced slope instability at the site appears low provided the building is located as shown on Figure 1, and cut and fill depths are limited. We assume that excavation into the existing cut slope at the south and west sides of the lot will not exceed one level, about 8 to 12 feet. We did not observe signs of slope instability along the existing cut slope at the time of our field exploration, but the slope was covered with snow. Fills should be limited to about 8 to 10 feet deep. Embankment fills should be 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. The fill should be benched into the portions of the hillside exceeding 20% grade.

Permanent unretained cut and fill slopes should be graded at 1½ horizontal to 1 vertical or flatter and protected against erosion by revegetation or other means. The risk of slope instability will be increased if seepage is encountered in cuts and flatter slopes may be necessary. If seepage is encountered in permanent cuts, an investigation should be conducted to determine if the seepage will adversely affect the cut stability. This office should review site grading plans for the project prior to construction.

SURFACE DRAINAGE

The following drainage precautions should be observed during construction and maintained at all times after the building 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 about 2 feet of the on-site finer-grained soils to reduce surface water infiltration. A swale will be needed above the proposed retaining wall at the south side of the concrete drive area and west side of the proposed building to divert surface runoff around these structures.

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.

PAVEMENT SECTION THICKNESS

We understand asphalt and concrete drive areas are proposed at the site as shown on Figure 1. A maximum vehicle loading of 80,000 pounds was provided for use in our analysis. The subgrade soils encountered at the site generally consist of low plasticity clayey to silty sandy gravel with cobbles that are considered a fair support for pavement sections. Based on the subgrade soils encountered at the site and our experience in the area, the following values in our analyses: a Hveem 'R' Value of 20, a subgrade modulus of 230 pci, an 18 kip EDLA of 30 for main drive areas and 5 for areas limited to passenger vehicle traffic only, a Regional Factor of 1.5 and a serviceability index of 2.0. Base on the parameters listed above and our experience with similar projects, we recommend the minimum asphalt pavement section thickness consist of 4 inches of asphalt on 8 inches of base course in the main drive areas and 3 inches of asphalt on 6 inches of base course in areas limited to passenger vehicle traffic only. The concrete pavement section thickness should consist of 6 inches of portland cement concrete over 4 inches of base course.

The asphalt should be a batched hot mix, approved by the engineer and placed and compacted to the project specifications. The base course should meet CDOT Class 6 specifications. All base course and required subgrade fill should be adjusted to within 2% of optimum moisture content and compacted to at least 95% of the maximum standard Proctor density. Concrete should have a minimum design compressive strength of 4,000 psi and contain sufficient entrained air.

Required fill to establish design subgrade level can consist of the on-site soils or suitable imported granular soils approved by the geotechnical engineer. Prior to fill placement the subgrade should be scarified to a depth of 8 inches, adjusted to near optimum moisture and compacted to at least 95% of standard Proctor density. Any soft or wet subgrade areas 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 proof-rolled and areas that deflect excessively should be corrected before placing pavement materials. Any subgrade improvements and placement and compaction of pavement materials should be monitored on a regular basis by a representative of the geotechnical engineer. If traffic loadings differ from those presented above, 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 construction, structure loadings 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 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
TREVOR L. KNELL
38469
2-10-09
PROFESSIONAL ENGINEER

Trevor L. Knell, P.E.

Reviewed by:

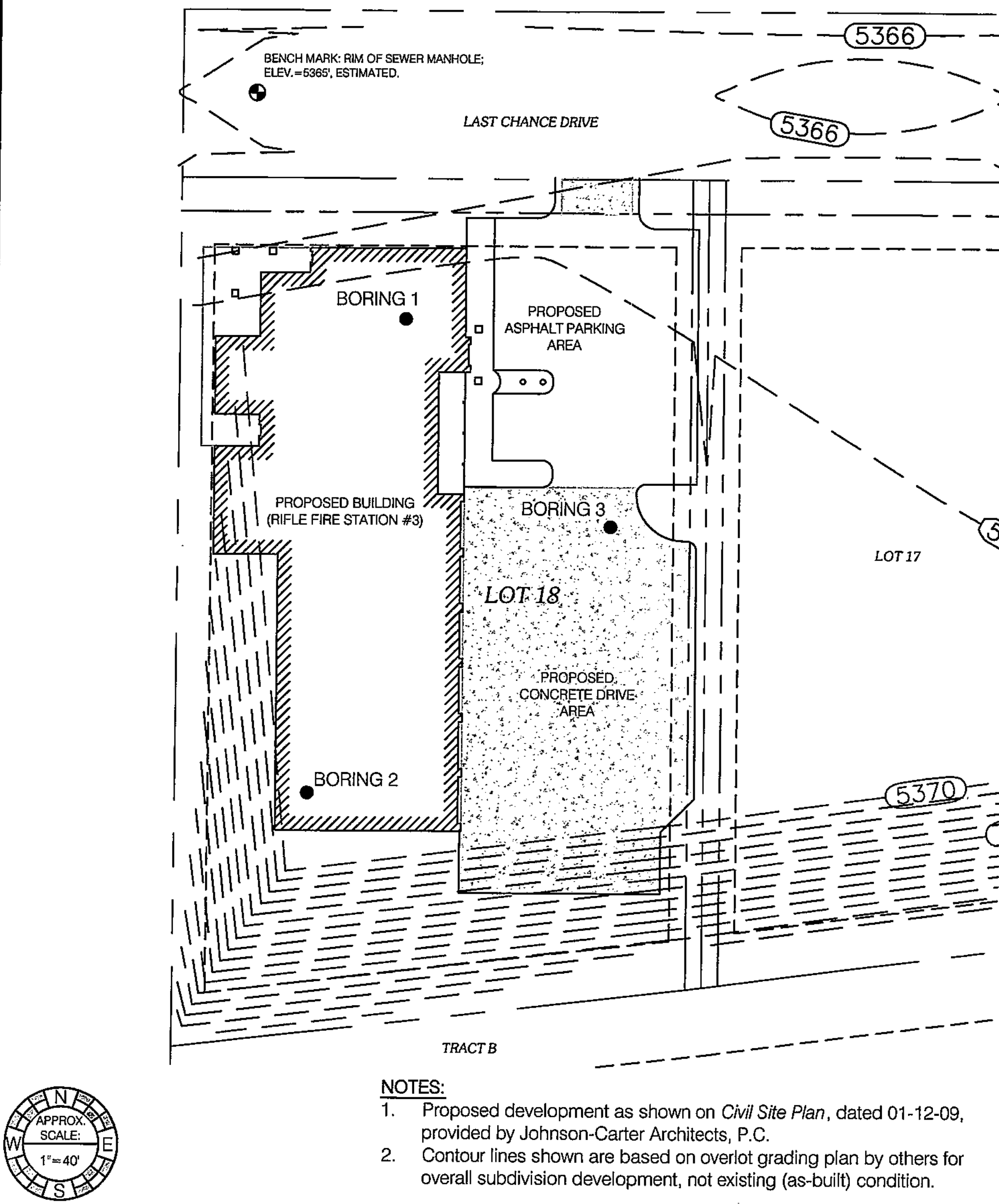
Steven L. Pawlak, P.E.

TLK/vam

cc: Johnson-Carter Architects, PC – Attn: Richard Carter

REFERENCE

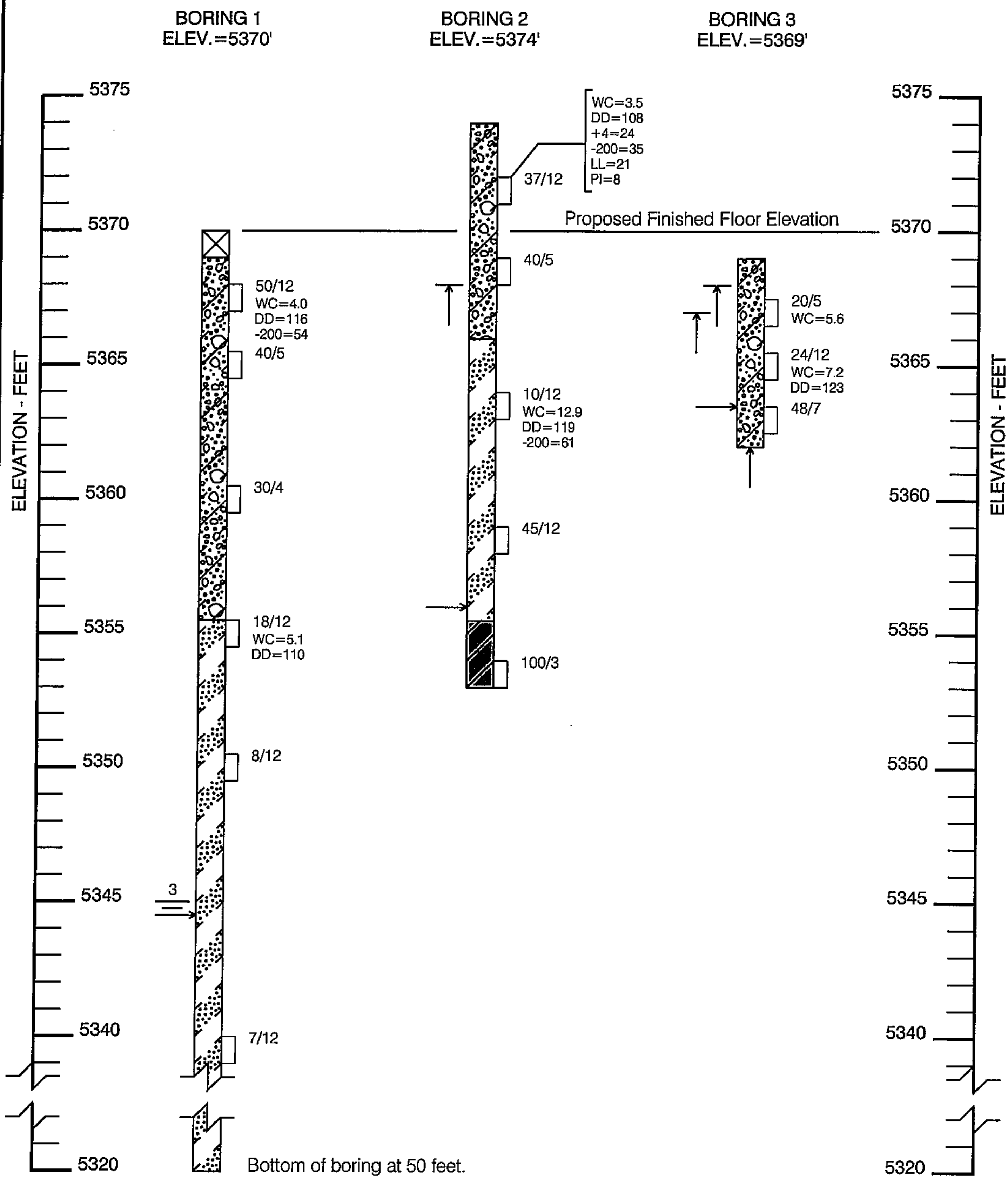
Hepworth-Pawlak Geotechnical, Inc., 2003. *Preliminary Geotechnical Study, Proposed Gould Subdivision, South of Taugenbaugh Boulevard and Airport Road, Rifle, Colorado.* Prepared for Gould Construction, Dated April 30, 2003, Job No. 102 779.



NOTES:

1. Proposed development as shown on *Civil Site Plan*, dated 01-12-09, provided by Johnson-Carter Architects, P.C.
2. Contour lines shown are based on overlot grading plan by others for overall subdivision development, not existing (as-built) condition.

109 001A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	LOCATIONS OF EXPLORATORY BORINGS	FIGURE 1



Note: Explanation of symbols is shown on Figure 3.

109 001A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	LOGS OF EXPLORATORY BORINGS	FIGURE 2

LEGEND:

FILL; clayey sand and gravel with scattered cobbles, medium dense, slightly moist, light brown, likely derived from on-site soils and placed during overlot grading operations.

SAND AND CLAY (SC-CL); stratified, silty, gravelly, with scattered cobbles, medium dense/stiff to very stiff, moist to wet with depth at Boring 1, brown.

GRAVEL AND COBBLES (GC-GM); stratified, clayey to silty, sandy to very sandy zones, with boulders, medium dense, slightly moist to moist, brown, low plasticity fines.

SILTSTONE BEDROCK; weathered to very hard, slightly moist, brown, Wasatch Formation.

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

50/12 Drive sample blow count; indicates that 50 blows of 140 pound hammer falling 30 inches were required to drive the California sampler 12 inches.

Practical drilling refusal. Where shown above the bottom of the log indicates that multiple attempts were made to advance the boring.

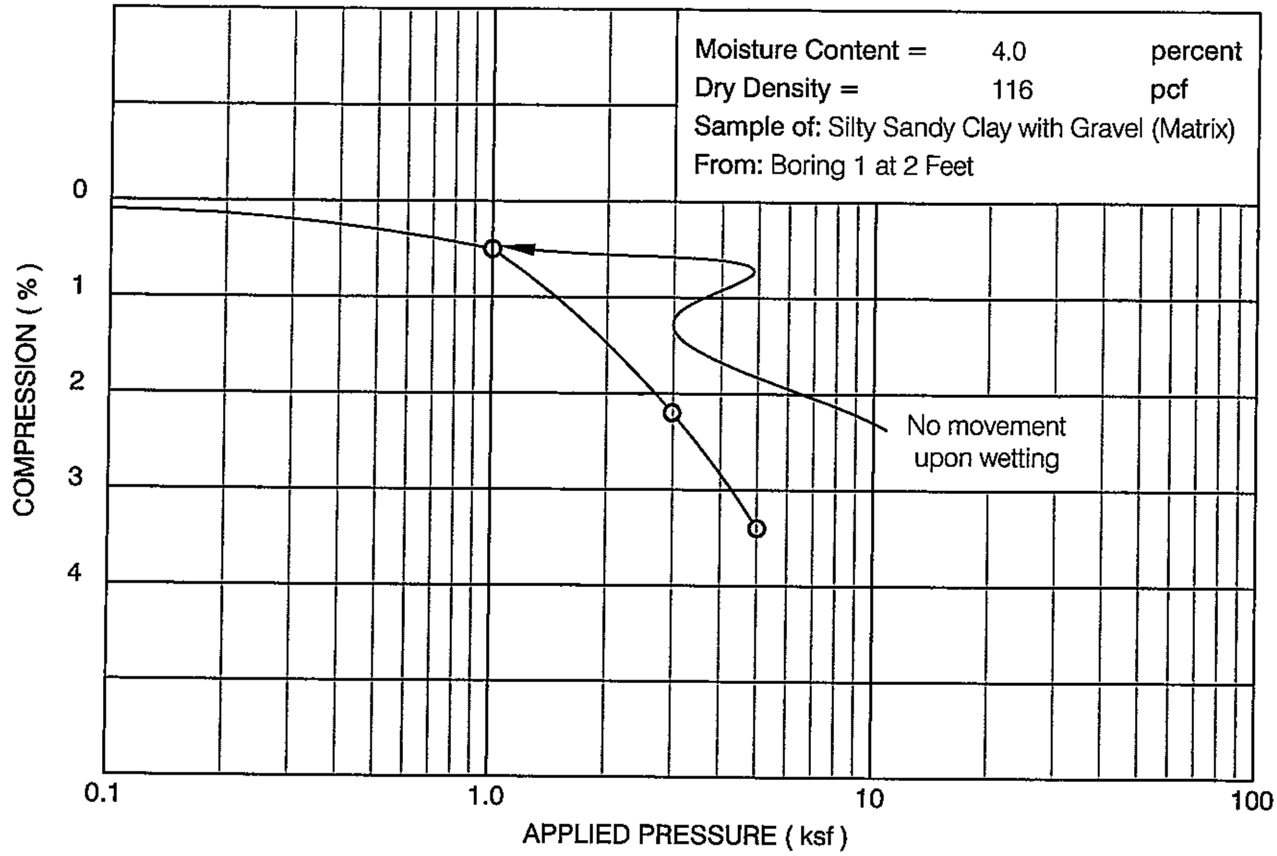
⟶ Depth at which boring had caved when checked on January 15, 2009.

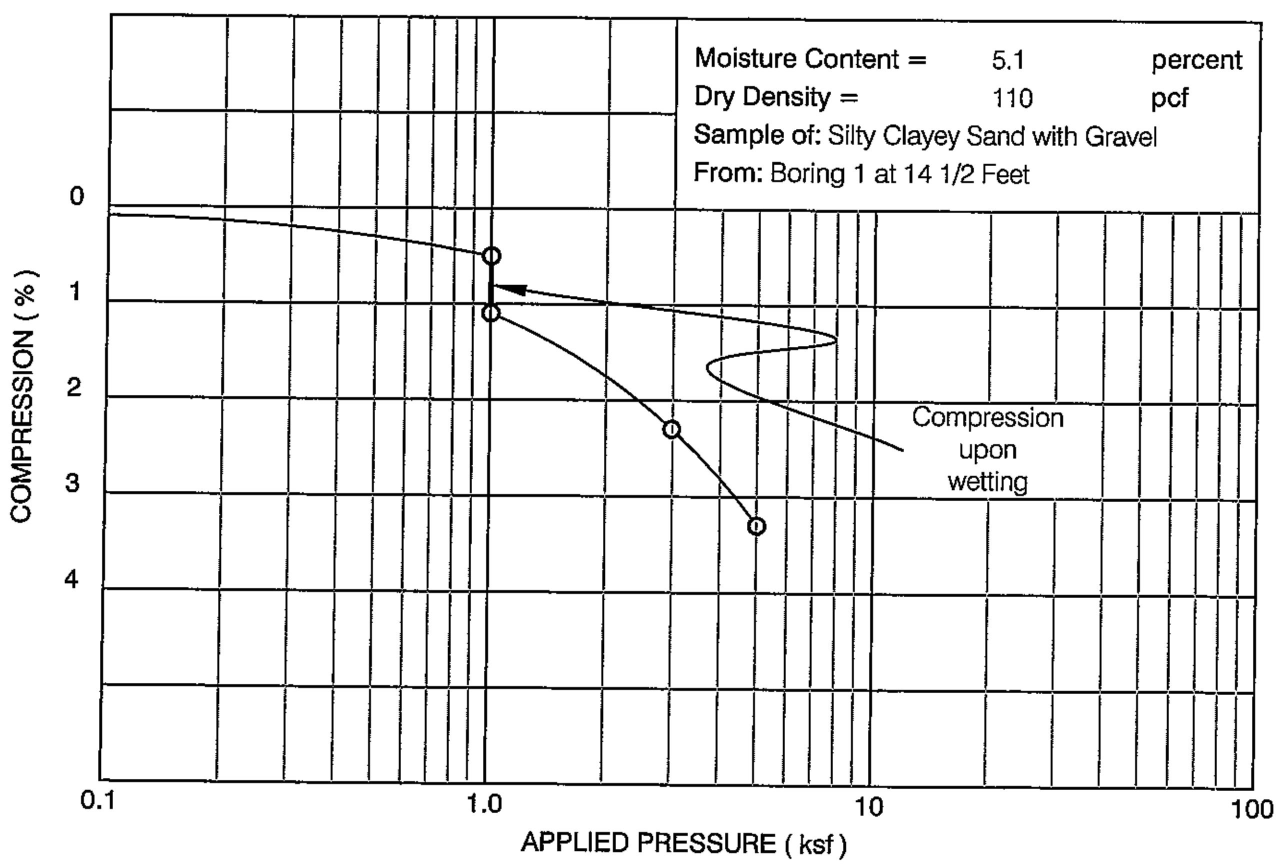
0,3 Free water level in boring and number of days following drilling measurement was taken.

NOTES:

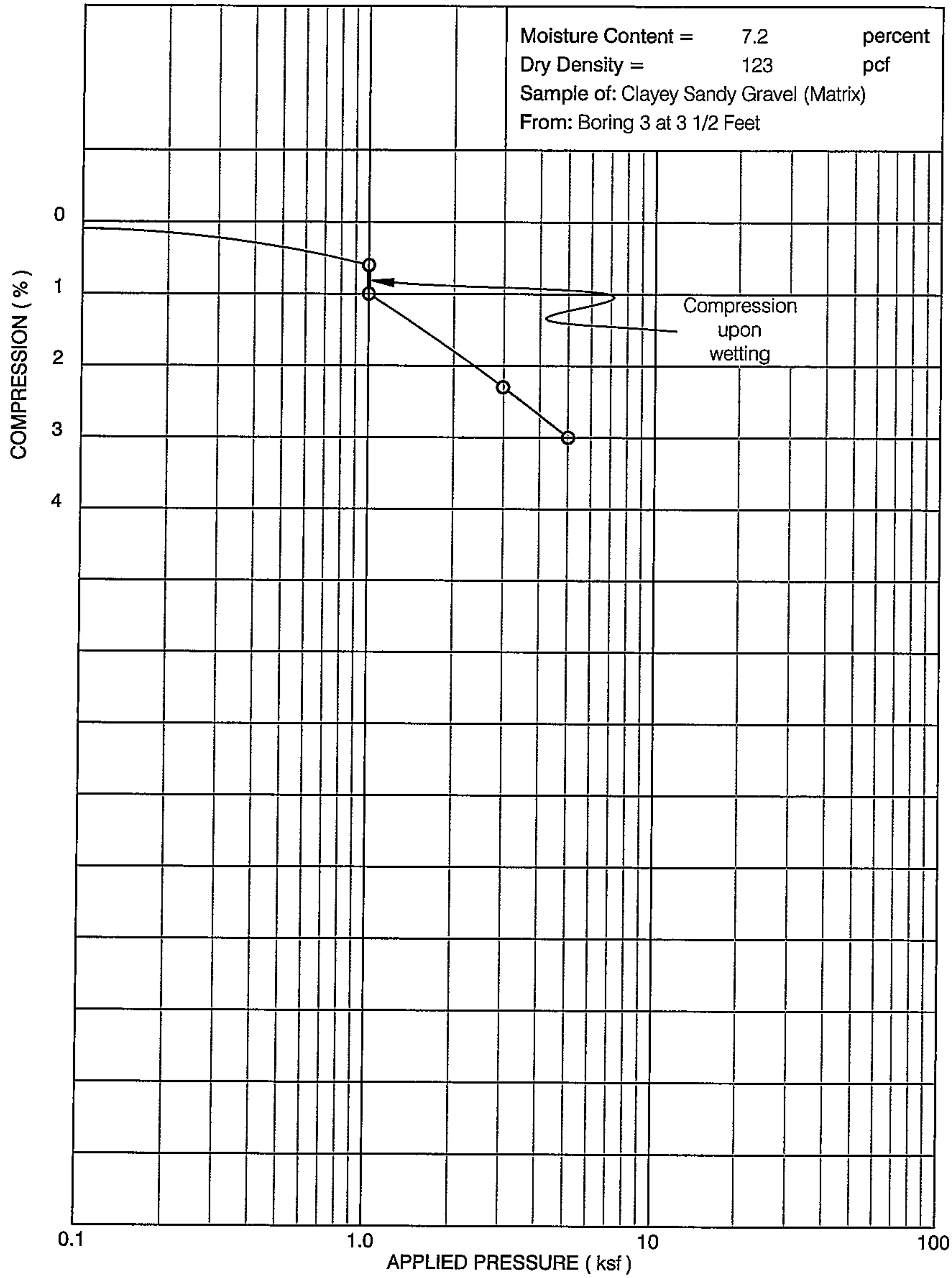
1. Exploratory borings were drilled on January 12, 2009 with 4-inch diameter continuous flight power auger.
2. Locations of exploratory borings were measured approximately by pacing from features shown on the site plan provided.
3. Elevations of exploratory borings were measured by instrument level and refer to the Bench Mark shown on Figure 1.
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. Free water was encountered in Boring 1. The other borings were dry at the time of drilling and when checked 3 days later. Fluctuation 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 (%)

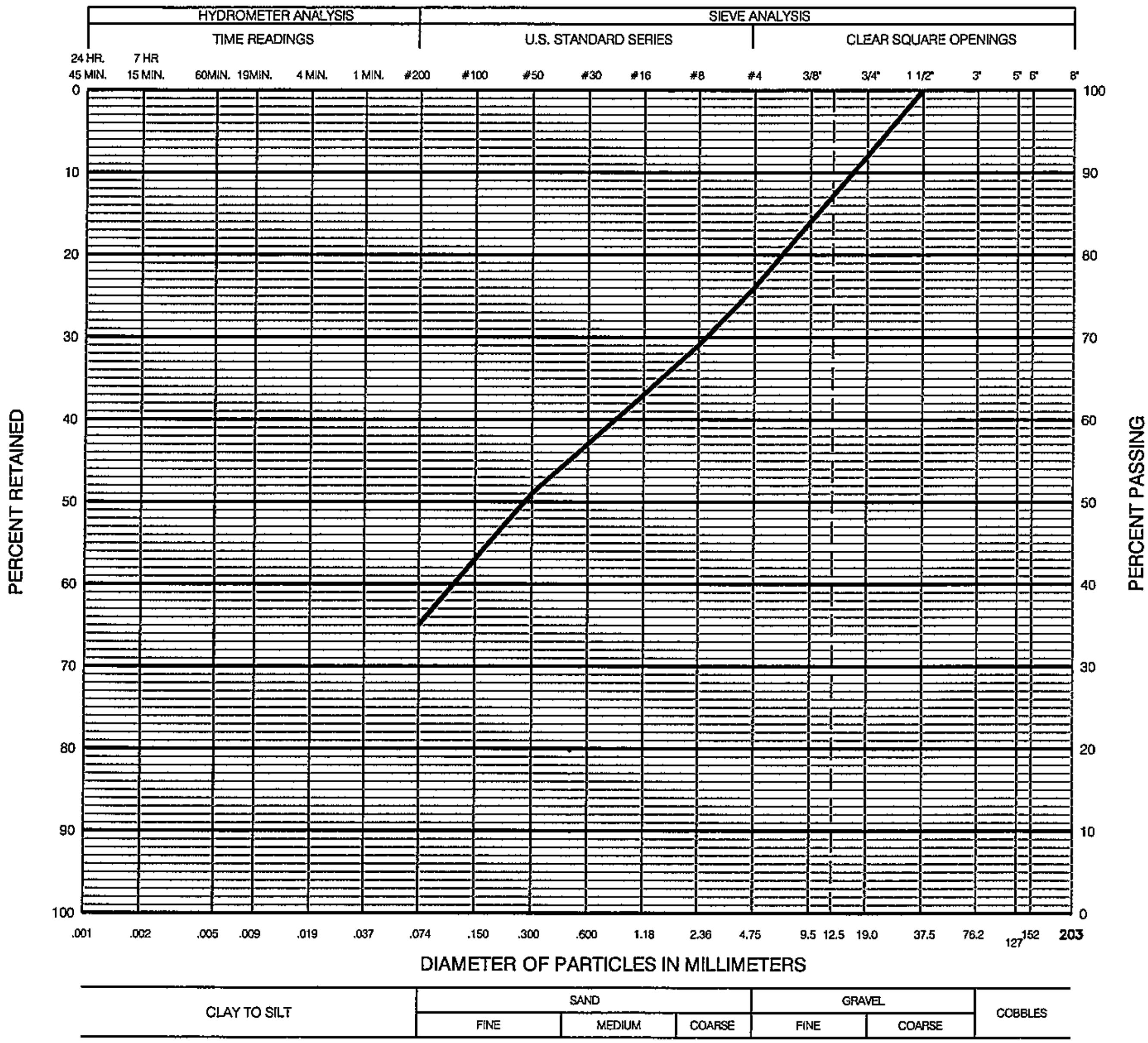




109 001A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	SWELL-CONSOLIDATION TEST RESULTS	FIGURE 4



109 001A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	SWELL-CONSOLIDATION TEST RESULTS	FIGURE 5



Gravel 24 % Sand 41 % Silt and Clay 35 %

Liquid Limit 21 % Plasticity Index 8 %

Sample of: Silty Sand and Clay with Gravel (Matrix) From: Boring 2 at 2 Feet

109 001A	HP Geotech HEPWORTH-PAWLAK GEOTECHNICAL	GRADATION TEST RESULTS	FIGURE 6

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TABLE 1
SUMMARY OF LABORATORY TEST RESULTS

Job No. 109 001A

SAMPLE LOCATION		NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (pcf)	GRADATION		PERCENT PASSING NO. 200 SIEVE	ATTERBERG LIMITS		AASHTO CLASSIFCATION		SOIL TYPE
BORING NO.	DEPTH (ft)			GRAVEL (%)	SAND (%)		LIQUID LIMIT (%)	PLASTIC INDEX (%)			
1	2	4.0	116			54					Silty sandy clay with gravel (matrix)
	14½	5.1	110								Silty clayey sand with gravel
2	2	3.5	108	24	41	35	21	8	A-2-4(10)		Silty sand and clay with gravel (matrix)
	10	12.9	119			61					Very sandy clay with gravel
3	1½	5.6									Clayey sandy gravel (matrix)
	3½	7.2	123								Clayey sandy gravel (matrix)