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# GEOTECHNICAL STUDY
# HDT REGIONAL WATER RECLAMATION FACILITY
# BIRDSALL ROAD EAST OF OLD PUEBLO ROAD
# EL PASO COUNTY, COLORADO

JOB NUMBER 308121B

MARCH 4, 2009

PREPARED FOR:

LOWER FOUNTAIN METROPOLITAN SEWAGE DISPOSAL DISTRICT
901 SOUTH SANTA FE AVENUE
FOUNTAIN, COLORADO 80817

## TABLE OF CONTENTS


Page

SCOPE .......... 1

PROPOSED CONSTRUCTION .......... 1

SITE CONDITIONS .......... 2

FIELD EXPLORATION .......... 3

SUBSURFACE CONDITIONS .......... 3

RECOMMENDATIONS

- FOUNDATIONS .......... 4
- DRILLED PIERS .......... 5
- SPREAD FOOTINGS .......... 7
- BUILDING FLOOR SLABS .......... 9
- FOUNDATION WALLS .......... 10
- PERIMETER DRAIN SYSTEM .......... 11
- SURFACE DRAINAGE .......... 12
- RESISTIVITY .......... 13
- WATER SOLUBLE SULFATES .......... 13
- PAVEMENTS .......... 14

LIMITATIONS .......... 15

BORING LOCATIONS .......... Figure 1
BORING LOGS .......... Figures 2-7
BORING LEGEND AND NOTES .......... Figure 8
SWELL-CONSOLIDATION TEST RESULTS .......... Figures 9-18
GRADATION TEST RESULTS .......... Figure 19
MOISTURE-DENSITY RELATIONSHIPS .......... Figures 20-21

SUMMARY OF LABORATORY TEST RESULTS .......... Table 1

## SCOPE

This report presents the results of a Geotechnical Study performed for the proposed HDT Regional Water Reclamation Facility in El Paso County, Colorado. The project site, which is shown on Figure 1, is located just south of Birdsall Road approximately ¾ mile east of Old Pueblo Road. The study was conducted in accordance with our proposal number 112A308 dated December 26, 2008. Chen & Associates previously conducted a Preliminary Geotechnical Investigation at the site under their job number 2-411-86, report dated November 5, 1986.

A field exploration program consisting of twenty-two borings was conducted to obtain information on the subsurface conditions. Samples obtained during the field exploration were tested in the laboratory to determine their 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 structure 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 subsoil conditions encountered.

## PROPOSED CONSTRUCTION

The proposed treatment facility will consist of several buildings with underground concrete tankage. The buildings will be of concrete or metal construction. Tankage will be of cast-in-place concrete construction. Foundations loads are not known but are assumed to vary from light to moderate. Cuts and fills up to about 12 feet will be required to achieve final grades. Individual structures are discussed below:

- Preliminary treatment building (Borings B1 and B2) having a finish floor elevation of 5421 with tankage extending to about 10 feet below finish floor.
- Operations building (B3 and B4) having a finish floor elevation of 5419.

- Biosolids thickening, dewatering and treatment structure (B5 through B8) having a finish floor elevation of 5414 with tankage extending about 12 to 14 feet below finish floor.
- Maintenance equipment and parts storage building (B9) having a finish floor elevation of 5417.
- Primary clarifiers (B7 through B11) having a finish floor elevation of 5415 with tankage extending to about 12 to 14 feet below finish floor.
- Aeration basins (B10 through B15) having a finish floor elevation of 5412 with tankage extending to about 20 to 24 feet below finish floor.
- Settling tanks (B14 through B17) having a finish floor elevation of 5409 with tankage extending to about 16 to 18 feet below finish floor.
- Disinfection and discharge building (B18 and B19) having a finish floor elevation of 5406 with tankage extending to about 10 feet below finish floor.

Once structure loadings and elevations have been finalized, we should be notified to reevaluate the recommendations contained in this report.

## SITE CONDITIONS

The site is currently vacant and has been used as a cattle pasture. Vegetation consists of native grass, weeds, cactus and a few deciduous trees. The ground surface is generally flat with a gentle slope down to the east and southeast. The maximum elevation difference across the site is approximately 60 feet. A large, steeply sided channel crosses the central portion of the site from the northwest down to the southeast. The channel leads to a shallow drainage with several small earthen embankments located south and east of the site. The channel slopes are near vertical in many areas and show signs of erosion. The channel varies from about 30 to 100 feet wide and is up to about 25 feet deep. Several small ditches having a depth of a few feet extend out from the large channel. No water was observed in either the large channel or the smaller ditches. During our field investigation, an access road was rough cut from the northwest corner of the site down to the southern portion of the site. Except for the rough cut road, channel and ditches, it does not appear that any significant grading has occurred on the site. Properties adjacent to the site are currently vacant and have also been used as cattle pastures.

## FIELD EXPLORATION

The field exploration was conducted by drilling twenty-two borings at the locations shown on Figure 1. Borings B1 through B19 were drilled in the area of the proposed structures. Borings B20 through B22 were drilled along the east property line to provide subsurface information for a proposed well. The borings were logged by a representative of H-P Geotech. Samples of the subsurface materials were obtained with 2-inch I.D. California liner 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 D1586. 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 Boring Logs (Figures 2 through 7). The samples were returned to our laboratory for review and testing.

## SUBSURFACE CONDITIONS

The subsoils encountered at the site consisted of clay and sand overburden soils overlying claystone bedrock.

The overburden soils encountered in the borings consisted predominately of sandy clay. The clay possesses a moderate plasticity and is dry to wet. Penetration tests indicate the clay is stiff to very stiff. The results of swell-consolidation tests indicate the clay possesses variable swell-consolidation characteristics. The clay generally possesses a low to moderate swell potential with swelling pressures up to about 10,000 psf. However, some of the near surface clay is relatively compressible upon loading and wetting.

Layers of clayey sand, silty sand and clean to slightly silty sand were encountered in the overburden clays. These sands possess a low to moderate plasticity and are fine to coarse grained. Penetration tests indicate the sands are medium to very dense.

The overburden soils are underlain by claystone bedrock which was encountered at depths of 22 to 45 feet below the existing ground surface. The claystone possesses a moderate to high plasticity and is fine grained. Penetration tests indicate the claystone is hard to very hard. The results of a swell-consolidation test indicate the claystone possesses a high swell potential.

When checked immediately after drilling, water was measured in 13 of the borings at depths of 18 to 35 feet. When checked up to 41 days after drilling, water was measured in 21 of the borings at depths of 14 to 49 feet.

## RECOMMENDATIONS

### FOUNDATIONS

Based on the subsurface conditions encountered, we recommend the proposed buildings and concrete tankage be founded on straight shaft piers drilled into bedrock. The upper clays vary from possessing a swell potential to being relatively compressible. Shallow foundations placed directly on these soils could experience movement resulting in structural distress to the structure. The drilled pier foundation is intended to transfer structural loads to a zone of relatively stable moisture content and make it possible to load the piers sufficiently to resist uplift movements.

Shallow foundations placed on a layer of nonexpansive structural fill would be a suitable alternative to drilled piers. Building footings and concrete tankage should bear on at least 4 feet of nonexpansive fill. This would require overexcavation of the onsite expansive soils and replacement with an imported, nonexpansive material.

## DRILLED PIERS

The design and construction criteria presented below should be observed for a drilled pier foundation system.

1) Piers bearing in the unweathered bedrock should be designed for an allowable end bearing pressure of 30,000 pounds per square foot (psf) and a skin friction of 2,500 psf for the portion of the pier in unweathered bedrock. Uplift due to structural loadings on the piers can be resisted by using 75 percent of the allowable skin friction value plus an allowance for pier weight.
2) Piers should also be designed for a minimum dead load pressure of 15,000 psf based on the pier bottom end area. If the minimum dead load requirement cannot be achieved and the piers are spaced as far apart as practical, the pier length should be extended beyond the minimum bedrock penetration and minimum length recommended below to make up the dead load deficit. This can be accomplished by assuming 75 percent of the skin friction value given above acts in the direction to resist uplift caused by swelling materials near the top of the pier.
3) Piers should be designed to resist lateral loads using a modulus of horizontal subgrade reaction of 50 tcf in the overburden soils or new compacted fill, and 250 tcf in the bedrock. The modulus values are for a long 1-foot wide pier and must be corrected for pier size.
4) Piers should penetrate at least three pier diameters or 8 feet, whichever is greater, into unweathered bedrock. Piers should also have a minimum length of 20 feet. Piers that do not meet the minimum dead load requirement will need to be extended

beyond the 8-foot minimum bedrock penetration and 20-foot minimum length as discussed in 2) above.

5) Piers should have a minimum diameter of 18 inches to facilitate proper construction and observation of the pier hole. Piers should also have a maximum length to diameter ratio of 25.

6) Piers should be reinforced their full length. The cross sectional area of the reinforcement should be at least ¾ percent of the cross sectional area of the pier. Reinforcing steel should be provided with spacers to insure proper horizontal positioning and adequate concrete cover.

7) We estimate total settlement for foundations designed and constructed as discussed in this section will be less than one inch.

8) A 6-inch void should be provided beneath the grade beams to concentrate pier loadings and to prevent the expansive materials from exerting uplift forces on the grade beams.

9) The minimum spacing requirement between piers should be 3 diameters from center to center. Piers grouped less than 3 diameters from center to center should be studied on an individual basis to determine the appropriate reduction in capacity.

10) Concrete used in the piers should contain Type II or V cement and be a fluid mix with a minimum slump of 5 inches. The concrete in the upper five feet of the piers should be consolidated with a mechanical vibrator.

11) The lower 8 feet of the pier holes should be provided with shear rings to assist the development of peripheral shear stress between the pier and the bedrock. The shear rings should be 2 inches deep and 3 inches high, and be constructed on 18-inch centers.

12) Pier holes should be properly cleaned prior to placement of concrete.

13) The presence of water in the borings indicates casing and/or dewatering of the piers will be required. The requirements for casing and dewatering can sometimes be reduced by placing concrete immediately upon cleaning and observing the pier hole. In no case should concrete be placed in more than 3 inches of water unless the

tremie method is used.

14) Care should be taken that the piers are not oversized at the top. Mushroomed pier tops can reduce the effective dead load pressure on the piers. The upper 18 inches of the piers should be formed with a sonotube to insure a uniform diameter at the top.
15) Concrete should be placed in piers the same day they are drilled. The presence of water or caving soils will require that concrete be placed immediately after the pier hole is completed. Failure to place concrete the day of drilling will normally result in a requirement for additional bedrock penetration.
16) A representative of the geotechnical engineer should observe installation of the piers on a full-time basis.

SPREAD FOOTINGS

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

1) Building footings and concrete tankage foundations should be designed for a maximum allowable soil bearing pressure of 2,500 psf.
2) Foundations should bear on at least 4 feet of nonexpansive structural fill.
3) Any existing fill encountered below the foundations should be removed and replaced.
4) Fill placed for support of foundations should be compacted to at least 100 percent of the maximum standard proctor density (ASTM D698) at a moisture content within 2 percent of optimum. Fill placed within 4 feet of the bottom of the foundations should consist of a granular, nonexpansive material. The nonexpansive fill should extend down from the bottom of the foundations at a 1 horizontal to 1 vertical projection. Nonexpansive fill should consist of minus 2-inch material having between 12 and 35 percent passing the No. 200 sieve, a liquid limit less than 30 and a plasticity index less than 15. The onsite clay is not suitable for use as

nonexpansive fill placed within 4 feet of the bottom of the foundations. The onsite clay can be used as fill at depths greater than 4 feet below the bottom of the foundations.

5) Resistance to sliding at the bottom of the foundations should be calculated based on a coefficient of friction of 0.35. Passive pressure against the sides of the foundations should be calculated using an equivalent fluid unit weight of 250 pcf above the groundwater level. The coefficient of friction and passive pressure values recommended above assume mobilization of the ultimate soil strength. Suitable factors of safety should be included in the design to limit the strain that will occur at the ultimate strength, particularly in the case of passive resistance. Fill placed against the sides of the foundations to resist lateral loads should be compacted to at least 95 percent of the maximum standard Proctor density at a moisture content within 2 percent of optimum.
6) Footings should have a minimum width of 16 inches.
7) We estimate total settlement for foundations designed and constructed as discussed in this section will be approximately 1 inch.
8) Exterior footings and footings beneath unheated areas should be provided with adequate soil cover above their bearing elevation for frost protection.
9) Continuous foundation walls should be reinforced top and bottom to span an unsupported length of at least 15 feet.
10) Areas of loose or disturbed materials, or soils containing roots or other organic material encountered within the foundation excavations should be removed and replaced with new structural fill.
11) Foundation soils should be compacted with a mechanical compactor prior to the placement of structural fill or concrete.
12) A representative of the geotechnical engineer should observe all foundation excavations prior to placement of structural fill and concrete.

## BUILDING FLOOR SLABS

Slab-on-grade construction presents a problem where expansive materials are present near floor slab elevation because sufficient dead load cannot be imposed on them to resist the uplift pressure generated when the materials are wetted and expand. The only way to prevent damage as a result of slab movement is to construct structural floors above a crawl space or void material.

Slab-on-grade construction may be considered as an alternate for the building floor slabs provided the risk of distress resulting from floor slab movement is recognized and accepted by the owner. If slab-on-grade construction is used, the following design and construction details should be observed to reduce the effects of movement.

1) Floor slabs should be separated from all bearing walls and columns with expansion joints which allow unrestrained vertical movement.
2) Interior nonbearing partitions resting on the floor slabs should be provided with slip joints at the bottoms so that, if the slabs move, the movement cannot be transmitted to the upper structure. This detail is also important for wallboards, stairways and door frames. Slip joints which allow at least 2 inches of vertical movement are recommended. If slab bearing masonry partitions are used, the slip joints will have to be placed at the tops of the walls. If the floors move, the masonry partitions may show distress where they meet the bearing walls unless the partition walls can move independent of the bearing walls.
3) Floor slabs should be provided with control joints to reduce damage due to shrinkage cracking. The requirements for slab reinforcement should be established by the designer based on experience and the intended slab use.
4) Any existing fill encountered below the slabs should be removed and replaced.

5) Fill placed below floor slabs should be compacted to at least 95 percent of the maximum standard proctor density at a moisture content within 2 percent of optimum.
6) The risk of slab movement due to expansive soils can be reduced by placing the slabs on a 3-foot layer of nonexpansive fill. This would require overexcavation of the onsite soils and replacement with an imported nonexpansive material. Specifications for nonexpansive fill were given under SPREAD FOOTINGS.
7) Where plumbing lines enter through the floor, a positive bond break should be provided. Flexible connections should be provided for slab-bearing mechanical equipment.

The precautions and recommendations itemized above will not prevent the movement of floor slabs if the underlying expansive soils are subjected to a change in moisture content. However, they will reduce the damage if movement occurs.

FOUNDATION WALLS

Granular, nonexpansive soils are recommended for backfilling foundation walls because their use results in lower lateral earth pressures. The granular, nonexpansive backfill should be sloped up from the base of the wall at an angle of at least 35 degrees from vertical. The upper 2 feet of wall backfill should be a relatively impervious soil or a pavement structure should be provided to prevent surface water infiltration into the backfill. The onsite clays are not suitable for use as granular, nonexpansive backfill but are suitable for use as the upper impervious layer. Specifications for nonexpansive fill are given under SPREAD FOOTINGS.

Foundation walls which are laterally supported and can be expected to undergo a moderate amount of deflection should be designed for a lateral earth pressure computed on the basis on an equivalent fluid unit weight of 45 pcf for granular backfill. Rigid structures that cannot deflect should be designed for the at rest lateral pressure computed on the basis of an equivalent fluid unit weight of 65 pcf for granular backfill.

As an alternative, the walls could be backfilled with the onsite soils if designed for higher lateral earth pressures. Walls which are laterally supported and can be expected to undergo a moderate amount of deflection should be designed for a lateral earth pressure computed on the basis on an equivalent fluid unit weight of 65 pcf for onsite backfill. Rigid structures that cannot deflect should be designed for the at rest lateral pressure computed on the basis of an equivalent fluid unit weight of 85 pcf for onsite backfill.

The lateral pressures recommended above assume drained conditions behind the walls and a horizontal backfill surface. The buildup of water behind the walls or an upward sloping backfill surface will increase the lateral pressure imposed on a foundation wall. Foundation walls should also be designed for the appropriate surcharge pressures such as adjacent buildings, traffic and construction materials.

Foundation wall backfill should be compacted to at least 95 percent of the maximum standard Proctor density at a moisture content within 2 percent of optimum. Care should be taken not to over compact the backfill since this could cause excessive lateral pressures on the walls. Some settlement of foundation wall backfill could occur even if the backfill is placed correctly.

## PERIMETER DRAIN SYSTEM

All structures that extend below the ground surface should either be protected by a perimeter drain system or be waterproofed and designed to resist hydrostatic uplift. The perimeter drain should consist of solid, perforated drain pipe placed in the bottom of a trench and surrounded above the invert level with free draining gravel. The gravel filled trench should extend up at least 12 inches above the top of the pipe and be completely surrounded with filter fabric. The drain pipe should have a minimum diameter of 4 inches and be placed at least 12 inches below the slab or ground surface in crawlspaces. The drain pipe should be graded at a minimum slope of 1 percent to a gravity outlet or to a sump where the water can be removed by pumping. The gravel used in the drain system should consist of minus 2-inch aggregate having less than 10 percent passing the No. 4 sieve and less than 5 percent passing the No. 200 sieve.

## SURFACE DRAINAGE

The success of the foundation and slab-on-grade floor systems is contingent upon keeping the bearing soils at more or less constant moisture content, and by not allowing surface water a path to the subsurface. Positive surface drainage away from the structures must be maintained at all times. Landscaped areas should be designed and built such that irrigation and other surface water will be collected and carried away from foundation elements.

The ground surface surrounding the exterior of the structures and any overlying pavements should have a positive slope away from foundation walls on all sides. We recommend a minimum slope of 9 inches in the first 10 feet in unpaved areas and a minimum slope of 3 inches in the first 10 feet in paved areas. Pavements and exterior slabs that abut structures should be sealed against moisture intrusion at the joint.

Exterior backfill should be placed near optimum moisture content and compacted to at least 95 percent of the maximum standard Proctor density at a moisture content. All roof downspouts and drains should discharge well beyond the limits of all backfill. Irrigation within ten feet of the structures should be carefully controlled and minimized. Plastic membranes should not be used to cover the ground surface adjacent to foundation walls.

ELECTRICAL RESISTIVITY

Laboratory electrical resistivity tests were performed on soil samples at varying moisture contents. The test results are presented on the following table. Resistivities below 2,000 ohm-cm indicate poor corrosion resistance (American Iron and Steel Institute's Handbook of Steel Drainage and Highway Construction Products).

| Boring | Depth | Natural Moisture (%) | Moisture As Tested (%) | Resistivity (ohm-cm) | Soil Type |
|---|---|---|---|---|---|
| B9 | 10 | 7.3 | 11.6 | 7,600 | Sandy clay |
| | | | 15.6 | 2,120 | |
| | | | 19.7 | 1,360 | |
| | | | 23.6 | 1,240 | |
| | | | 27.5 | 1,320 | |
| B13 | 19 | 15.9 | 19.1 | 4,000 | Sandy clay |
| | | | 22.3 | 920 | |
| | | | 25.7 | 560 | |
| | | | 29.0 | 480 | |
| | | | 32.1 | 440 | |
| | | | 35.3 | 400 | |
| | | | 38.6 | 480 | |
| B19 | 1 | 6.3 | 9.5 | 10,800 | Sandy clay |
| | | | 13.7 | 3,000 | |
| | | | 17.9 | 1,640 | |
| | | | 22.1 | 1,600 | |
| | | | 26.2 | 2,040 | |

WATER SOLUBLE SULFATES

Samples of the overburden soils were tested to determine the amount of water soluble sulfates. As shown below, the sulfate content varied from 0.02 to 0.20 percent. These concentrations of sulfates represent a negligible to positive degree of sulfate attack on concrete exposed to these materials. For this condition, we recommend the use of sulfate resistant cement (Type II or Type V) in all concrete exposed to the onsite soils or bedrock.

| Boring | Depth | Sulfates (%) | Soil Type |
|---|---|---|---|
| B7 | 5 | 0.07 | Sandy clay |
| B11 | 24 | 0.18 | Sandy clay |
| B14 | 15 | 0.20 | Sandy clay |
| B16 | 1 | 0.02 | Sandy clay |

## PAVEMENTS

The upper onsite soils generally classify as A-6 and A-7-6 in accordance with the AASHTO soil classification system. These soils have a general subgrade rating of poor for support pavements. For design purposes, a soil support value of 3 was assumed for flexible pavements and a modulus of subgrade reaction of 100 pci was assumed for rigid pavements. An equivalent 18-kip daily load application (EDLA) of 5 was assumed for areas subjected to automobile traffic only and an EDLA of 10 was assumed for areas subjected to occasional truck traffic.

In areas subjected to automobile traffic only, we recommend either 5½ inches of full depth asphalt or 3½ inches of asphalt overlying 7 inches of base course. In areas subjected to occasional truck traffic, we recommend either 6½ inches of full depth asphalt or 4½ inches of asphalt overlying 7 inches of base course. Truck loading dock areas and other areas where truck turning movements are concentrated should be paved with 6 inches of portland cement concrete.

Asphalt should be a plant mix material (minus ½ inch aggregate) meeting El Paso County or Colorado Department of Transportation (CDOT) specifications. Portland cement concrete should meet CDOT Class P specifications. Base course should be Class 6 in accordance with CDOT Specifications.

All fill placed below pavements should be compacted to at least 95 percent of the maximum standard Proctor density at a moisture content within 2 percent of optimum. Prior to placement of the pavement section, the final subgrade should be scarified to a depth of 12 inches, adjusted to within 2 percent of the optimum moisture content and recompacted to at least 95 percent of the maximum standard Proctor density. The subgrade should then be proofrolled with a heavy, pneumatic tired vehicle. Areas which deform under wheel loads should be removed and replaced. Base course should be compacted to at least 100 percent of the maximum standard Proctor density.

Surface drainage is important for the satisfactory performance of pavement. Wetting of the subgrade soils or base course will cause a loss of strength which can result in pavement distress. Surface drainage should be carefully designed to remove all water from paved areas.

## LIMITATIONS

This report has been prepared 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, the proposed type of construction and our experience in the area. 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 revaluation 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 onsite observation of excavations and foundation bearing strata and testing of structural fill by a representative of the geotechnical engineer.

Sincerely,

HEPWORTH-PAWLAK GEOTECHNICAL, INC.

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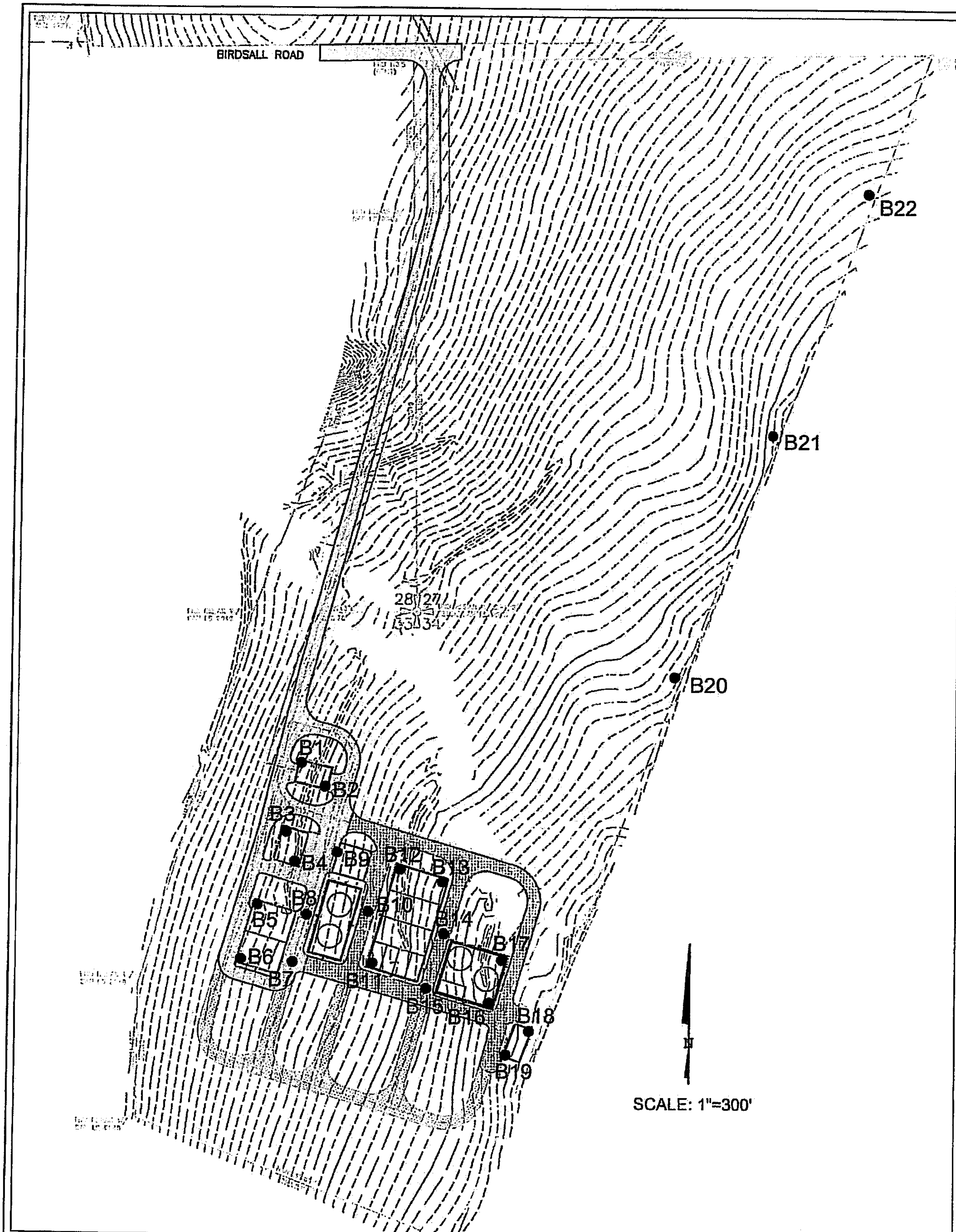
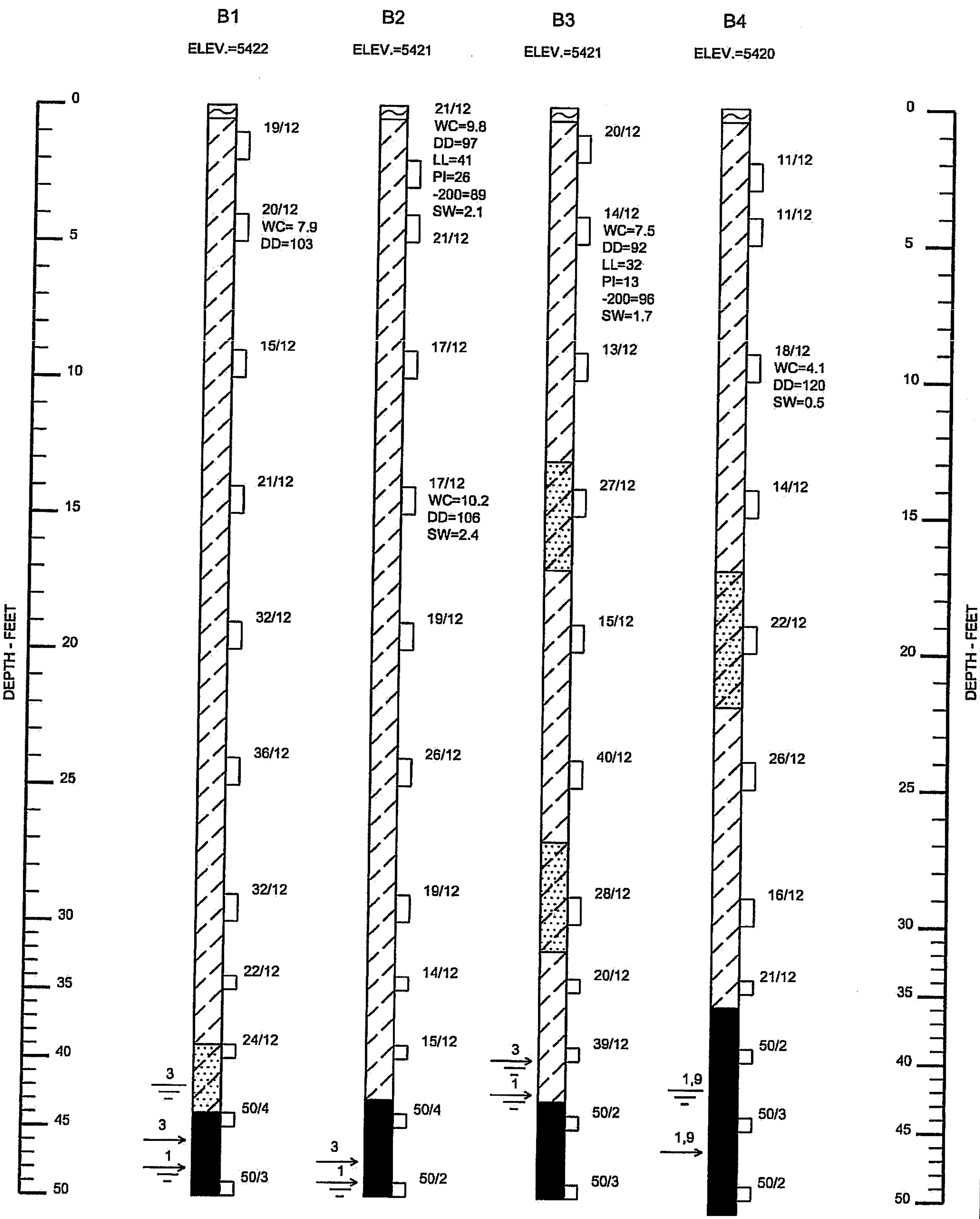



| | | | |
|---|---|---|---|
| 308121B | HEPWORTH-PAWLAK GEOTECHNICAL, Inc. | HDT REGIONAL WATER RECLAMATION FACILITY BORING LOCATIONS | FIG. 1 |

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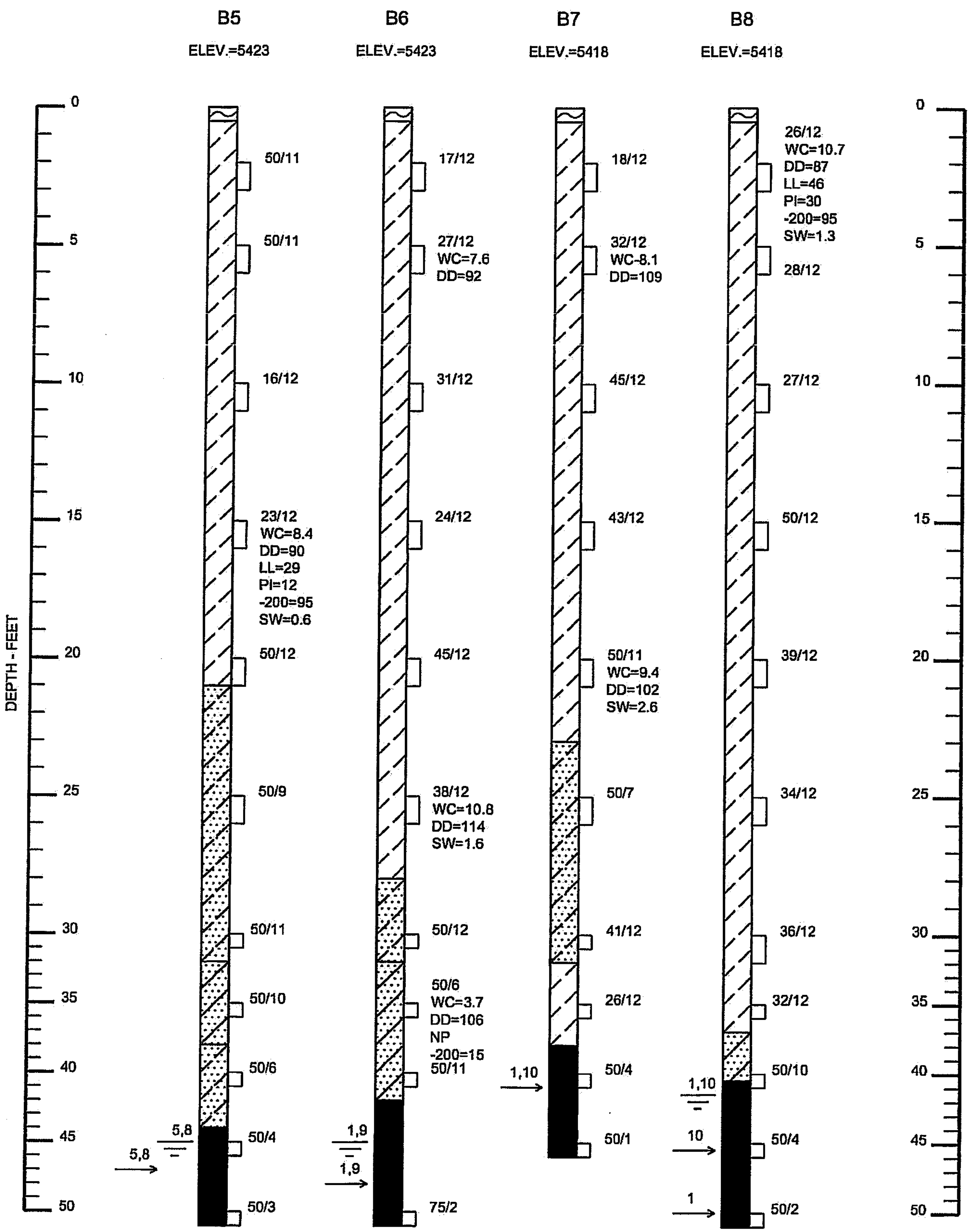
| 308121B | HEPWORTH-PAWLAK GEOTECHNICAL, Inc. | HDT REGIONAL WATER RECLAMATION FACILITY BORING LOGS | FIG. 2 |

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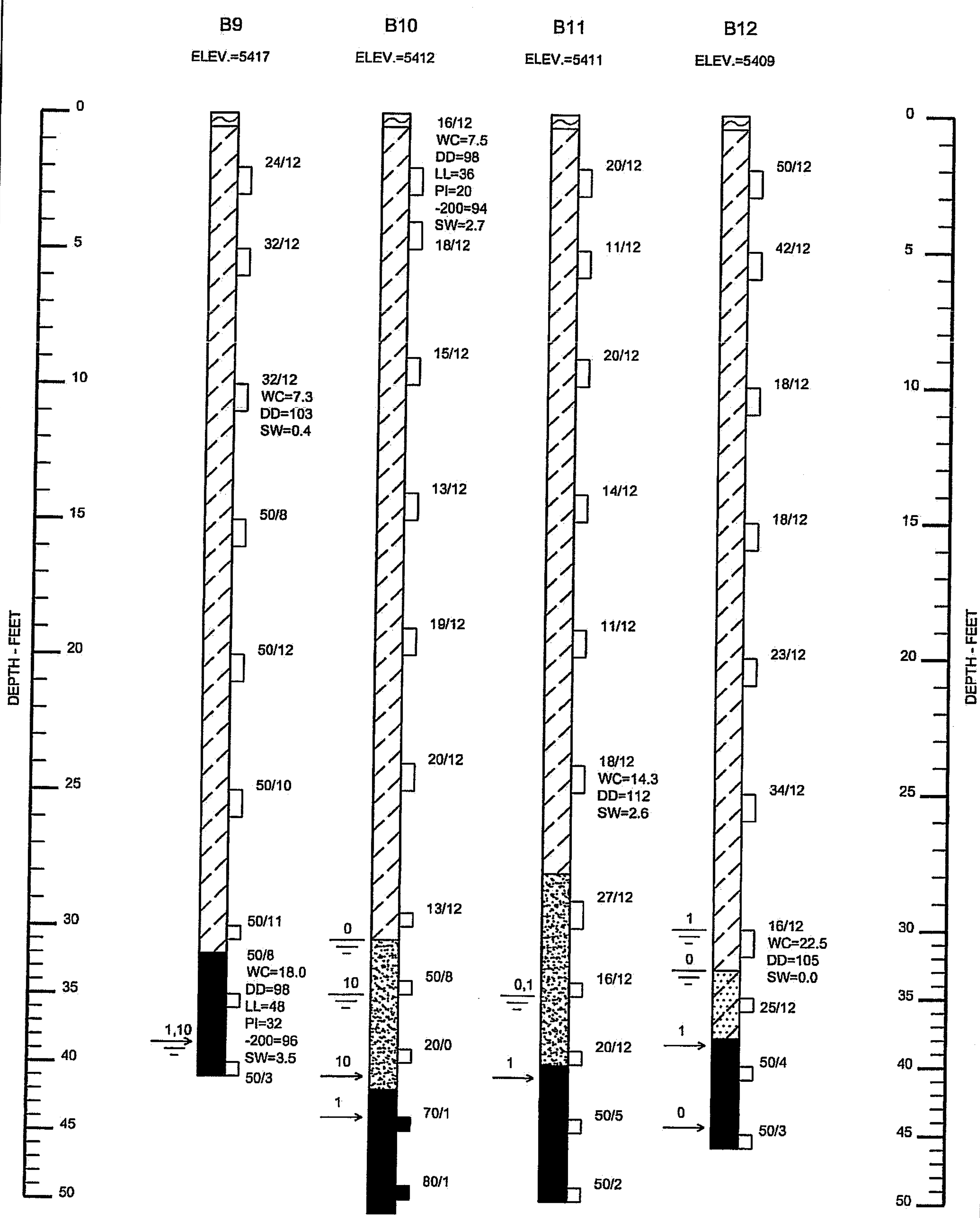
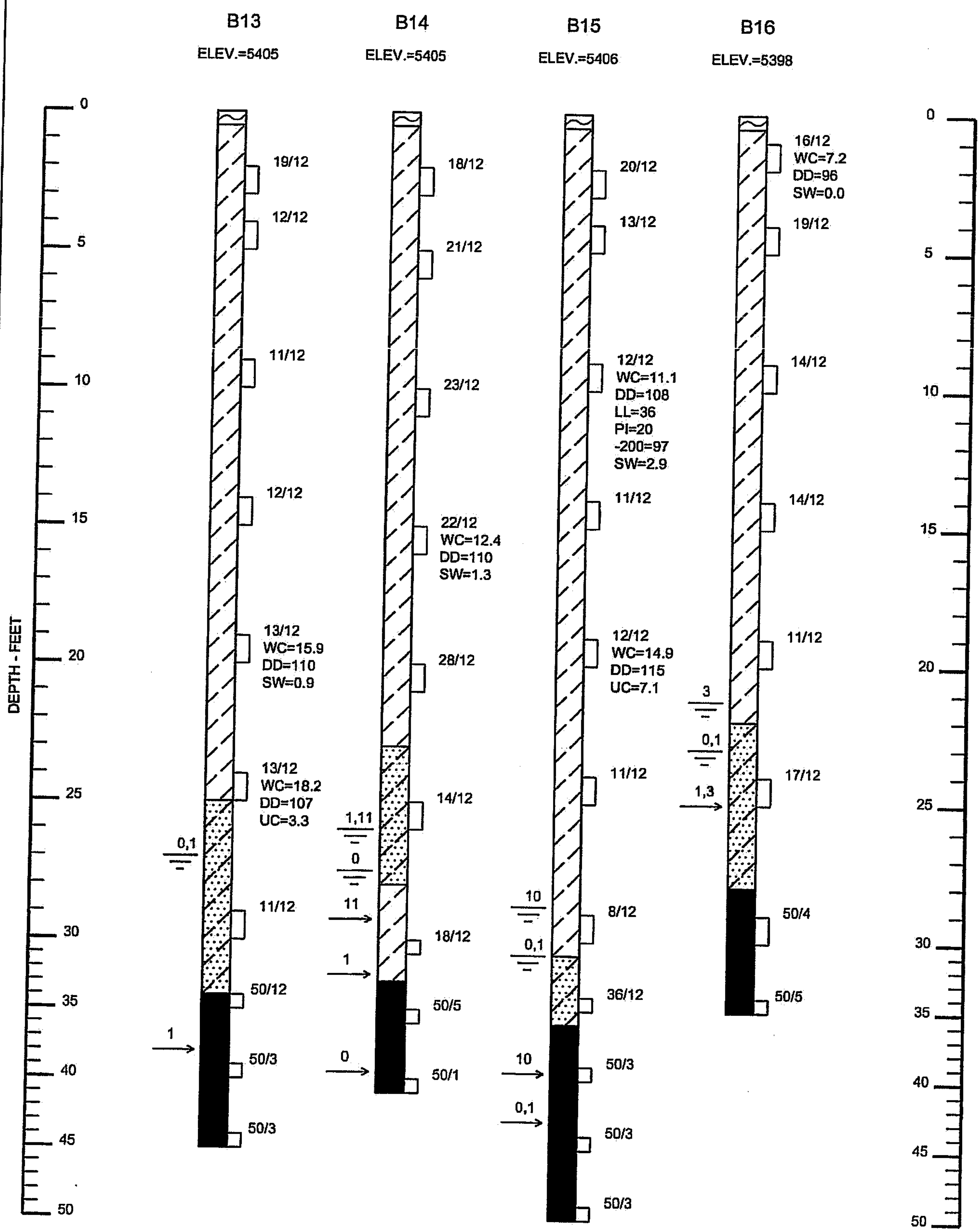
| 308121B | HEPWORTH-PAWLAK GEOTECHNICAL, Inc. | HDT REGIONAL WATER RECLAMATION FACILITY BORING LOGS | FIG. 3 |




| 308121B | HEPWORTH-PAWLAK GEOTECHNICAL, Inc. | HDT REGIONAL WATER RECLAMATION FACILITY BORING LOGS | FIG. 4 |
|---|---|---|---|

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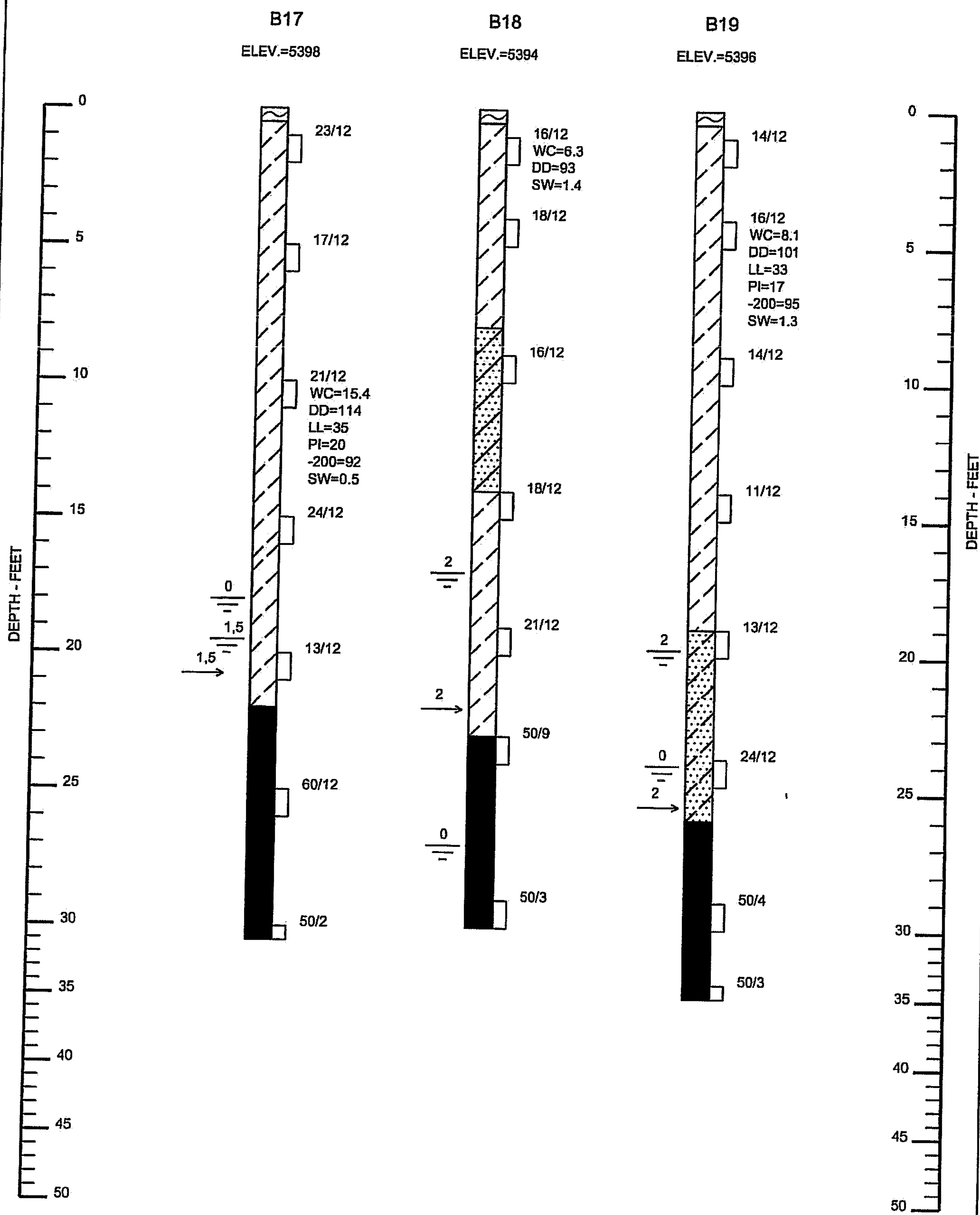
|---|---|---|---|
| 308121B | HEPWORTH-PAWLAK GEOTECHNICAL, Inc. | HDT REGIONAL WATER RECLAMATION FACILITY BORING LOGS | FIG. 5 |

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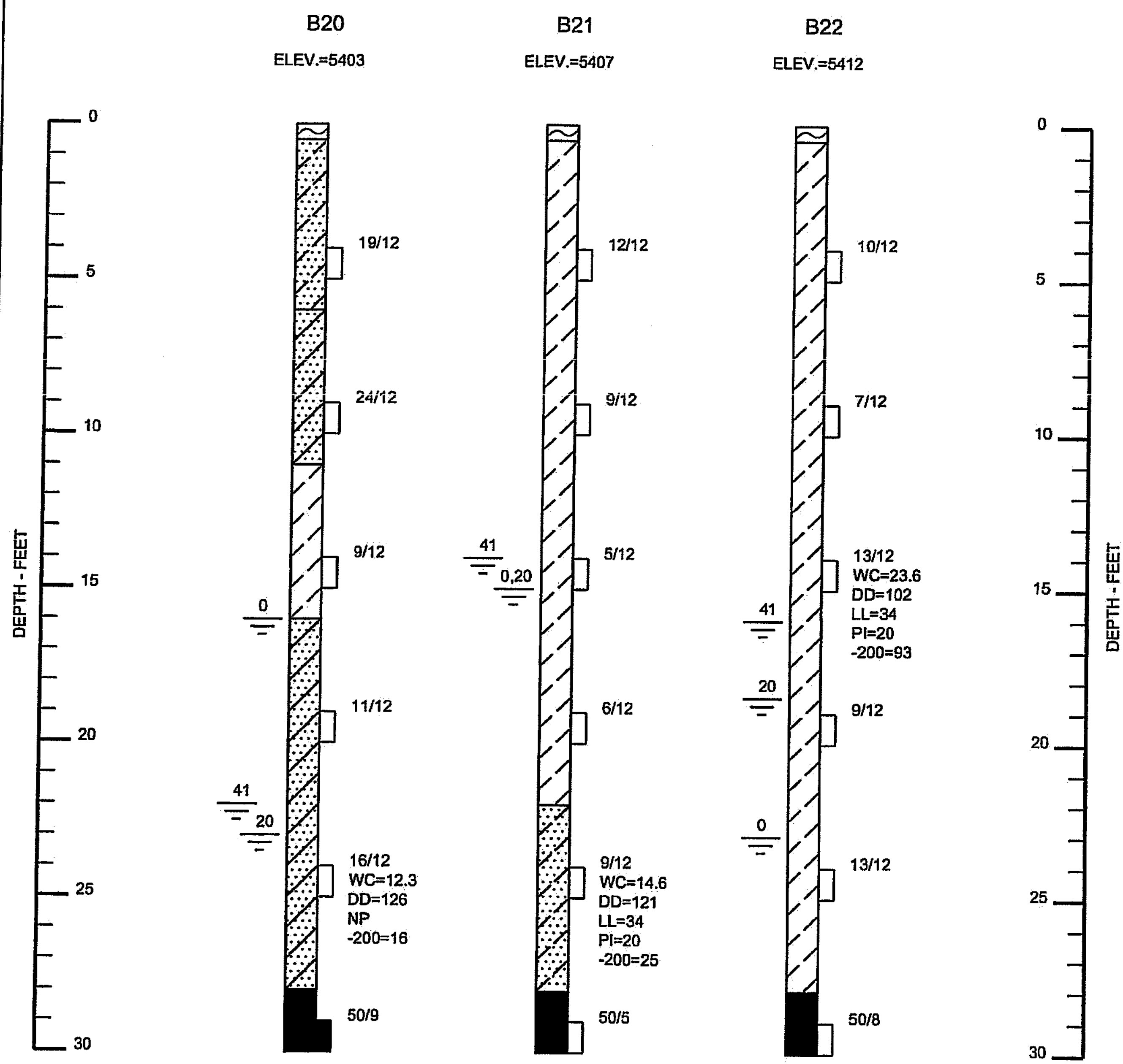
| 308121B | HEPWORTH-PAWLAK GEOTECHNICAL, Inc. | HDT REGIONAL WATER RECLAMATION FACILITY BORING LOGS | FIG. 6 |




| | | | |
|---|---|---|---|
| 308121B | HEPWORTH-PAWLAK GEOTECHNICAL, Inc. | HDT REGIONAL WATER RECLAMATION FACILITY BORING LOGS | FIG. 7 |

LEGEND

Topsoil.

Clay (CL), slightly sandy to sandy, moderate plasticity, locally porous and calcareous, stiff to very stiff, dry to wet, brown.

Sand (SC), clayey, moderate plasticiy, medium to very dense, slightly moist to wet, brown.

Sand (SM), silty, low plasticity, medium to very dense, slightly moist to wet, brown.

Sand (SP-SM), clean to slightly silty, gravelly, medium to coarse grained, medium to very dense, moist to wet, brown.

Claystone bedrock, moderate to high plasticity, hard to very hard, moist, gray and brown.

17/12 Indicates 2-inch I.D. California sampler. 17/12 indicates 17 blows of a 140-pound hammer falling 30 inches were required to drive the sampler 12 inches.

28/12 Indicates 1 3/8- inch I.D. split spoon sampler.

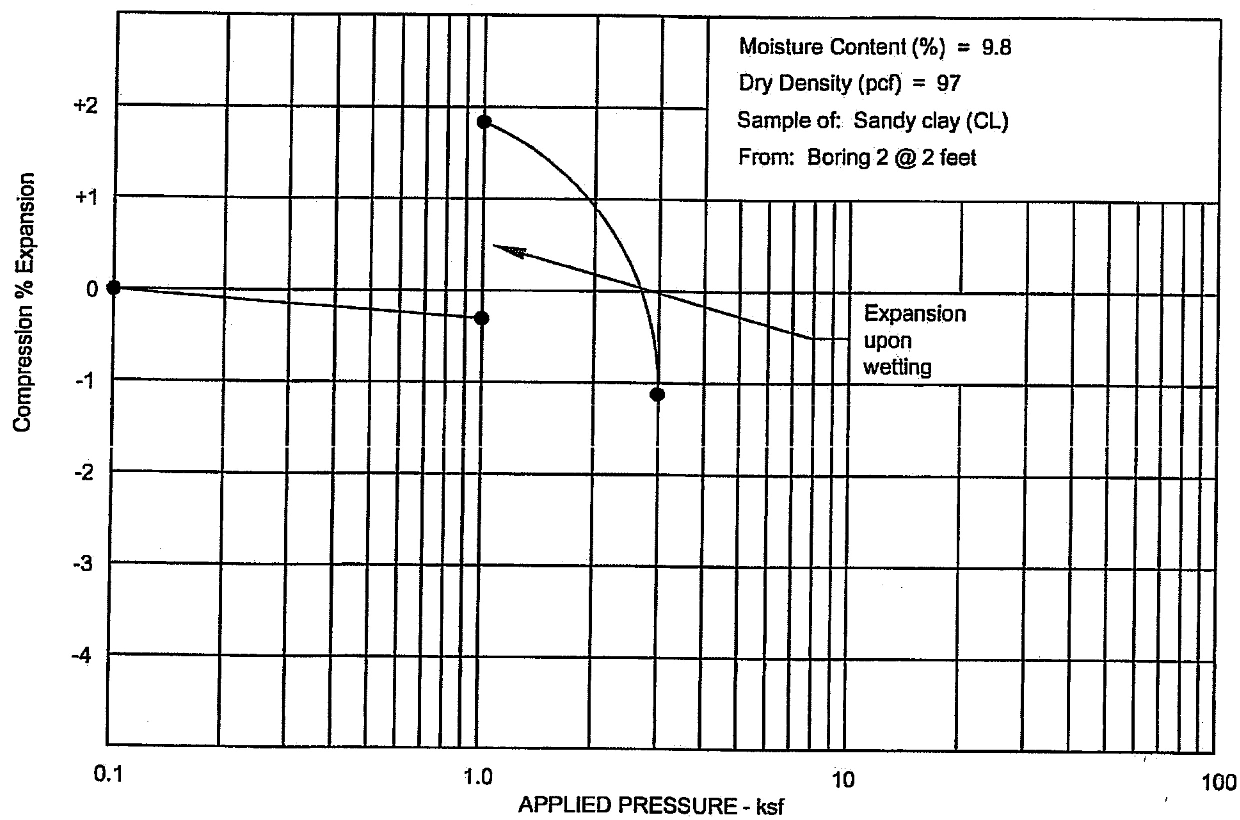
1 Indicates depth of free water and number of days after drilling measurement was made.

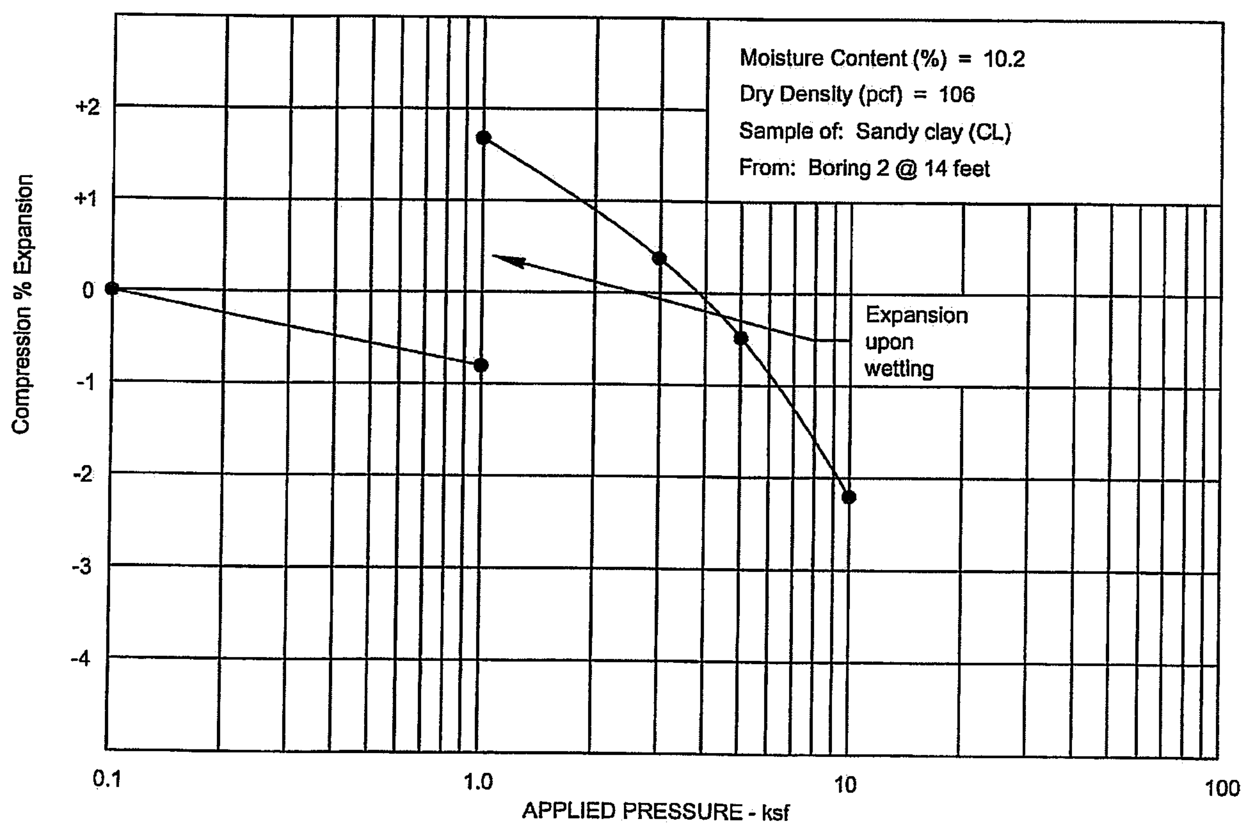
1 Indicates depth to which hole caved and number of days after drilling measurement was made.

NOTES:

1. Borings were drilled between January 15, 2009 and January 28, 2009 with truck mounted Longyear BK51 and CME 55 drill rigs powering 4-inch diameter, solid stem augers.
2. Locations of borings were measured by GMS, Inc.
3. Elevations of borings were determined from the furnished topographic map.
4. The 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 boring logs represent the approximate boundaries between material types and the transitions may be gradual.
6. Water level readings shown on the logs were made at the time indicated. Fluctuations in the water level may occur with time.
7. Laboratory Test Results:
   - WC=Moisture content (%).
   - DD=Dry density (pcf).
   - LL=Liquid limit.
   - PI=Plasticity index.
   - NP=Nonplastic.
   - UC=Unconfined compressive stregnth (ksf).
   - -200=Percent passing no. 200 sieve.
   - SW=Swell under 1 ksf surcharge (%).

| | | | |
|---|---|---|---|
| 308121B | HEPWORTH-PAWLAK GEOTECHNICAL, Inc. | HDT REGIONAL WATER RECLAMATION FACILITY BORING LEGEND AND NOTES | FIG. 8 |

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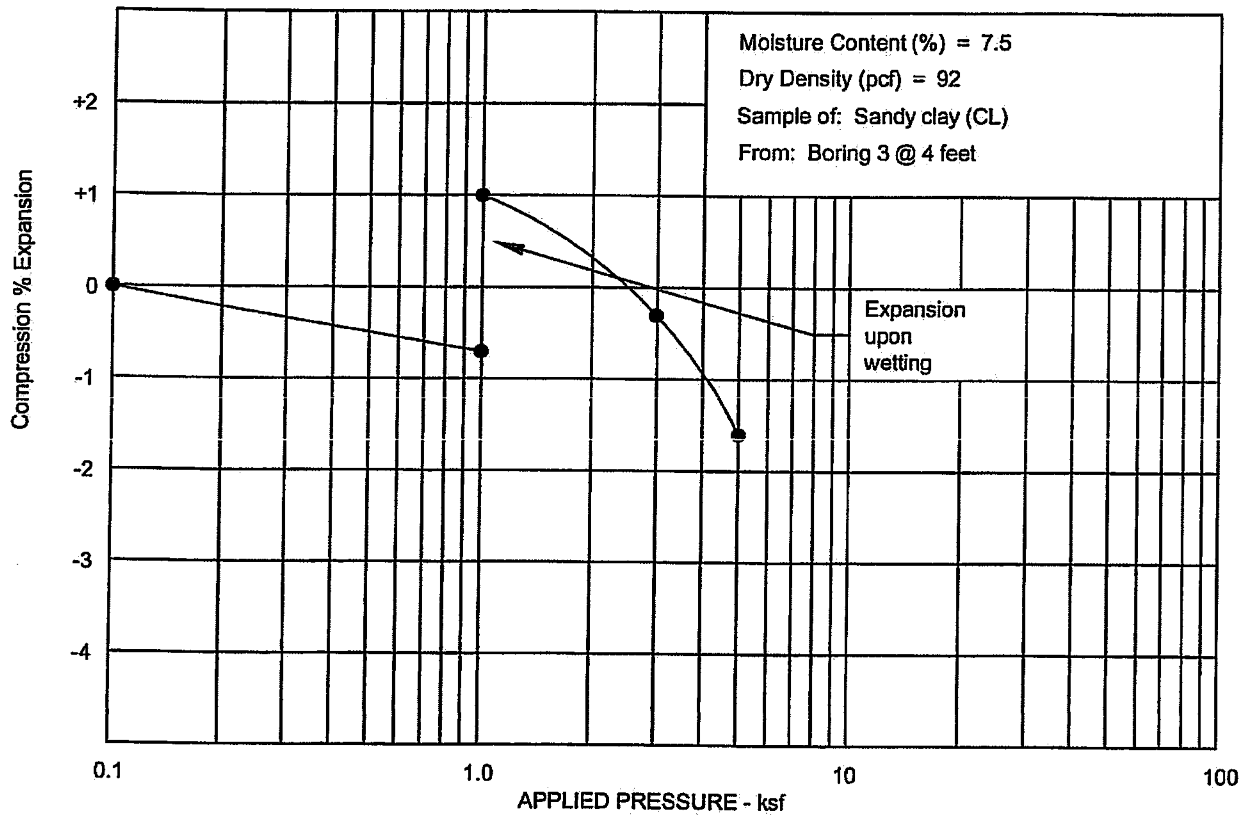
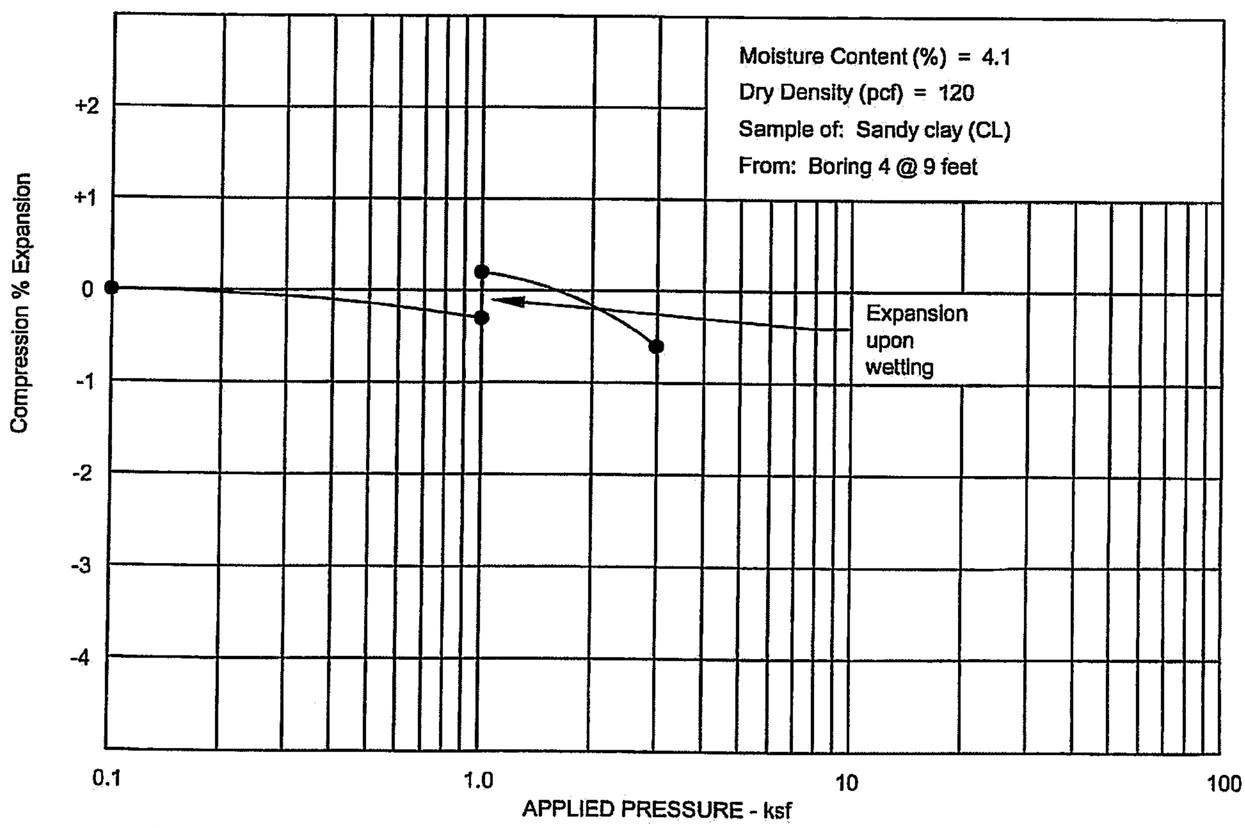
|---|---|---|---|
| 308121B | HEPWORTH - PAWLAK GEOTECHNICAL, INC. | HDT REGIONAL WATER RECLAMATION FACILITY<br>SWELL-CONSOLIDATION TEST RESULTS | FIG. 9 |

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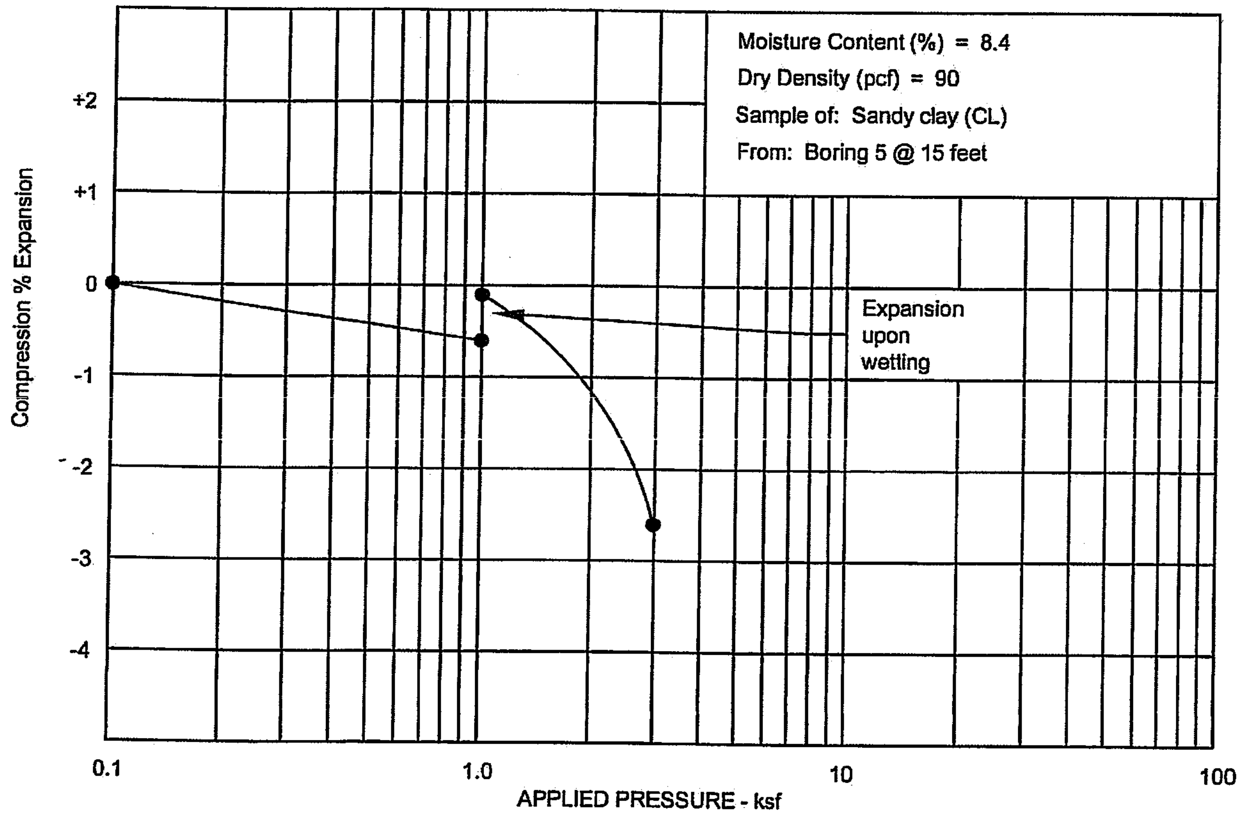
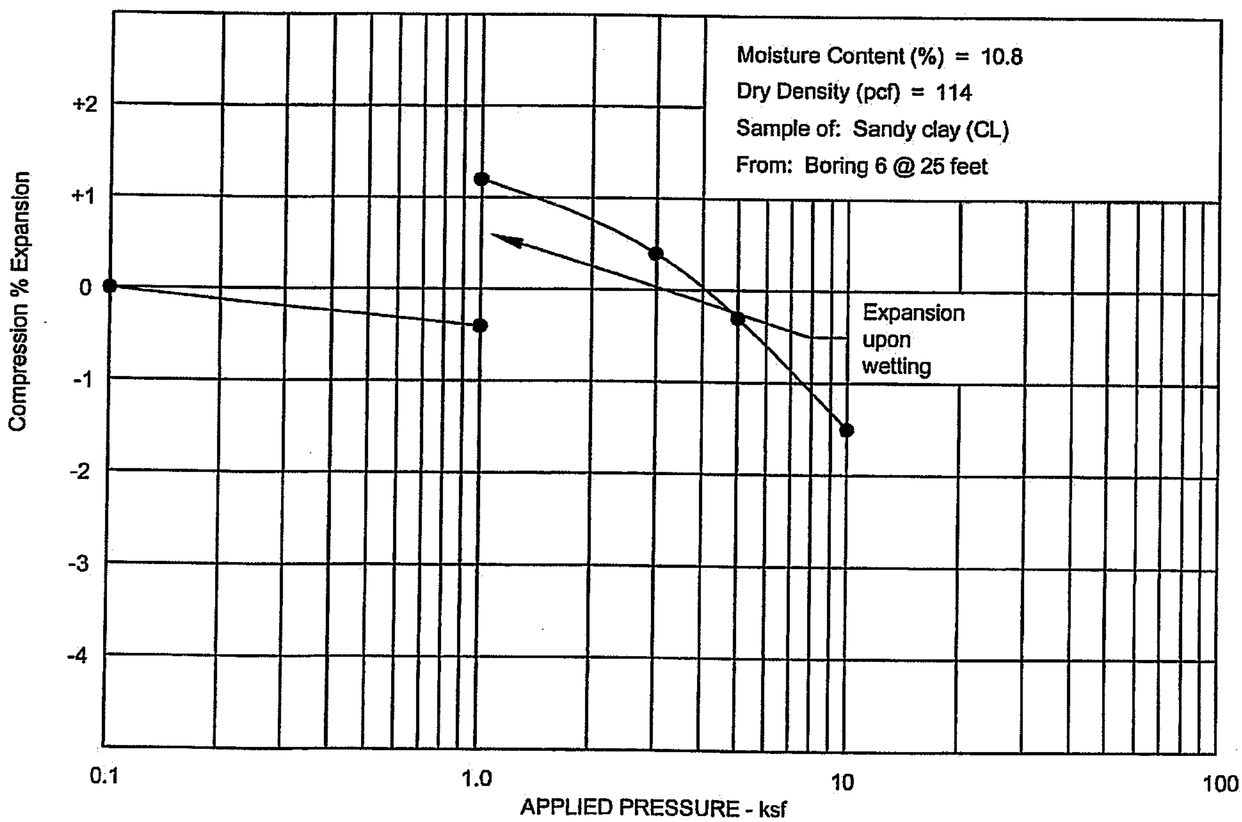
|---|---|---|---|
| 308121B | HEPWORTH - PAWLAK GEOTECHNICAL, INC. | HDT REGIONAL WATER RECLAMATION FACILITY SWELL-CONSOLIDATION TEST RESULTS | FIG. 10 |

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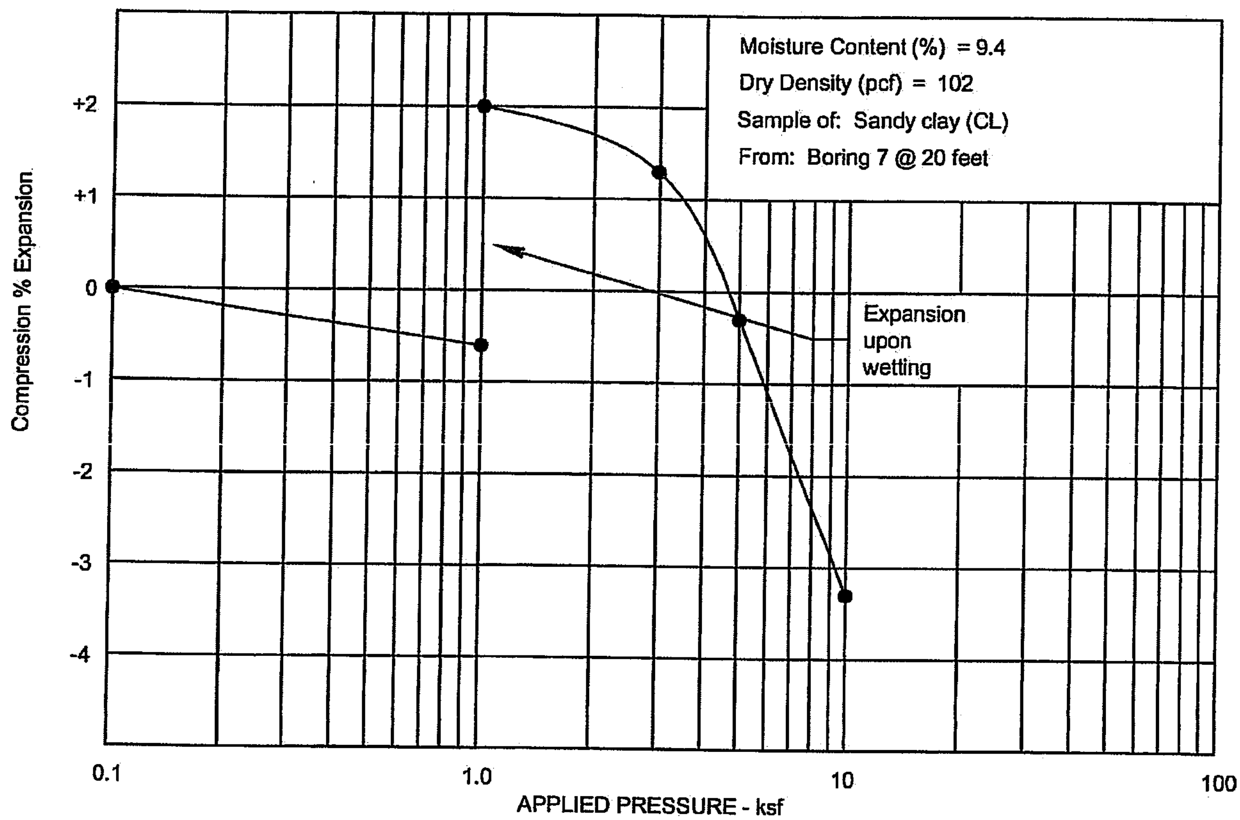
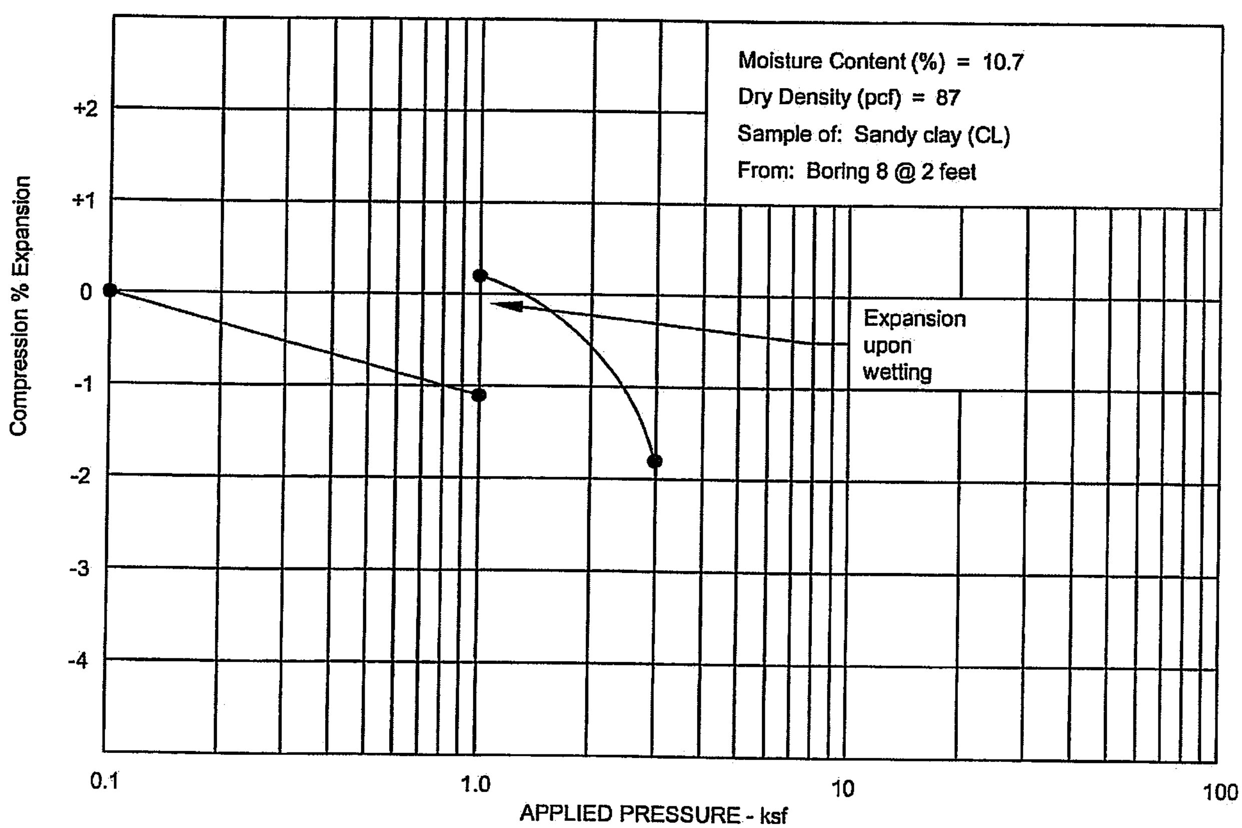
|---|---|---|---|
| 308121B | HEPWORTH - PAWLAK GEOTECHNICAL, INC. | HDT REGIONAL WATER RECLAMATION FACILITY SWELL-CONSOLIDATION TEST RESULTS | FIG. 11 |

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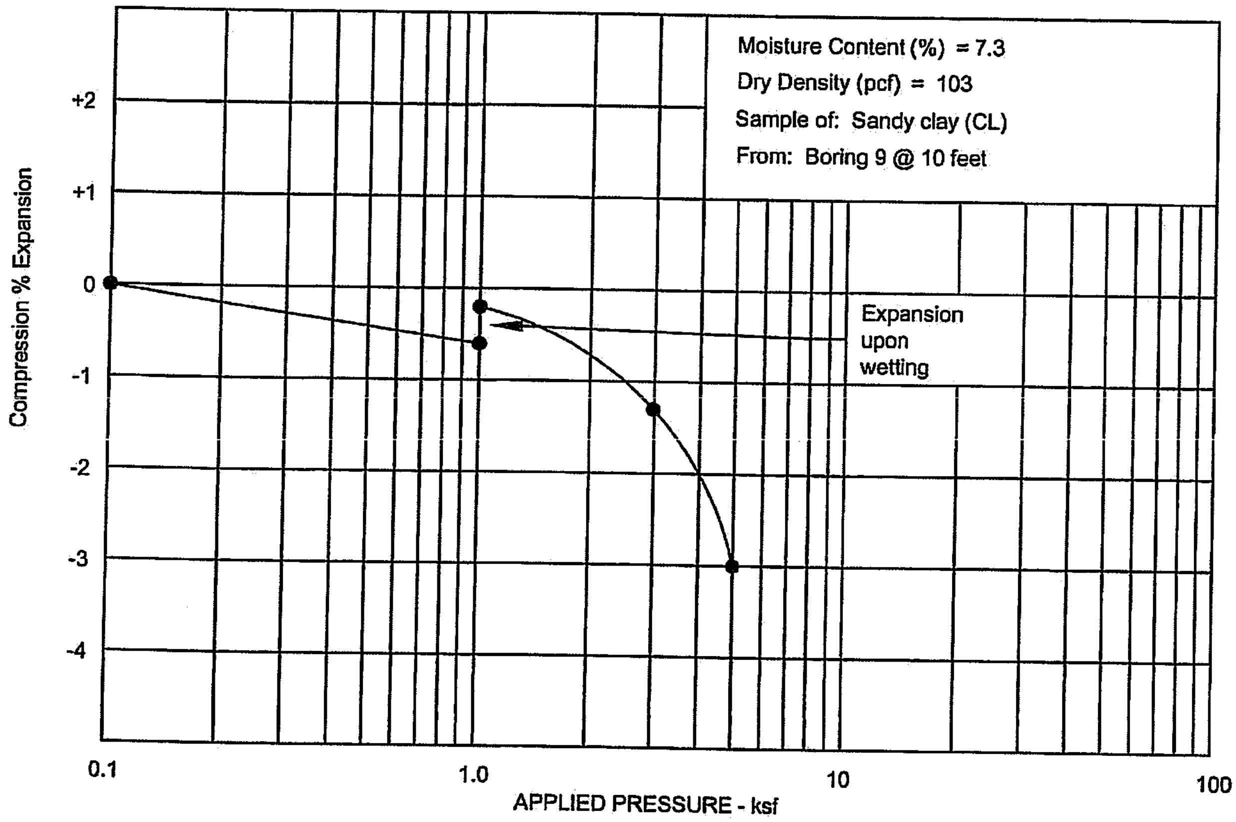
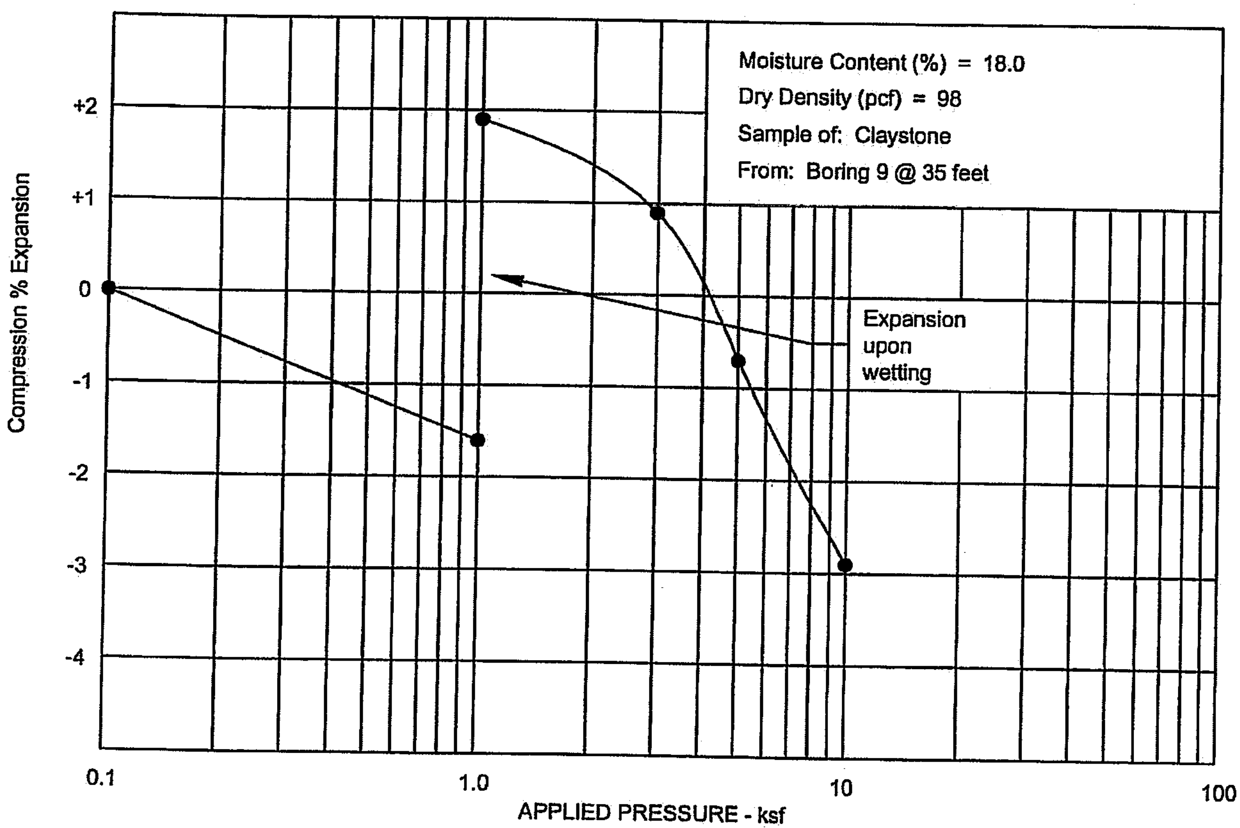
|---|---|---|---|
| 308121B | HEPWORTH - PAWLAK GEOTECHNICAL, INC. | HDT REGIONAL WATER RECLAMATION FACILITY SWELL-CONSOLIDATION TEST RESULTS | FIG. 12 |

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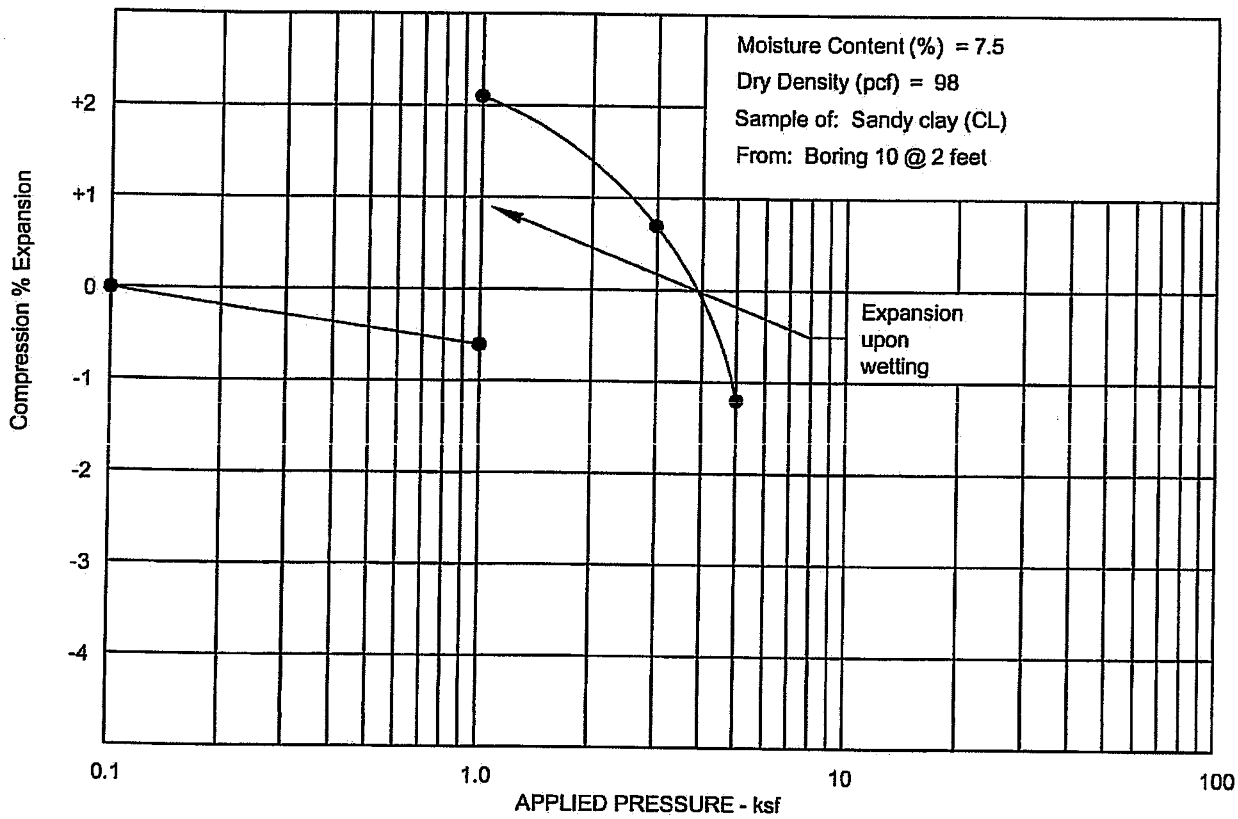
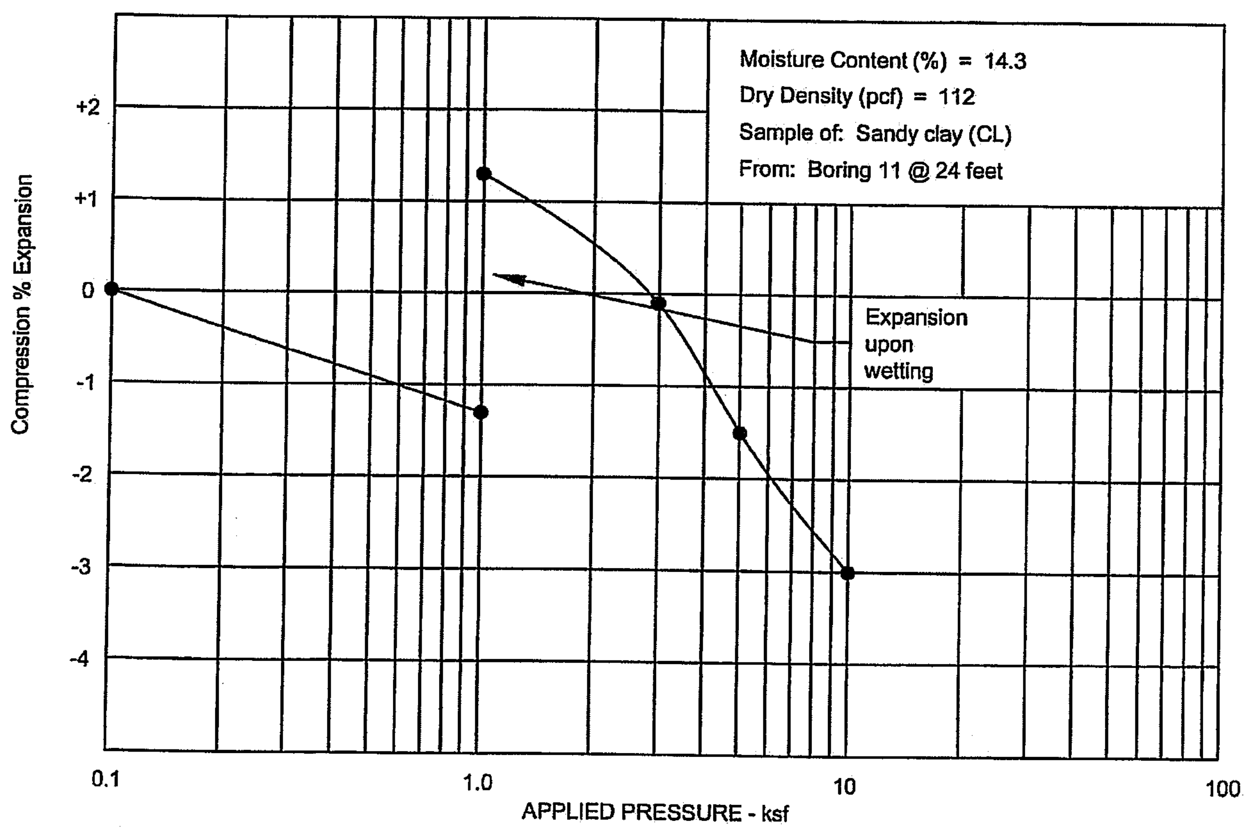
|---|---|---|---|
| 308121B | HEPWORTH - PAWLAK GEOTECHNICAL, INC. | HDT REGIONAL WATER RECLAMATION FACILITY SWELL-CONSOLIDATION TEST RESULTS | FIG. 13 |

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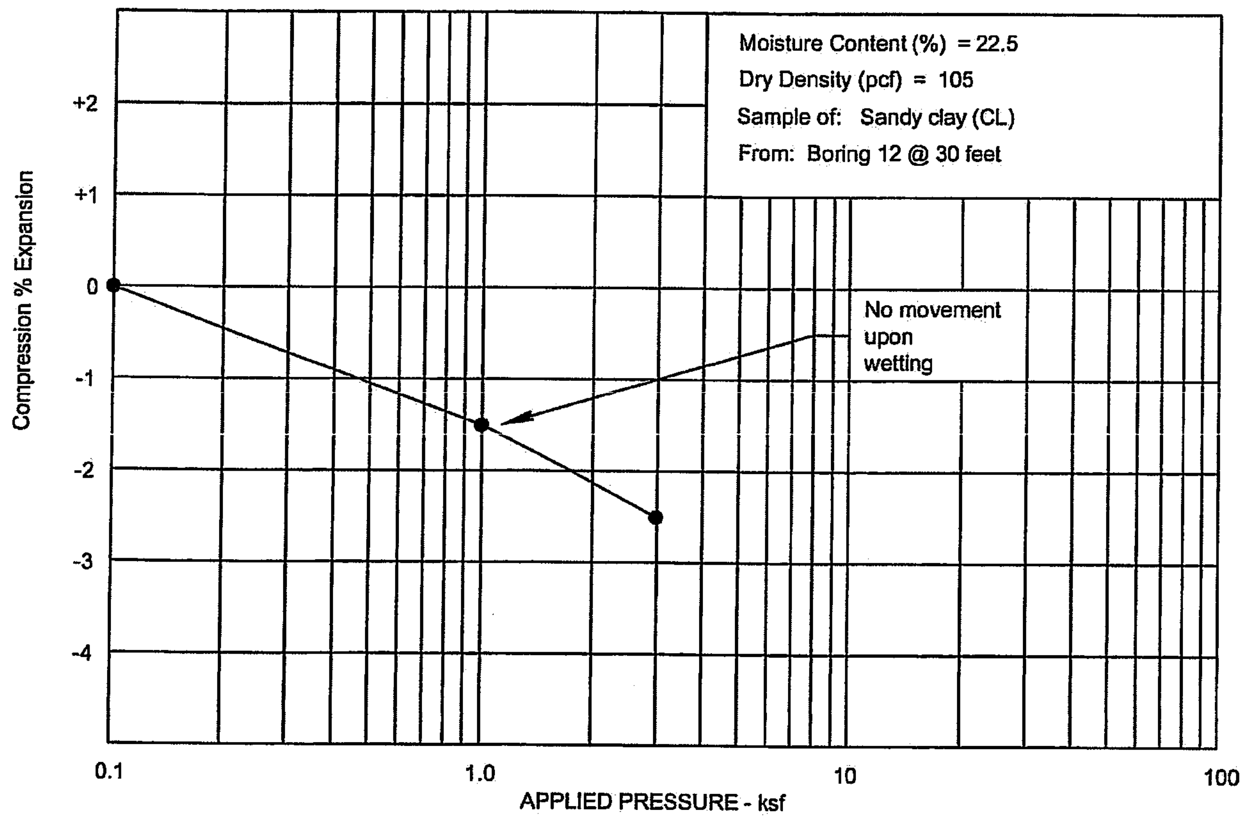
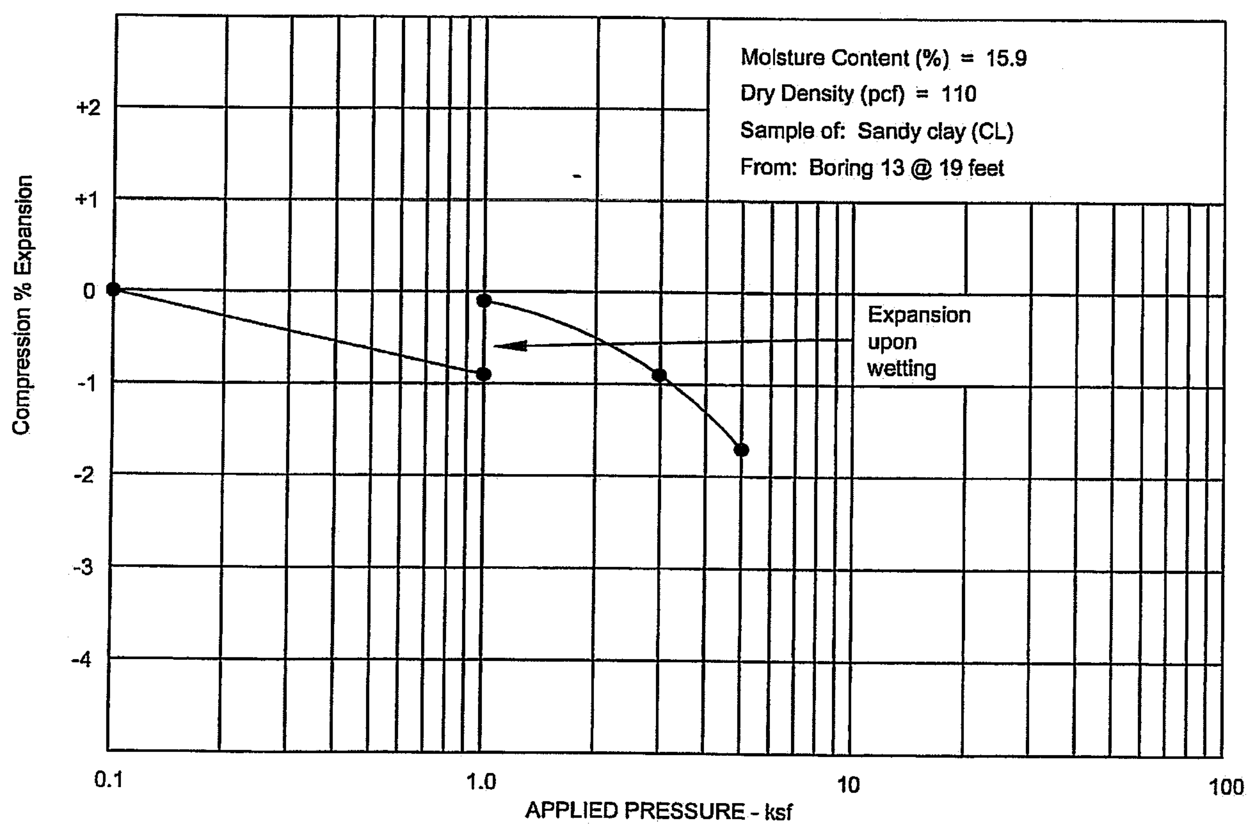
| 308121B | HEPWORTH - PAWLAK GEOTECHNICAL, INC. | HDT REGIONAL WATER RECLAMATION FACILITY SWELL-CONSOLIDATION TEST RESULTS | FIG. 14 |

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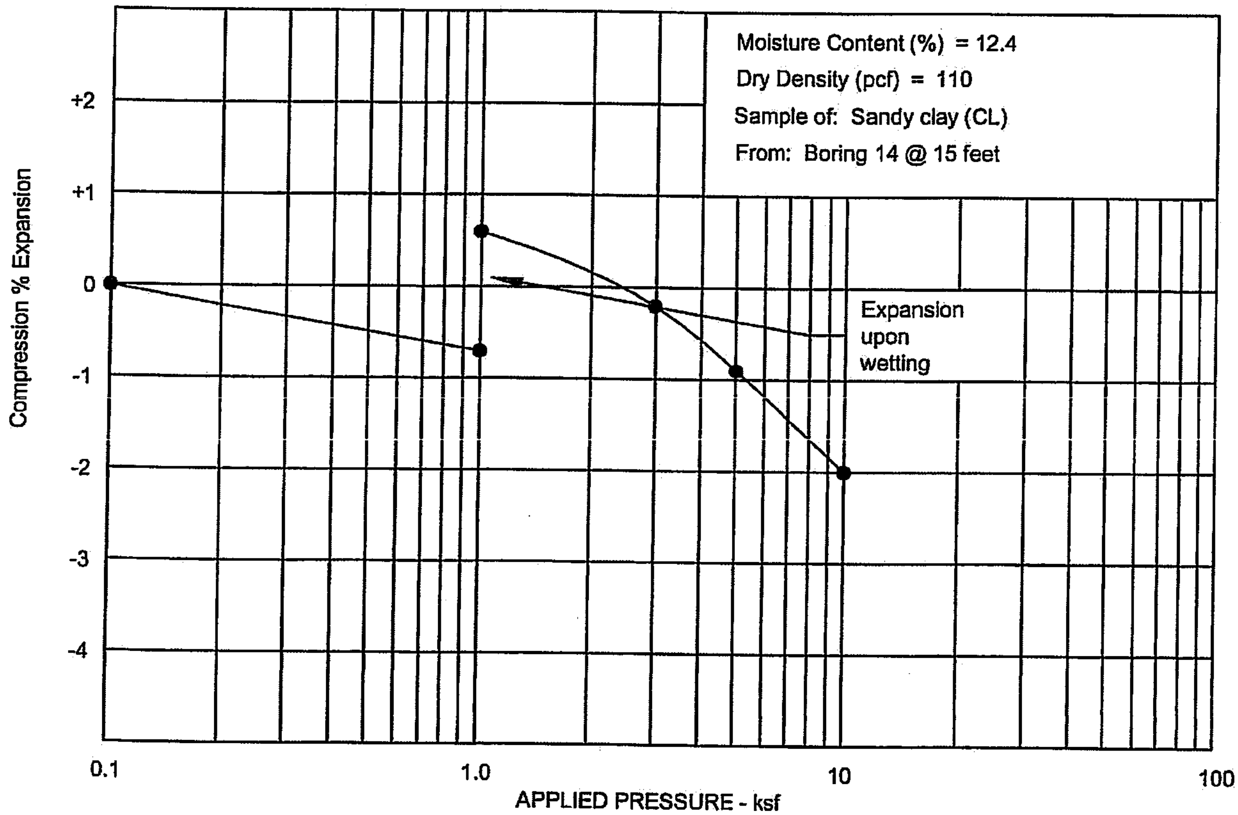
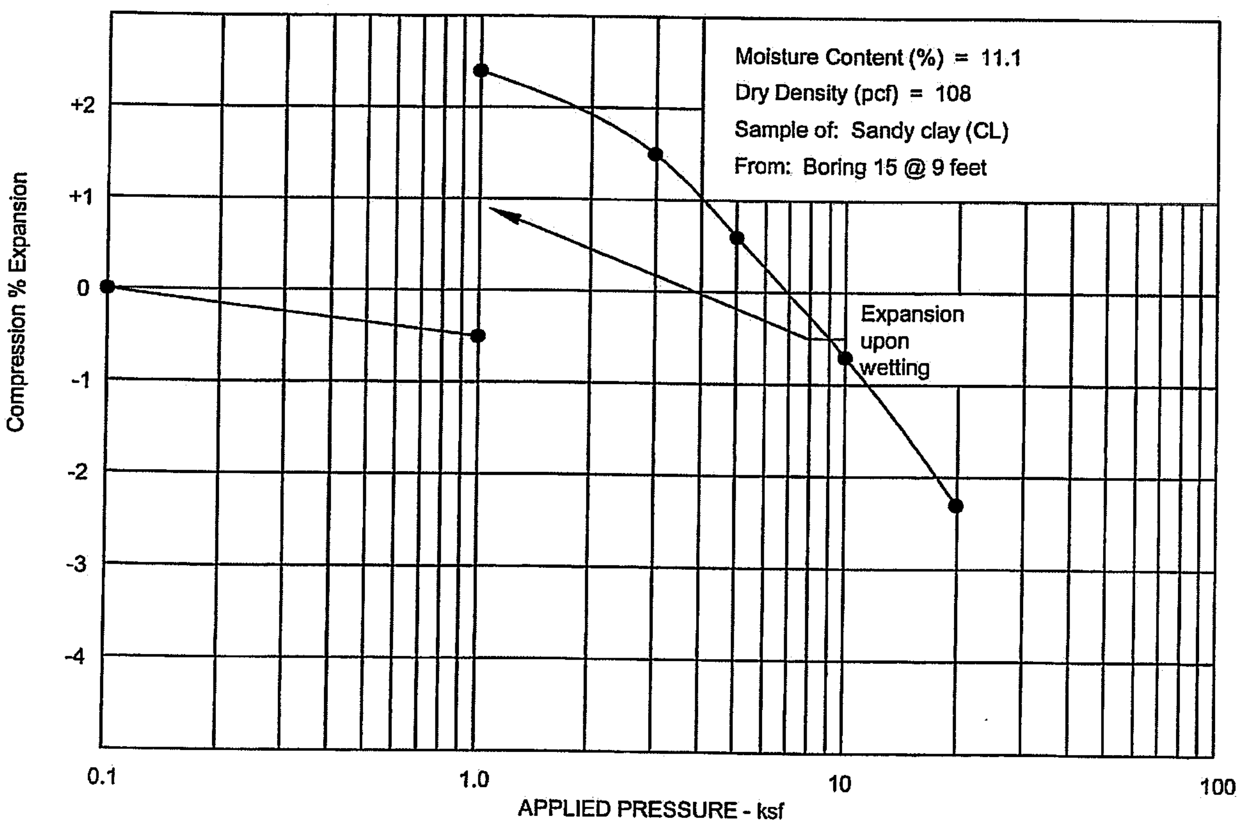
|---|---|---|---|
| 308121B | HEPWORTH - PAWLAK GEOTECHNICAL, INC. | HDT REGIONAL WATER RECLAMATION FACILITY<br>SWELL-CONSOLIDATION TEST RESULTS | FIG. 15 |

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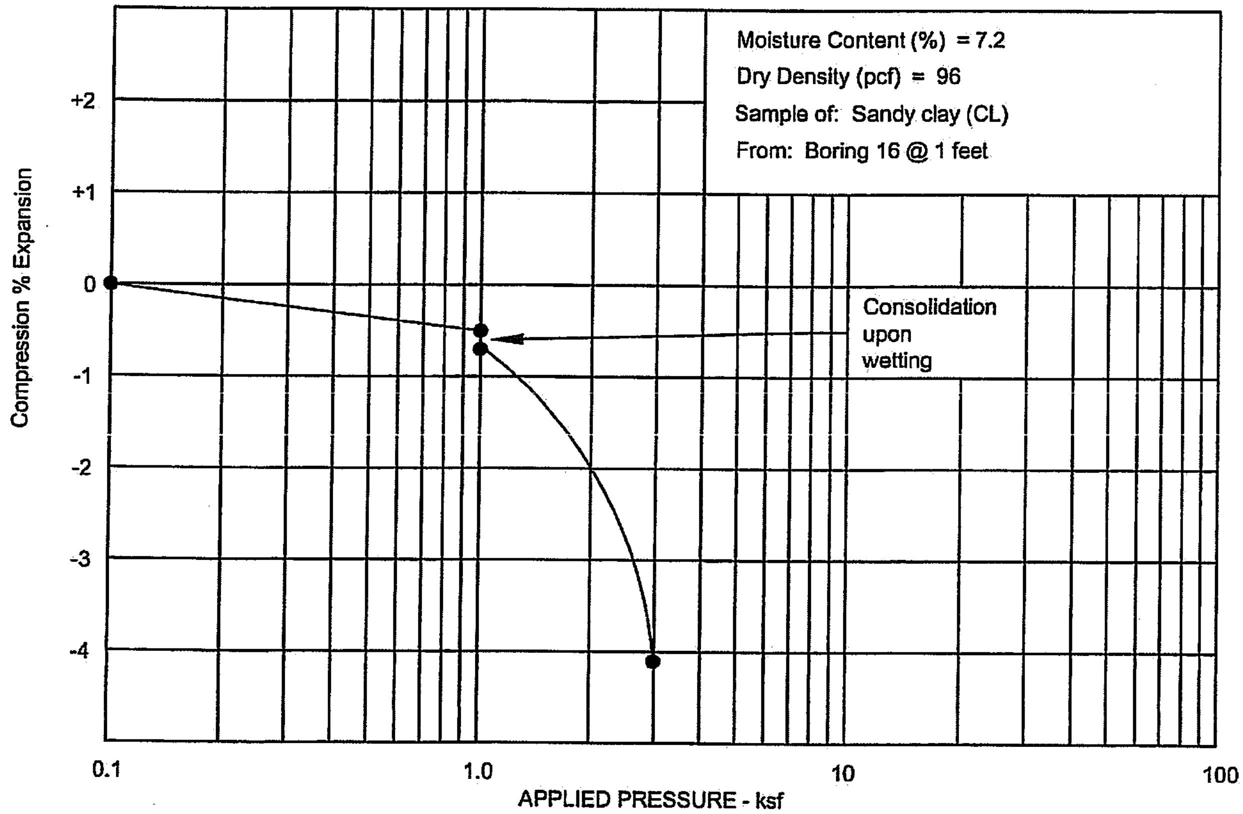
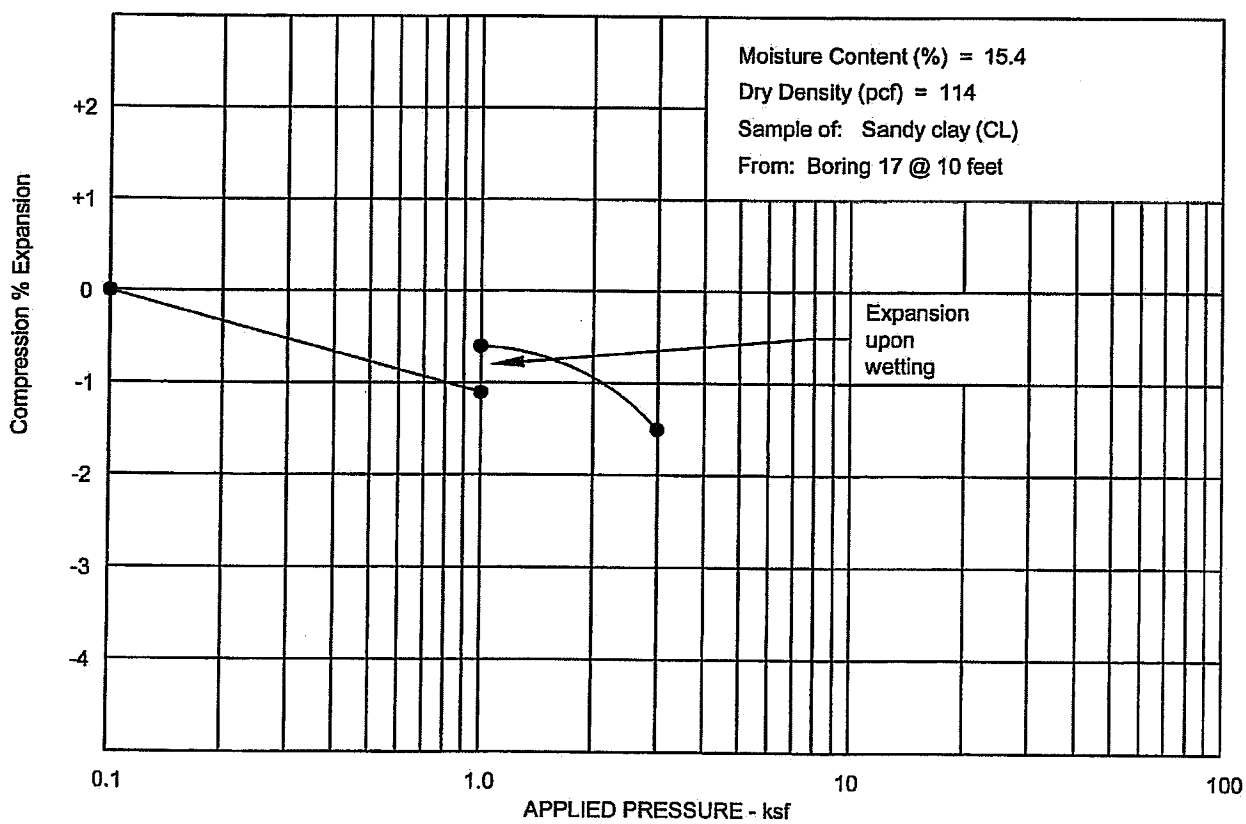
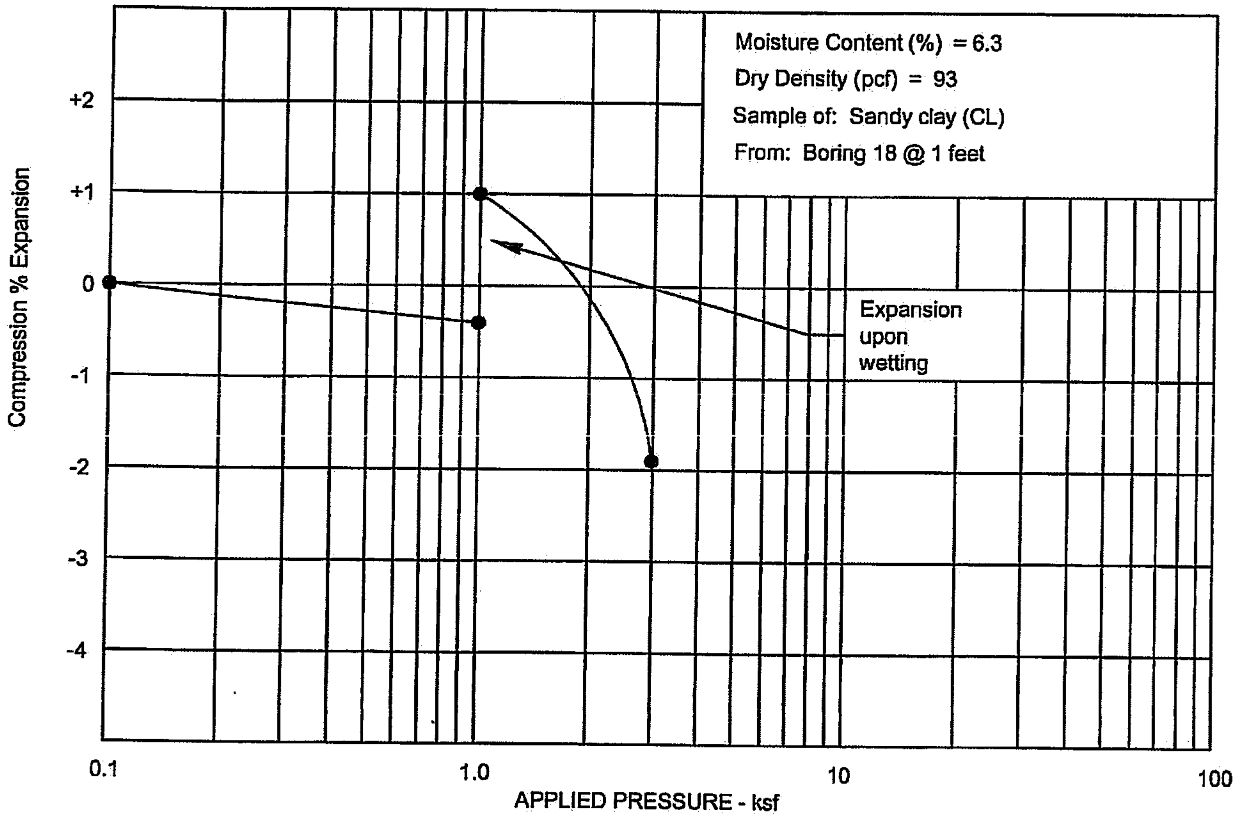
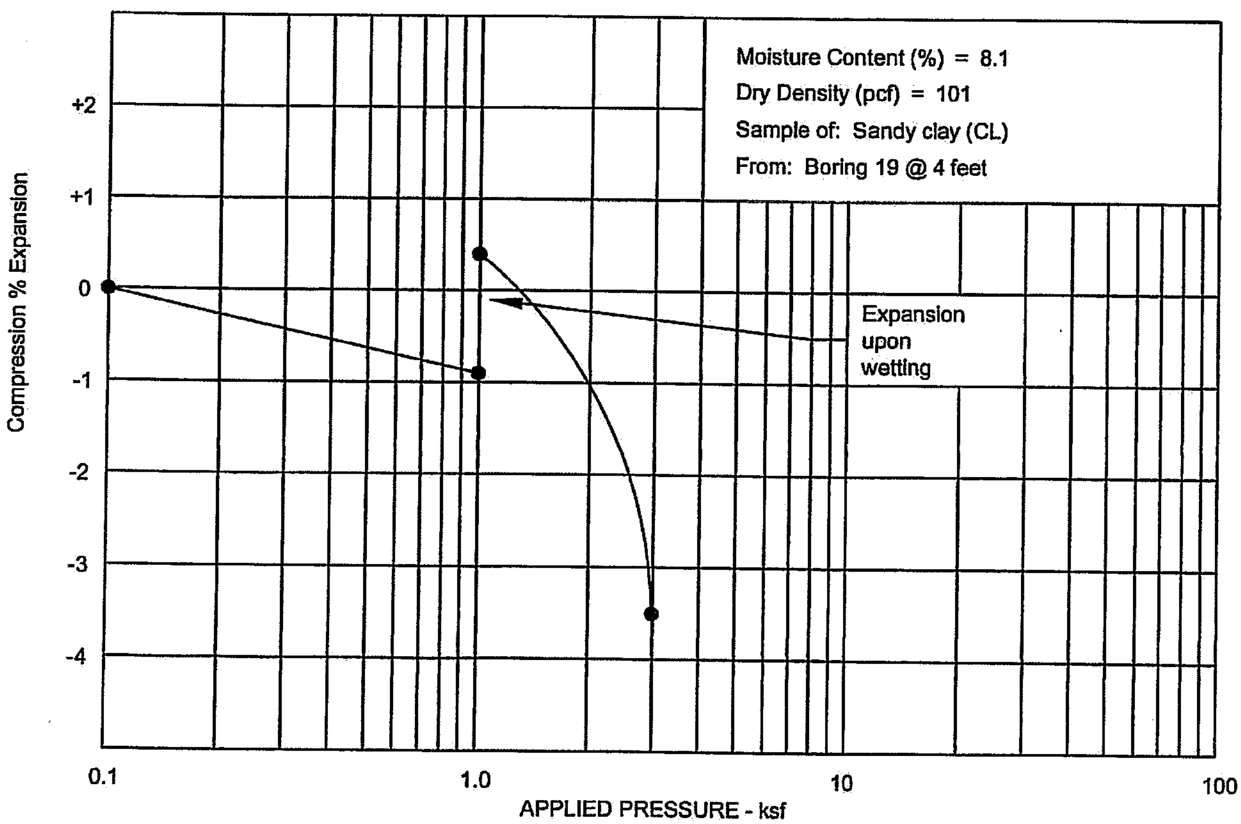
|---|---|---|---|
| 308121B | HEPWORTH - PAWLAK GEOTECHNICAL, INC. | HDT REGIONAL WATER RECLAMATION FACILITY<br>SWELL-CONSOLIDATION TEST RESULTS | FIG. 16 |




| | | | |
|---|---|---|---|
| 308121B | HEPWORTH - PAWLAK GEOTECHNICAL, INC. | HDT REGIONAL WATER RECLAMATION FACILITY SWELL-CONSOLIDATION TEST RESULTS | FIG. 17 |

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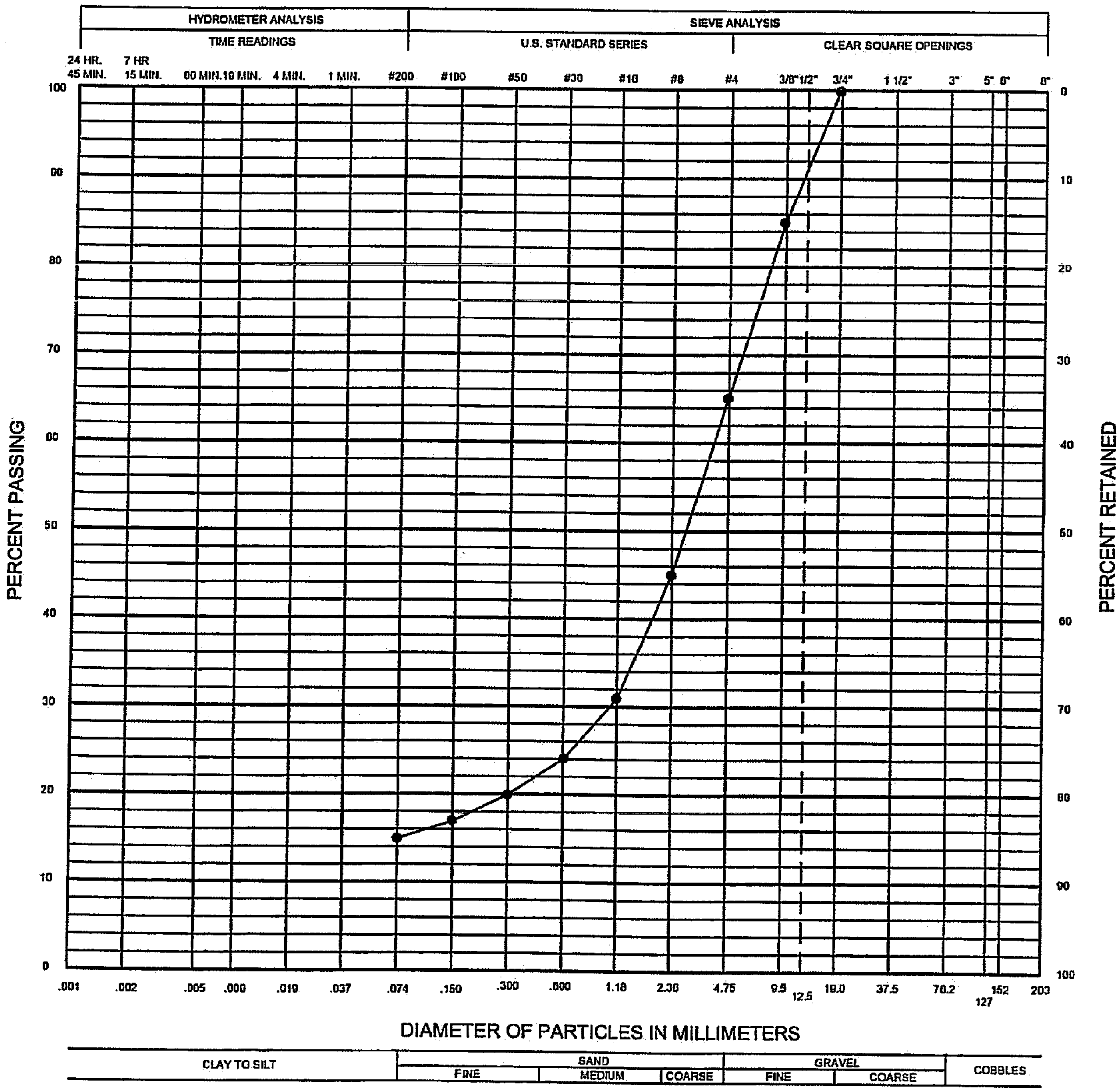
|---|---|---|---|
| 308121B | HEPWORTH - PAWLAK GEOTECHNICAL, INC. | HDT REGIONAL WATER RECLAMATION FACILITY<br>SWELL-CONSOLIDATION TEST RESULTS | FIG. 18 |

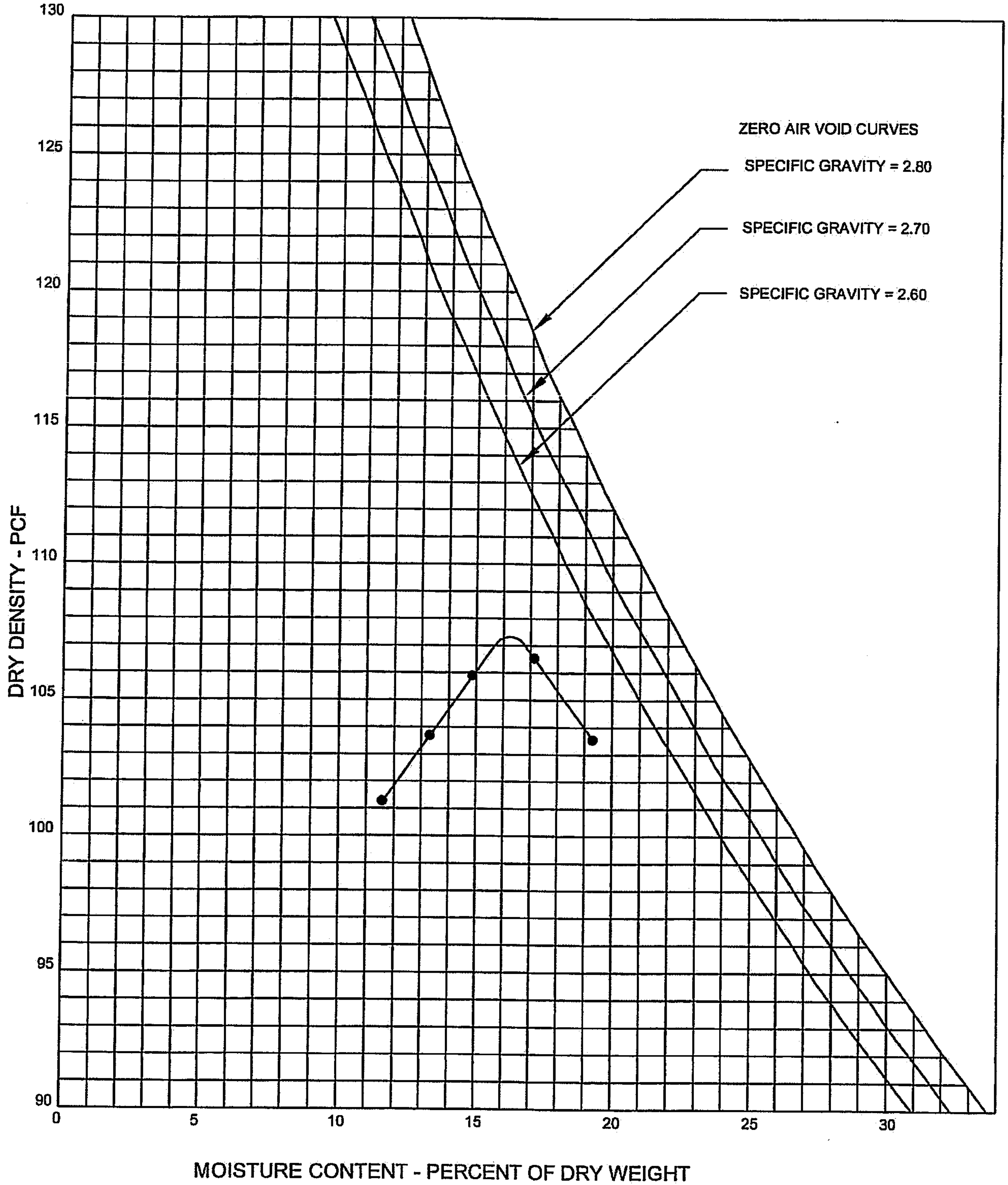



GRAVEL 35%    SAND 49%    SILT AND CLAY 16%

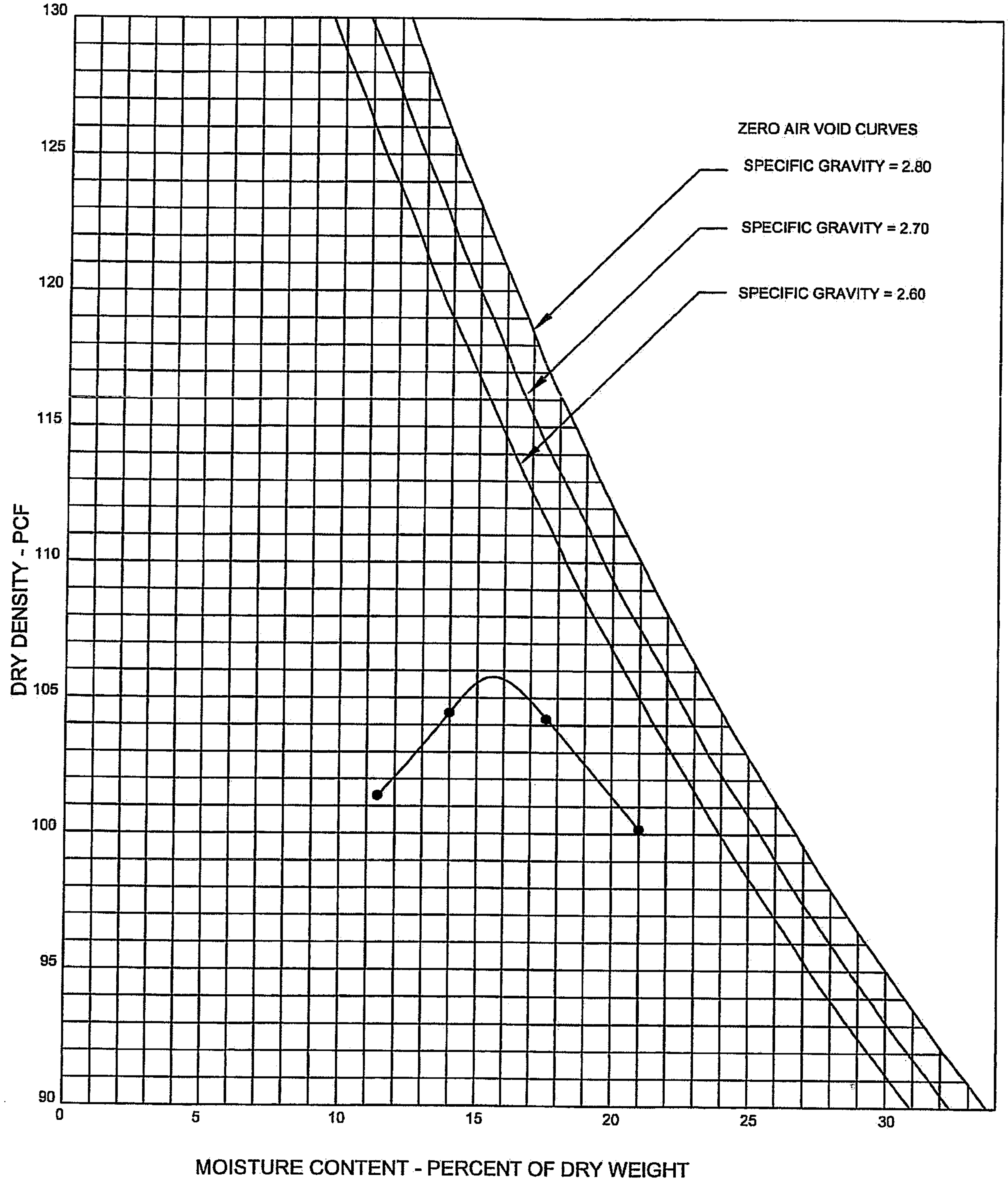
LIQUID LIMIT:    PLASTICITY INDEX: NP

SAMPLE OF: Silty gravelly sand (SM)    FROM: Boring 20 at 24 feet.

| 308121B | HEPWORTH - PAWLAK GEOTECHNICAL, INC. | HDT REGIONAL WATER RECLAMATION FACILITY<br>GRADATION TEST RESULTS | FIG. 19 |
|---|---|---|---|




| | | |
|---|---|---|
| LOCATION : HDT REGIONAL WATER RECLAMATION FACILITY | | MOISTURE-DENSITY RELATIONSHIPS |
| B3 @ 1-3 feet | | |
| SOIL DESCRIPTION : Sandy clay (CL) | | HEPWORTH-PAWLAK GEOTECHNICAL, Inc. |
| MAX. DRY DENSITY : 107.3 PCF | OPT. MOIST. CONTENT : 16.2 % | PROCEDURE : ASTM D698 |
| LIQUID LIMIT : 35 | PLASTICITY INDEX : 19 | JOB NO. : 308121B |
| GRAVEL : % SAND : % | SILT AND CLAY (-200) : 72 % | DATE : 1-29-08    Fig. 20 |




| | |
|---|---|
| LOCATION : HDT REGIONAL WATER RECLAMATION FACILITY<br>B10 at 1-5 feet | MOISTURE-DENSITY RELATIONSHIPS |
| SOIL DESCRIPTION : Sandy clay (CL) | HEPWORTH-PAWLAK GEOTECHNICAL, Inc. |
| MAX. DRY DENSITY : 105.8 PCF OPT. MOIST. CONTENT : 15.6 % | PROCEDURE : ASTM D698 |
| LIQUID LIMIT : 38 PLASTICITY INDEX : 23 | JOB NO. : 308121B |
| GRAVEL : % SAND : % SILT AND CLAY (-200) : 77 % | DATE : 1-29-08 FIG. 21 |

| SAMPLE LOCATION | | MOISTURE CONTENT (%) | DRY DENSITY (pcf) | GRADATION | | | ATTERBERG LIMITS | | SWELL UNDER 1KSF SURCHARGE (%) | AASHTO CLASSIFICATION | SOIL OR BEDROCK TYPE |
|---|---|---|---|---|---|---|---|---|---|---|---|
| BORING | DEPTH (FEET) | | | GRAVEL (%) | SAND (%) | SILT & CLAY (%) | LIQUID LIMIT | PLASTIC INDEX | | | |
| 1 | 4 | 7.9 | 103 | | | | | | | | Sandy clay (CL) |
| 2 | 2 | 9.8 | 97 | | | 89 | 41 | 26 | 2.1 | A-7-6(23) | Sandy clay )CL) |
| 2 | 14 | 10.2 | 106 | | | | | | 2.4 | | Sandy clay (CL) |
| 3 | 4 | 7.5 | 92 | | | 96 | 32 | 13 | 1.7 | A-6(12) | Sandy clay (CL) |
| 4 | 9 | 4.1 | 120 | | | | | | 0.5 | | Sandy clay (CL) |
| 5 | 15 | 8.4 | 90 | | | 95 | 29 | 12 | 0.6 | A-6(10) | Sandy clay (CL) |
| 6 | 5 | 7.6 | 92 | | | | | | | | Sandy clay (CL) |
| 6 | 25 | 10.8 | 114 | | | | | | 1.6 | | Sandy clay (CL) |
| 6 | 35 | 3.7 | 106 | | | 15 | | NP | | | Silty sand (SM) |
| 7 | 5 | 8.1 | 109 | | | | | | | | Sandy clay (CL) |
| 7 | 20 | 9.4 | 102 | | | | | | 2.6 | | Sandy clay (CL) |
| 8 | 2 | 10.7 | 87 | | | 95 | 46 | 30 | 1.3 | A-7-6(30) | Sandy clay (CL) |
| 9 | 10 | 7.3 | 103 | | | | | | 0.4 | | Sandy clay (CL) |
| 9 | 35 | 18.0 | 98 | | | 96 | 48 | 32 | 3.5 | A-7-6(32) | Claystone |
| 10 | 1-5 | | | | | 77 | 38 | 23 | | A-6(16) | Sandy clay (CL) |
| 10 | 2 | 7.5 | 98 | | | 94 | 36 | 20 | 2.7 | A-6(18) | Sandy clay (CL) |

| 308121B | HEPWORTH-PAWLAK GEOTECHNICAL, INC. | HDT REGIONAL WATER RECLAMATION FACILITY<br>SUMMARY OF LABORATORY TEST RESULTS | TABLE 1 |
|---|---|---|---|

| SAMPLE LOCATION | | MOISTURE CONTENT (%) | DRY DENSITY (pcf) | GRADATION | | | ATTERBERG LIMITS | | SWELL UNDER 1KSF SUR-CHARGE (%) | AASHTO CLASSI-FICATION | SOIL OR BEDROCK TYPE |
|---|---|---|---|---|---|---|---|---|---|---|---|
| BORING | DEPTH (FEET) | | | GRAVEL (%) | SAND (%) | SILT & CLAY (%) | LIQUID LIMIT | PLASTIC INDEX | | | |
| 11 | 24 | 14.3 | 112 | | | | | | 2.6 | | Sandy clay (CL) |
| 12 | 30 | 22.5 | 105 | | | | | | 0.0 | | Sandy clay (CL) |
| 13 | 19 | 15.9 | 110 | | | | | | 0.9 | | Sandy clay (CL) |
| 13 | 24 | 18.2 | 107 | | | | | | | | Sandy clay (CL) |
| 14 | 15 | 12.4 | 110 | | | | | | +1.3 | | Sandy clay (CL) |
| 15 | 9 | 11.1 | 108 | | | 97 | 36 | 20 | +2.9 | A-6(19) | Sandy clay (CL) |
| 15 | 19 | 14.9 | 115 | | | | | | | | Sandy clay (CL) |
| 16 | 1-5 | | | | | 80 | 34 | 18 | | A-6(13) | Sandy clay (CL) |
| 16 | 1 | 7.2 | 96 | | | | | | 0.0 | | Sandy clay (CL) |
| 17 | 10 | 15.4 | 114 | | | 92 | 35 | 20 | 0.5 | A-6(18) | Sandy clay (CL) |
| 18 | 1 | 6.3 | 93 | | | | | | 1.4 | | Sandy clay (CL) |
| 19 | 4 | 8.1 | 101 | | | 95 | 33 | 17 | 1.3 | A-6(16) | Sandy clay (CL) |
| 20 | 24 | 12.3 | 126 | 35 | 49 | 16 | | NP | | A-1-b(0) | Silty, gravelly sand (SM) |
| 21 | 24 | 14.6 | 121 | | | 25 | 34 | 20 | | A-2-6(0) | Clayey sand (SC) |
| 22 | 14 | 23.6 | 102 | | | 93 | 34 | 20 | | A-6(18) | Sandy clay (CL) |
| | | | | | | | | | | | |

| | | | |
|---|---|---|---|
| 308121B | HEPWORTH-PAWLAK GEOTECHNICAL, INC. | HDT REGIONAL WATER RECLAMATION FACILITY<br>SUMMARY OF LABORATORY TEST RESULTS | TABLE 1 |