HEPWORTH-PAWLAK GEOTECHNICAL, INC.

5020 Road 154
Glenwood Springs, CO 81601

Fax 970 945-8454
Phone 970 945-7988

SUBSOIL STUDY
FOR FOUNDATION DESIGN
PROPOSED COMMERCIAL BUILDING
LOT I-13, EAGLE VALLEY COMMERCIAL PARK
MARMOT LANE, EAGLE, COLORADO

JOB NO. 198 831

JANUARY 12, 1999

PREPARED FOR:

KAREN AND DAVID SHAW
P.O. BOX 1451
AVON, COLORADO 81620

TABLE OF CONTENTS

PURPOSE AND SCOPE OF STUDY 1

PROPOSED CONSTRUCTION 1

SITE CONDITIONS 2

SUBSIDENCE POTENTIAL 2

FIELD EXPLORATION 3

SUBSURFACE CONDITIONS 3

DESIGN RECOMMENDATIONS 4
FOUNDATIONS 4
FOUNDATION AND RETAINING WALLS 5
FLOOR SLABS 6
UNDERDRAIN SYSTEM 6
SURFACE DRAINAGE 7

LIMITATIONS 7

FIGURE 1 - LOCATION OF EXPLORATORY BORINGS

FIGURE 2 - LOGS OF EXPLORATORY BORINGS

FIGURE 3 - LEGEND AND NOTES

FIGURE 4 - SWELL-CONSOLIDATION TEST RESULTS

TABLE I - SUMMARY OF LABORATORY TEST RESULTS

PURPOSE AND SCOPE OF STUDY

This report presents the results of a subsoil study for a proposed commercial building to be located on Lot I-13, Eagle Valley Commercial Park, Marmot Lane, Eagle, Colorado. The project site is shown on Fig. 1. The purpose of the study was to develop recommendations for the foundation design. The study was conducted in accordance with our agreement for geotechnical engineering services to Karen and David Shaw dated December 6, 1998. Hepworth-Pawlak Geotechnical, Inc. previously performed a preliminary geotechnical study for the Eagle Valley Commercial Park, and presented our findings in a report dated November 29, 1995, Job No. 195 523.

A field exploration program consisting of exploratory borings was conducted to obtain information on 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. 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 subsoil conditions encountered.

PROPOSED CONSTRUCTION

The proposed commercial building will be a one and two story metal structure. The ground floor will be slab-on-grade. Excavation for the structure is assumed to be relatively minor with a maximum depth of about 3 to 4 feet below the existing ground surface. We assume relatively light foundation loadings, typical of the proposed type of construction.

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

SITE CONDITIONS

At the time of our field work, the site was vacant and covered with patches of snow up to about 2 inches deep. Prior to development of the commercial park, the area was used as irrigated grass field. Vegetation and topsoil was apparently removed during overlot grading. Existing vegetation consists of scattered grass and weeds. The ground surface is relatively flat and slopes gently down to the south. There is about 2 feet of elevation difference across the lot.

SUBSIDENCE POTENTIAL

Bedrock of the Pennsylvanian age Eagle Valley Evaporite underlies the Eagle area. These rocks are a sequence of gypsiferous shale, fine-grained sandstone and siltstone with some massive beds of gypsum and limestone. There is a possibility that massive gypsum deposits associated with the Eagle Valley Evaporite underlie the lot. Dissolution of the gypsum under certain conditions can cause sinkholes to develop and can produce areas of localized subsidence.

Sinkholes were not observed in the immediate area of the subject lot. No evidence of cavities was encountered in the subsurface materials; however, the exploratory borings were relatively shallow, for foundation design only. Based on our present knowledge of the subsurface conditions at the site, it cannot be said for certain that sinkholes will not develop. The risk of future ground subsidence on Lot I-13 throughout the service life of the proposed building, in our opinion, is low; however, the owner should be made aware of the potential for sinkhole development. If further investigation of possible cavities in the bedrock below the site is desired, we should be contacted.

FIELD EXPLORATION

The field exploration for the project was conducted on December 18, 1998. Two exploratory borings were drilled at the locations shown on Fig. 1 to evaluate the subsurface conditions. The borings were advanced with 4 inch diameter continuous flight augers powered by a truck-mounted Longyear BK-51HD drill rig. 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 subsoils at various depths with blows from a 140 pound hammer falling 30 inches. This test is similar to the standard penetration test described by ASTM Method D-1586. The penetration resistance values are an indication of the relative density or consistency of the subsoils. Depths at which the samples were taken and the penetration resistance values are shown on the Logs of Exploratory Borings, Fig. 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 Fig. 2. The subsoils consist of about 9 to 11½ feet of very stiff to hard, slightly sandy to sandy, clay and silt, overlying relatively dense, slightly silty, sandy gravel with cobbles and possible boulders. Drilling in the dense gravel with auger equipment was difficult due to the cobbles and possible boulders and drilling refusal was encountered in the deposit.

Laboratory testing performed on samples obtained from the borings included natural moisture content and density and finer than sand size gradation analyses. Results of consolidation testing performed on relatively undisturbed drive samples, presented on Fig. 4, indicate moderate compressibility when loaded and wetted. The laboratory testing is summarized in Table I.

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

DESIGN RECOMMENDATIONS

FOUNDATIONS

Considering the subsoil conditions encountered in the exploratory borings and the nature of the proposed construction, we recommend the building be founded with spread footings bearing on the natural soils. The fine grained soils tend to compress when loaded and wetted which could result in some post construction settlement if the soils were to become wetted. Heavily loaded or settlement sensitive structures can be founded on a deep foundation that penetrates the upper compressible soils and bears on the underlying dense gravel alluvium. We should be contacted for further recommendations if deep foundations are considered.

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

1) Footings placed on the undisturbed natural fine grained soils should be designed for an allowable soil bearing pressure of 1,500 psf. Based on experience, we expect settlement of footings designed and constructed as discussed in this section will be about 1 inch or less. If the subsoils become wetted, additional differential settlements on the order of ½ to 1½ inches could occur.
2) The footings should have a minimum width of 18 inches for continuous walls and 2 feet for isolated pads.
3) Exterior footings and footings beneath unheated areas should be provided with adequate soil cover above their bearing elevation for frost protection. Placement of foundations at least 42 inches below exterior grade is typically used in this area.
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 loose or disturbed soils should be removed and the footing bearing level extended down to firm natural soils.

6) A representative of the geotechnical engineer should 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 fine-grained 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 45 pcf for backfill consisting of the on-site fine-grained soils.

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

Backfill should be placed in uniform lifts and compacted to at least 90% of the maximum standard Proctor density at a moisture content near optimum. Backfill in pavement and walkway areas should be compacted to at least 95% of the maximum standard Proctor density. Care should be taken not to overcompact the backfill or use large equipment near walls, 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.

FLOOR SLABS

The natural on-site soils, exclusive of topsoil, are suitable to support lightly loaded slab-on-grade construction. To reduce the effects of some differential movement, 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 sand and gravel should be placed beneath interior slabs for subgrade support. 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 a moisture content near optimum. Required fill can consist of the on-site soils devoid of vegetation, topsoil and oversized rock.

UNDERDRAIN SYSTEM

Although free water was not encountered during our exploration, it has been our experience in the area that local perched groundwater may 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 and basement areas, be protected from wetting and hydrostatic pressure buildup by an underdrain system.

The drains should consist of drainpipe placed in the bottom of the wall backfill surrounded above the invert level with free-draining granular material. The drain should be placed at each level of excavation and at least 1 foot below lowest adjacent finish grade and sloped at a minimum 1% to a suitable gravity outlet. Free-draining granular material used in the underdrain system should contain less than 2% passing the

No. 200 sieve, less than 50% passing the No. 4 sieve and have a maximum size of 2 inches. The drain gravel backfill should be at least 1½ feet deep.

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 6 inches in the first 10 feet in unpaved areas and a minimum slope of 3 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 5 feet from foundation walls.

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 expressed or implied. The conclusions and recommendations submitted in this report are based upon the data obtained from the exploratory borings drilled excavated at the locations indicated on Fig. 1, the proposed type of construction and our experience in the area. 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.

Sincerely,

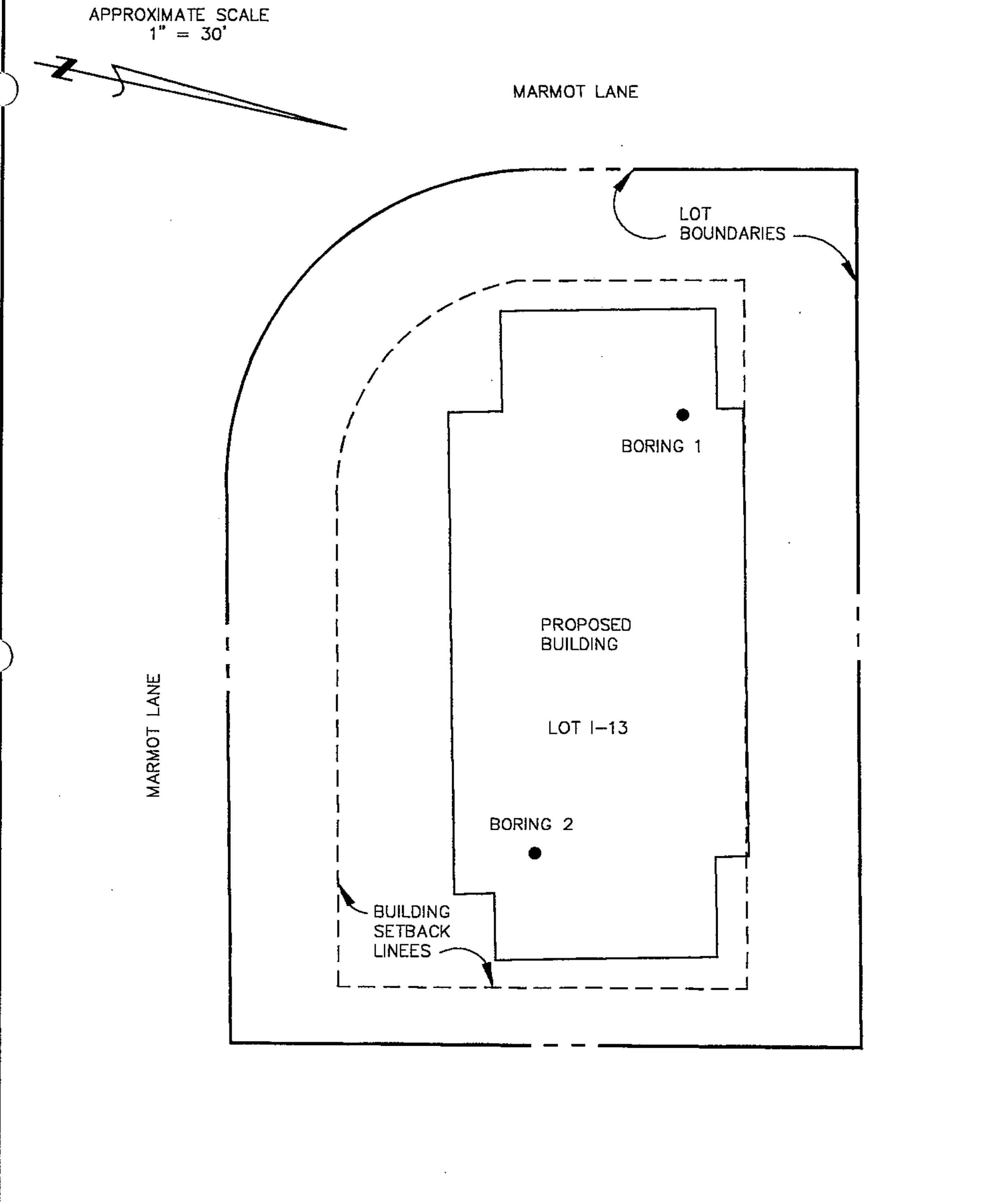
HEPWORTH - PAWLAK GEOTECHNICAL, INC.

Thomas J. Westhoff, C.E.T.

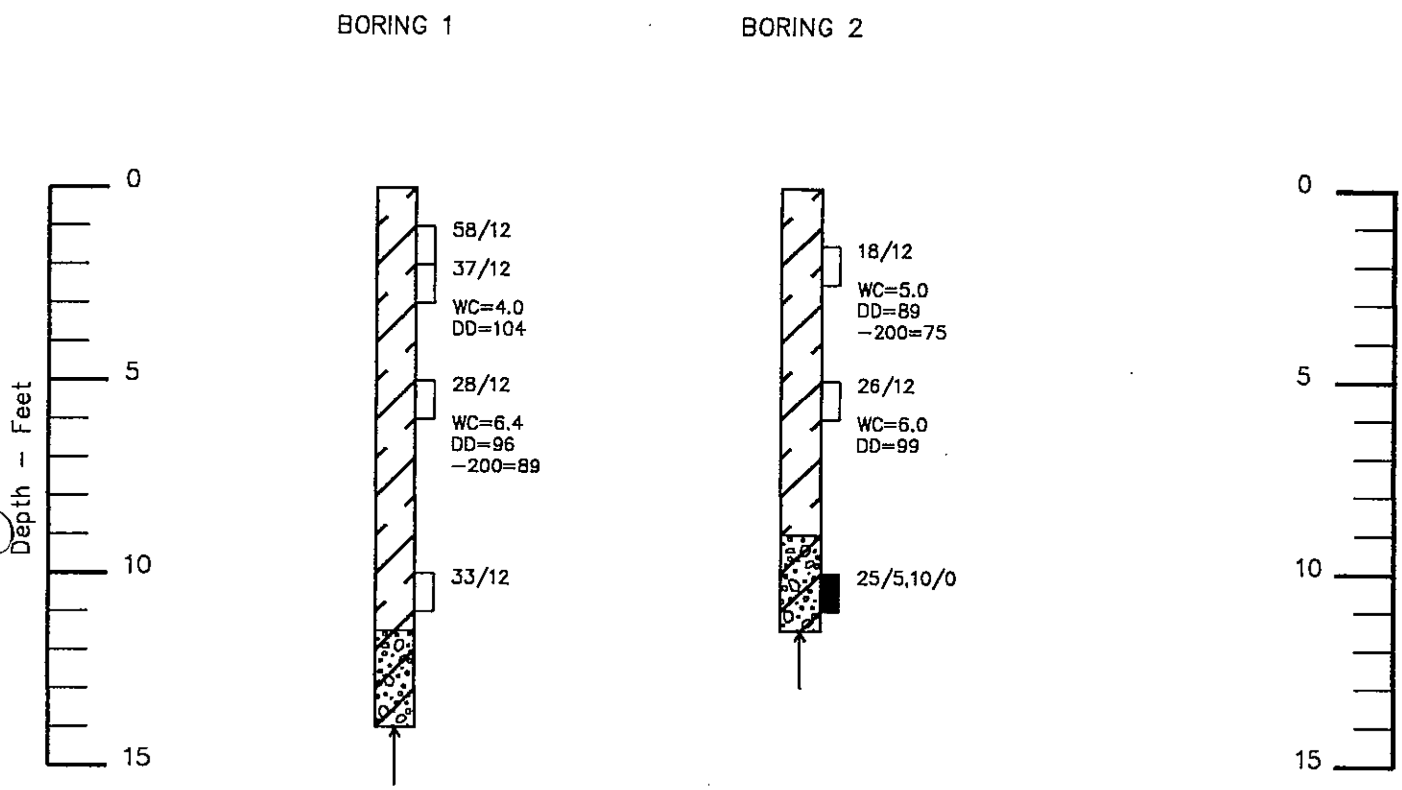
Reviewed By:

Steven L. Pawlak, P.E.

TJW/ksm



198 831	HEPWORTH – PAWLAK GEOTECHNICAL, INC.	LOCATION OF EXPLORATORY BORINGS	Fig. 1



Note: Explanation of symbols is shown on Fig. 3.

198 831	HEPWORTH – PAWLAK GEOTECHNICAL, INC.	LOGS OF EXPLORATORY BORINGS	Fig. 2

LEGEND:

CLAY AND SILT (CL–ML); slightly sandy to sandy, occasional gravel, very stiff to hard, slightly moist, slightly calcareous and porous.

GRAVEL AND COBBLES (GM–GP); sandy, slightly silty, possible boulders, dense, slightly moist, reddish brown, rounded rock.

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.

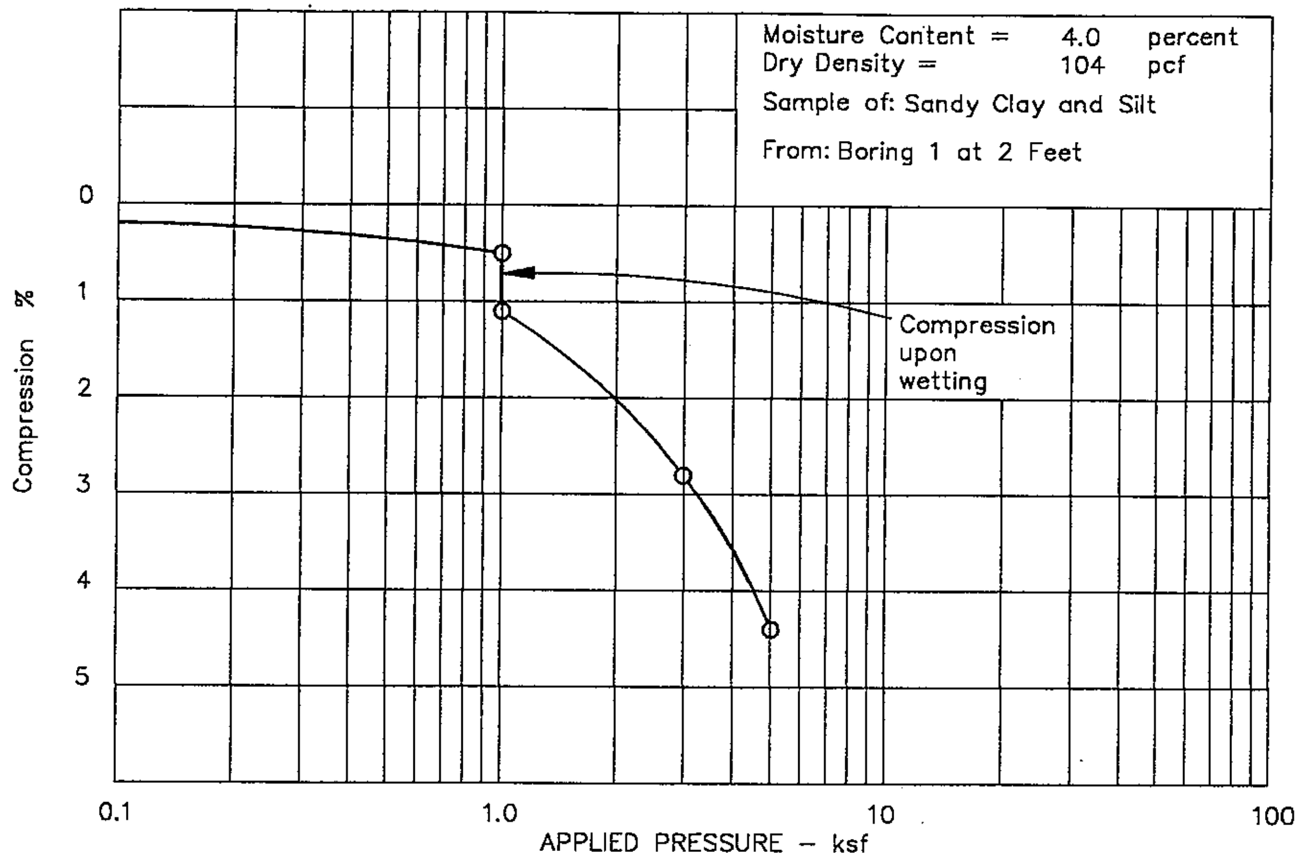
58/12 Drive sample blow count; indicates that 58 blows of a 140–pound hammer falling 30 inches were required to drive the California or SPT sampler 12 inches.

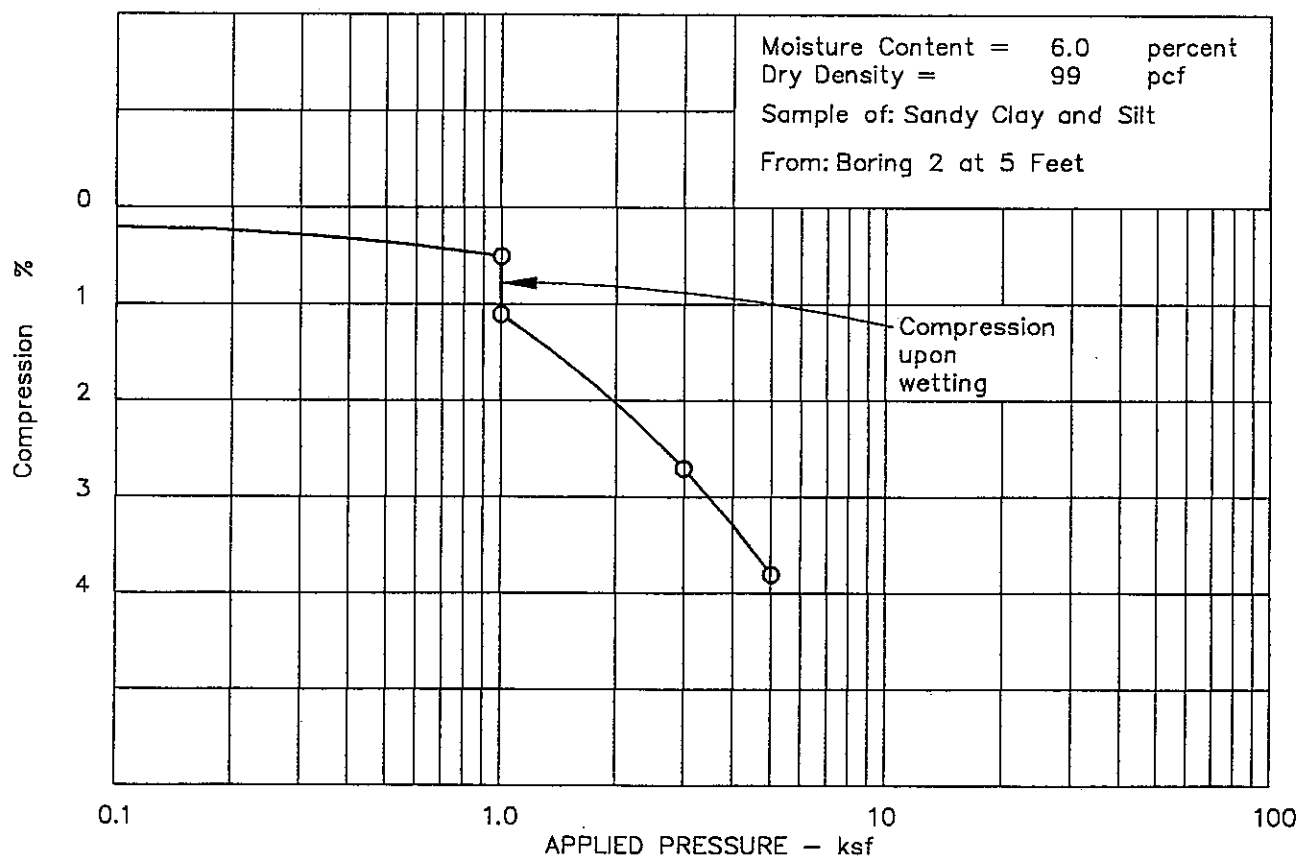
Practical drilling refusal.

NOTES:

1. Exploratory borings were drilled on December 18, 1998 with a 4–inch diameter continuous flight power auger.
2. Locations of exploratory borings were measured approximately by pacing from features in the field shown on the site plan provided.
3. Elevations of exploratory borings were not measured and the log of exploratory borings are drawn to depth.
4. The exploratory boring locations 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.

198 831	HEPWORTH – PAWLAK GEOTECHNICAL, INC.	LEGEND AND NOTES	Fig. 3





198 831	HEPWORTH – PAWLAK GEOTECHNICAL, INC.	SWELL–CONSOLIDATION TEST RESULTS	Fig. 4

HEPWORTH-PAWLAK GEOTECHNICAL, INC.

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

JOB NO.198 831

SAMPLE LOCATION		NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (pcf)	GRADATION		PERCENT PASSING NO. 200 SIEVE	ATTERBERG LIMITS		UNCONFINED COMPRESSIVE STRENGTH (PSF)		SOIL OR BEDROCK TYPE
BORING	DEPTH (feet)			GRAVEL (%)	SAND (%)		LIQUID LIMIT (%)	PLASTIC INDEX (%)			
1	2	4.0	104								Sandy Clay and Silt
	5	6.4	96			89					Sandy Clay and Silt
2	1½	5.0	89			75					Sandy Clay and Silt
	5	6.0	99								Sandy Clay and Silt