



**GEOTECHNICAL INVESTIGATION  
BECHTEL TRAINING  
JETWAY COURT  
PUEBLO, COLORADO**

**Prepared for:**

**SHORT ELLIOTT HENDRICKSON, INC.  
503 N. Main Street, Suite 225  
Pueblo, Colorado 81003**

**Attention: Richard Cherry**

**Project No. SC02874-125**

**September 20, 2011**



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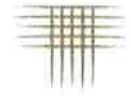
## **SCOPE**

**This report presents the results of our Geotechnical Investigation for improvements at the future Bechtel Training Center at the Pueblo Memorial Airport Business Park in Pueblo, Colorado. The purpose of our investigation was to evaluate subsurface conditions at the site and develop geotechnical design criteria for the proposed improvements. This report summarizes the results of our field and laboratory investigations, and presents our design and construction recommendations for new foundations, floor slabs, and pavement. We believe this investigation was completed in general conformance with our proposal (SC-11-0081) dated August 16, 2011.**

**The report was prepared based on conditions encountered in our exploratory borings, results of laboratory tests, engineering analysis, and our experience. The criteria presented are for the construction as described. Revision of the scope of the project could influence our recommendations. If changes occur, we should review the plans and their effect on our recommended design criteria. The following section summarizes the report. A more complete description of conditions found and our recommendations are included in the report.**

## **SUMMARY**

- 1. Subsoils found in our borings consisted of about 8 to 11 feet of sandy clay and 6 to 10 feet of clayey sand with gravel over claystone and shale bedrock. The sandy clay is moderately expansive.**
- 2. Ground water was not encountered in our borings during this investigation. Our experience in the area indicates the presence and amount of ground water can change significantly over relatively short horizontal and vertical distances.**
- 3. Considering the subsurface conditions encountered we believe a deep foundation system such as micropiles, helical piles, or drilled piers is more reliable to support new loads. Micropiles or helical piles will likely be more economical as installation of conventional drilled piers will be complicated by the limited head room inside the building. The presence of granular soils may limit the use of open-hole installation methods for micropiles. There is a risk that helical piles will torque out before required minimum length is reached. New foundations should be installed after the slab over-excavation is performed to reduce risk of damage to the new foundation elements.**



4. There is a high risk of heave and damage of slabs constructed on the sandy clay. Placing at least 4 feet of properly moisture conditioned and compacted fill beneath the slab will reduce the risk of movement. If granular fill material is used beneath the floor slab, a drain is recommended at the bottom of the fill layer. The over-excavation will be much more complicated than normal now that the building shell is in place. Care must be taken to not damage new or existing foundations during excavation.
5. New parking areas can be constructed with full depth asphalt or composite asphalt and base course sections.
6. Maintaining good surface drainage and controlling landscape irrigation around the building is critical to help reduce the risk that the underlying surficial soils will become wetted and heave. Site grading plans should provide for the rapid removal of surface water away from the structure and off of pavements. Landscaping plans should require vegetation that requires little or no supplemental irrigation.

## SITE CONDITIONS

The site is located in the Pueblo Memorial Airport Business Park in Pueblo, Colorado. The site contains an existing building shell. The single-story structure is about 26,000 square feet. The building is reportedly constructed on a driven pile foundation. The floor system has not been installed. The warehouse part of the building occupies most of the area and has masonry walls. The office part of the structure is located on the western part of the building and has light gauge steel framing with a brick veneer. The ground surface around the building is sparsely vegetated with weeds and grass. The ground surface slopes down to the south estimated at less than 2 percent. Industrial buildings were present on some of the surrounding lots. The general vicinity of the site is shown on Fig. 1.



Photo A – Exterior of building looking northeast



Photo B – Interior of Warehouse area (note no slab is installed).



## **PROPOSED CONSTRUCTION**

Based on the information provided, we understand the existing building shell will be finished. The existing unfinished office area will be finished with new office space. The western half of the warehouse area will be finished with new classrooms and the eastern part of the warehouse will be finished as the laboratory/training area. The laboratory/training area will require new foundations for a small mezzanine level and new floor-mounted cranes. Concrete slab-on-grade floors are planned in all areas. New parking areas are planned on the south side of the building.

## **GEOLOGY**

The area was mapped by Tweto in the USGS 1979 “Geologic Map of Colorado”. The site is mapped as being underlain by the Modern Alluvium (Qa). More detailed mapping of the site performed in 1964 by Scott, “Geology of the Northwest and Northeast Pueblo Quadrangles, Colorado” indicate the site is underlain by Piney Creek Alluvium (Qp). Our borings generally correlate with this mapping.

## **SUBSURFACE CONDITIONS**

Subsurface conditions for the addition were investigated by drilling three exploratory borings inside the existing warehouse portion of the building. The approximate locations of the borings from this investigation are shown on Fig. 1.

The borings were drilled to depths of 25 to 45 feet using a 4-inch diameter, continuous flight, auger and a truck-mounted drill rig. A representative of our firm observed drilling operations, obtained samples, and logged the subsurface conditions. Graphical logs of the conditions encountered and the results of field penetration resistance tests are presented on Fig. 2. Laboratory test results are shown on the Summary Log of Exploratory Boring (Fig. 2) and are summarized on Table 1. Swell-consolidation test results are shown on Figs. 3 through 8 and gradation tests are shown on Figs. 9 through 11.



The subsoils encountered during this investigation consisted of about 8 to 11 feet of sandy clay and 6 to 10 feet of clayey sand with gravel over claystone and shale bedrock to the maximum depths explored. Ground water was not encountered during our investigation. A more detailed description of the subsurface conditions is presented below.

### Clay

Sandy clay was found in all three borings from the existing ground surface (near proposed slab subgrade elevation) and extended to depths of 8 to 11 feet. The clay was stiff to very stiff based on field penetration resistance testing. Samples of the clay exhibited compression or moderate measured swell upon wetting under an applied pressure of 1,000 psf. Samples tested contained 65 to 87 percent silt and clay (passing the No. 200 sieve).

### Sand

Clayey sand was encountered in all three borings beneath the clay and extended to depths of 17 and 18 feet. The clayey sand contained varying amounts of gravel. The sand was medium dense to very dense based on field penetration resistance testing. Samples of the clayey sand exhibited compression upon wetting under an applied pressure of 1,000 psf. Samples tested contained 14 to 41 percent silt and clay (Passing the No. 200 sieve).

### Bedrock

Claystone bedrock was encountered in all three borings beneath the clayey sand and extended to depths of 25 to 40 feet. Shale was found in one boring beneath the claystone and extended to the maximum depth explored of 45 feet. The claystone is medium hard to very hard and the shale is very hard based on field penetration resistance tests.



## Ground Water

Ground water was not encountered in our borings during drilling or when the holes were checked twelve days later. Our experience in the area indicates the presence and amount of ground water can change significantly over relatively short horizontal and vertical distances. The ground water in this formation commonly flows through fissures and cracks in the bedrock as well as at the contact between the surficial soils and the bedrock surface.

## Seismicity

This area, like most of Colorado, is subject to a degree of seismic activity. Based upon subsurface conditions encountered during our investigation and our experience in the area, we believe the property classifies as Site Class C according to the 2009 International Building Code (IBC). The site may classify as Site Class B due to the relatively shallow depth and hardness of the bedrock. Geophysical testing such as Refraction Microtremor (ReMi) testing is required to verify a Site Class B designation. We can arrange for this testing if a lower seismic designation would be beneficial to the design of the structure.

## FOUNDATION SYSTEMS

The soils encountered in our investigation showed a wide range of swell-consolidation characteristics. The clayey sand is judged to be low swelling or non-expansive and the sandy clay is dry, hard, and expansive. This is typical of the Piney Creek Alluvium. Considering the history of reported movement and damage for various structures in the area, we recommend constructing new foundations for the mezzanine and the cranes on deep foundations such as micropiles, helical piles, or drilled piers. Installation of drilled piers will likely be hampered by the limited head room inside the building and the presence of granular soils above the bedrock may require the use of temporary casing. Micropiles may be an economical alternative to drilled piers, although the presence of granular soils may limit open-hole installation of micropiles.



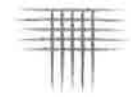
Recommendations for micropiles, helical piles, and drilled piers are presented below. We recommend foundations be designed and constructed in accordance with the following criteria:

### **Micropiles**

Micropiles are relatively small diameter (usually 3.5 to 8 inches), heavily reinforced, grouted piles. Micropiles can be constructed many ways including using open-hole methods where the hole is drilled and cuttings are flushed with air or a continuous-flight auger is used. The reinforcing is then set and grout is placed from the bottom of the hole using grout tubes. Other methods include grouting continuously with a thin grout that is pumped through hollow reinforcing steel with a sacrificial drill bit attached to the end. Considering the subsurface conditions encountered and the presence of granular soils, open-hole methods may be limited and continuous grouting or the use of a hollow-stem-auger will probably be required to install micropiles.

The installation methods for micropiles greatly affect the bond stress between the micropile and the surrounding soils. Design capacities must be verified in the field through full-scale load testing. Micropile construction is generally performed on a design/build basis. The micropile contractor and their engineer typically design the piles for minimum length depending on anticipated loading. We recommend micropiles penetrate a minimum of 6 feet into the claystone bedrock and have a minimum length of at least 22 feet. Additional guideline design criteria are provided below. We can provide contact information for some local micropile contractors if you would like.

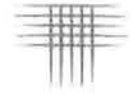
1. Commonly available micropile systems have working capacities in the range of 20 to 100 kips.
2. Micropiles should be designed and installed in accordance with Case I, Type A requirements as specified in USDOT publication FHWA-SA-97-070.
3. Micropiles should penetrate at least 12 feet into the claystone bedrock and have a minimum length of at least 22 feet.
4. Micropiles should be reinforced their full length.



5. There should be an 4-inch (or thicker) continuous void beneath all grade beams and foundation wall, between piles.
6. Drilling methods should be determined by the contractor. Dry rotary or air flush methods are preferred to water flush.
7. Micropiles should have a minimum diameter of 3.5 inches. Larger sizes may be used.
8. Preliminary design should be based on a grout/ground interface bond strength of 7,500 psf in the bedrock and a service load factor of 2.5 (i.e.  $7,500/2.5 = 3,000$  psf allowable). The contractor must verify this strength is appropriate through full-scale load testing in the field.
9. Micropiles should be spaced 3 feet or more apart to avoid group efficiency effects.
10. Installation of micropiles should be observed by a representative of our firm to confirm depth and penetration into bedrock.

#### Helical Piles

1. Helical piles should and have a minimum length of 14 feet. Test piles may be installed and load tested to obtain better estimate of pile length. Minimum length should be measured from the top of the pile to the top helical plate. The piles should be installed as close to vertical as possible or at angles specified by the structural engineer.
2. The ultimate capacity of helical piles should be calculated based on the manufacturer's recommendations. We recommend calculation of the installation torque using a factor of safety of at least 2 when converting ultimate values to working (allowable) capacity. The allowable pressure on the soils below the helical plate area should not exceed 8,000 psf for the over burden soils at least 14 feet below existing grades. An allowable bearing pressure of 20,000 psf is appropriate for the claystone. Helical pile capacity should be verified in the field using load tests and/or manufacturer recommended capacity torque ratios. Contractors should use the number and size of helices required to achieve depth, torque and capacity; the use of multiple helices may be necessary.
3. Twisting of the shaft can occur during the installation process. The structural engineer should evaluate the effect(s) twisting of the shaft may have on the capacity and corrosion protection of the helical piles. The structural engineer should specify the maximum torque which should be applied to avoid over-stressing the piles.
4. The pile caps and the connection between the piles and grade beam should be able to resist both tension and compression and be designed to resist lateral earth pressure. A structural engineer should design this connection.



5. We recommend contacting the manufacturer or the manufacturer's representative concerning corrosion protection of the steel. Manufacturer's recommendations should be followed.
6. Grade beams should be well reinforced; the reinforcement should be designed by the structural engineer. Lateral earth pressure and the effects of large openings within grade beams should be considered.
7. The installation and load testing of helical piles should be observed by a representative of our firm to confirm the depth and installation torque identified by structural engineer are achieved.

### Drilled Piers

1. Piers should be designed for a maximum allowable end pressure of 30,000 psf and an allowable skin friction of 3,000 psf for the portion of pier in bedrock.
2. Piers should be designed for a minimum deadload pressure of 10,000 psf based on pier cross-sectional area. If this deadload cannot be achieved through the weight of the structure, the pier length and bedrock penetration should be increased beyond the minimum values specified in the next paragraph. The bedrock should be assigned a skin friction value of 3,000 psf for uplift resistance.
3. Piers should penetrate at least 6 feet into the claystone bedrock and have a minimum drilled and concreted length of at least 22 feet. The limited headroom inside the building will likely require the use of a low overhead drilling rig to install piers.
4. Piers should be reinforced their full-length to resist tension in the event of swelling. We recommend the cross-sectional area of reinforcement be equal to at least 1.0 percent of the gross cross-sectional area of the pier. Grade 60 (420 Mpa) reinforcing bars (or equivalent) should be used. Reinforcement should extend into grade beams and foundation walls.
5. There should be an 4-inch (or thicker) continuous void beneath all grade beams and foundation walls, between piers, to concentrate the deadload of the structure on the piers.
6. Piers should have a center-to-center spacing of at least three pier diameters when designing for vertical loading conditions, or they should be designed as a group. Piers aligned in the direction of lateral forces should have a center-to-center spacing of at least six pier diameters. Reduction factors for closely-spaced piers are discussed in a subsequent section of the report.
7. Some movement of the drilled pier foundation is anticipated to mobilize the strength of the bedrock. We estimate this movement to be on the order of ¼ to ½-inch to mobilize skin friction. Differential movement may be



equal to the total movement. Designs should consider these potential movements and accommodate them as much as practical.

8. Piers should be carefully cleaned prior to placement of concrete. Our experience indicates the presence and amount of ground water can vary significantly over relatively short vertical and horizontal distances. Ground water was not encountered during our investigation but, sporadic ground water is common in the area. Temporary casing, tremie equipment, and/or pumping may be necessary for proper cleaning, dewatering, and concrete placement. Concrete should not be placed by free fall if there is more than 3 inches of water in the bottom of the hole.
9. Concrete placed in cased pier holes should have sufficient slump to fill the pier holes and not hang on the sides of the casing or reinforcement during casing removal. We recommend a slump in the range of 5 to 7 inches if casing is used. The slump should be 4 to 6 inches for uncased piers.
10. Formation of mushrooms or enlargements at the top of piers should be avoided during pier drilling and subsequent construction operations.
11. Installation of drilled piers should be observed by a representative of our firm to identify the proper bearing strata and observe construction techniques.

## **SLABS-ON-GRADE**

We believe there is high risk of movement and associated damage of slabs constructed on the sandy clay. Our experience indicates slab movement on the order of 2 to 3 inches is possible for these types of conditions. In our opinion, the most reliable solution is the use of a structurally supported floor, however these types of floor systems are typically uneconomical or impractical in warehouse or industrial type structures. Structurally supported floors are recommended in areas where floor slab movement cannot be tolerated such as areas where movement-sensitive equipment is located.

The risk of movement and associated damage of slabs can be reduced by placing a zone of properly moisture conditioned and compacted fill beneath slabs-on-grade. The fill layer tends to reduce differential movement that causes cracking and should reduce total heave. If the owner is willing to accept the risk of slab movement and associated damage, we recommend placement of at least 4 feet of moisture conditioned and compacted fill below slabs.



Either the on-site clay or imported granular fill may be used as slab subgrade fill. Clay fill placed beneath slabs will need to be conditioned to high moisture content to reduce potential swell. The on-site clay is relatively dry and will require extensive processing during placement as fill. Pre-wetting of the clay material during removal and watering stockpiles will aid processing. A construction disc will be required to thoroughly mix the material and obtain a relatively uniform moisture content during placement. Materials may need to be excavated and mixed outside of the building shell due to space limitations.

If the on-site clay is used as slab subgrade fill, the material should be moisture conditioned from 1 to 4 percent above optimum moisture content and compacted to at least 95 percent of maximum standard Proctor dry density (ASTM D 698).

It may be better to replace the clay with an inert sand and gravel soil. Imported granular slab subgrade fill should have a maximum particle size of 1-½ inches, less than 35 percent passing the No. 200 sieve, a Liquid Limit of 30 percent or less, and a Plasticity Index of 15 or less. Granular slab subgrade fill should be moisture conditioned to within 2 percent of optimum and compacted to at least 95 percent of maximum modified Proctor dry density (ASTM D 1557).

Removing a zone of the expansive material and replacing it with granular fill can create a low area of relatively higher permeability (bathtub) compared to the surrounding native soil or bedrock. This area can accumulate and retain water over time. We recommend construction of a below fill drain to collect and remove this water if a relatively permeable backfill such as granular fill is used. The surface of the excavation should be sloped toward the perimeter of the excavation and a drain installed. The drain should consist of a perforated, 3 or 4-inch diameter, rigid PVC pipe encased in free draining gravel wrapped in filter fabric. The drain should slope at constant grade to a positive gravity outfall on the ground surface well away from the structure or connect to a sump where water can be removed by pumping. A typical below granular fill drain detail is shown on Fig. 12. The below fill drain can be omitted if relatively impermeable material such as the on-site clay is used as slab subgrade fill.



Some movement of floor slabs relative to foundations is likely. We recommend slab-on-grade floors be separated from exterior walls and interior bearing members with joints that allow free vertical movement of slabs. Slip-joints in slab-bearing partitions should allow for at least 2 inches of free vertical movement. If the “float” is provided at the tops of partitions, the connection between interior, slab-supported partitions and exterior, foundation-supported walls should be detailed to allow differential movement. These architectural connections are critical to help reduce cosmetic damage when foundations and floor slabs move relative to each other, as happens when slabs heave or when foundations settle relative to more lightly loaded slabs. We have seen instances where these architectural connections were not designed and constructed properly and resulted in moderate cosmetic damage, even though the movement experienced was well within the anticipated range. The architect should pay special attention to these issues and detail the connections accordingly.

Frequent control joints should be provided in the slabs to reduce the effects of curling and to help control shrinkage cracking. Where underslab plumbing is necessary, service lines should be pressure tested for leaks during construction. Utility lines that penetrate the slabs should be separated and isolated from the slabs with joints to allow for free vertical movement.

The 2009 International Building Code (IBC) requires a vapor retarder be placed between base course or the subgrade soils and the concrete slab-on-grade floor, unless the designer of the floor (structural engineer) waives this requirement. The merits of installation of a vapor retarder below floor slabs depend on the sensitivity of floor coverings and building use to moisture. A properly installed vapor retarder (10 mil minimum) is more beneficial below concrete slab-on-grade floors where floor coverings, painted floor surfaces, or products stored on the floor will be sensitive to moisture. The vapor retarder is most effective when concrete is placed directly on top of it, rather than placing a sand or gravel leveling course between the vapor retarder and the floor slab. The placement of concrete on the vapor retarder may increase the risk of shrinkage cracking and curling. Use of concrete with reduced shrinkage characteristics including minimized water content, maximized coarse aggregate content, and reasonably low slump will reduce the risk of shrinkage



cracking and curling. Considerations and recommendations for the installation of vapor retarders below concrete slabs are outline in Section 3.2.3 of the 2006 report of the American Concrete Institute (ACI) Committee 302, "Guide for Concrete Floor and Slab Construction (ACI 302.R-96)".

## **BELOW-GRADE CONSTRUCTION**

We understand no new below-grade areas such as a basement is planned. For areas where floors are at or above exterior grades, a foundation drain is not required.

## **PAVEMENTS**

Our investigation indicates subsoils in new pavement areas will likely consist of clay. Based on the results of our investigation, we considered a Hveem-Stabilometer ("R") value of 5 for the subgrade soils. We assigned a Daily Traffic Number (DTN) of 5 to the proposed automobile parking areas and drive lanes. This value corresponds to an equivalent 20-year, 18-kip single-axle load (ESAL) application values of 36,500. Our calculations indicate the automobile parking areas and access driveways can be paved with 6 inches of asphalt concrete or 4 inches of asphalt concrete over 7 inches of aggregate base course. Concrete pavement (minimum 6 inches) may be used in trash enclosure areas where garbage trucks load.

Our design considers pavement construction will be completed in accordance with Colorado Department of Transportation (CDOT) specifications. The specifications contain requirements for the pavement materials (asphalt concrete, base course, and plain concrete) as well as the construction practices used (compaction, materials sampling, and proof-rolling). Of particular importance are those recommendations directed toward subgrade and base course compaction and proof-rolling. During proof-rolling, particular attention should be directed toward the areas of confined backfill compaction. Areas that pump or deflect significantly should be stabilized prior to pavement construction. A representative of our office should be present at the site during construction of pavements to perform density testing.



## CONCRETE

Concrete in contact with soils can be subject to sulfate attack. We measured soluble sulfate concentrations in one sample from this site. The measured concentration was 0.1 percent. Water-soluble sulfate concentrations between 0.1 and 0.2 percent indicate Class 1 exposure to sulfate attack, according to ACI 201.2R-01 as published in the 2008 ACI Manual of Concrete Practice. The American Concrete Institute (ACI) indicates adequate sulfate resistance can be achieved by using Type II cement with a water-to-cementitious material ratio of 0.5 or less. ACI also indicates concrete in Class 1 exposure environments should have a minimum compressive strength of 4,000 psi.

## SURFACE DRAINAGE / IRRIGATION

Performance of the foundations, floor slabs, concrete flatwork, and pavements is influenced, to a large degree, by the moisture conditions existing within the near-surface soils. Overall surface drainage patterns should be planned to provide for the rapid removal of storm runoff. Water should not be allowed to pond adjacent to building foundations, over pavements, or at the crest of permanent slopes. We recommend the following precautions be observed during construction and maintained at all times after the addition is completed.

1. Foundation wall backfill should be graded to provide for the rapid removal of runoff. We suggest a slope equivalent to at least 6 inches in the first 10 feet. In paved areas, the slope may be reduced to at least 2 inches in the first 10 feet.
2. Exterior foundation wall backfill should be moisture conditioned to within 2 percent of optimum and compacted to at least 95 percent of maximum standard Proctor dry density (ASTM D 698).
3. Roof downspouts and drains should discharge well beyond the limits of all backfill. Downspout extensions and splash blocks should be provided.
4. Landscaping concepts should concentrate on use of native plants that require little or no supplemental irrigation after the establishment period.



## **CONSTRUCTION OBSERVATIONS**

**We recommend that CTL|Thompson, Inc. provide observation and testing services during construction to allow us the opportunity to verify whether soil conditions are consistent with those found during this investigation. If others perform these observations, they must accept responsibility to judge whether the recommendations in this report remain appropriate.**

## **GEOTECHNICAL RISK**

**The concept of risk is an important aspect with any geotechnical evaluation primarily because the methods used to develop geotechnical recommendations do not comprise an exact science. We never have complete knowledge of subsurface conditions. Our analysis must be tempered with engineering judgment and experience. Therefore, the recommendations presented in any geotechnical evaluation should not be considered risk-free. Our recommendations represent our judgment of those measures that are necessary to increase the chances that the structure will perform satisfactorily. It is critical that all recommendations in this report are followed during construction.**

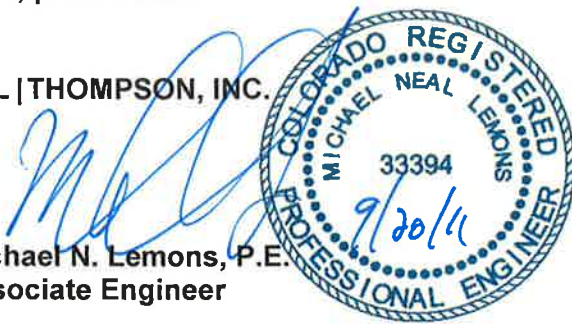
## **LIMITATIONS**

**Our borings was located to obtain a reasonably accurate indication of subsurface foundation conditions. The borings are representative of conditions encountered at the exact boring location only. Variations in subsurface conditions not indicated by the borings are possible. A representative of our firm should provide construction observation and materials testing services during construction.**



We believe this investigation was conducted with that level of skill and care normally used by geotechnical engineers practicing in this area at this time. No arranty, express or implied, is made. If we can be of further service in discussing the contents of this report, or in the proposed construction from a geotechnical point of view, please call.

CTL | THOMPSON, INC.



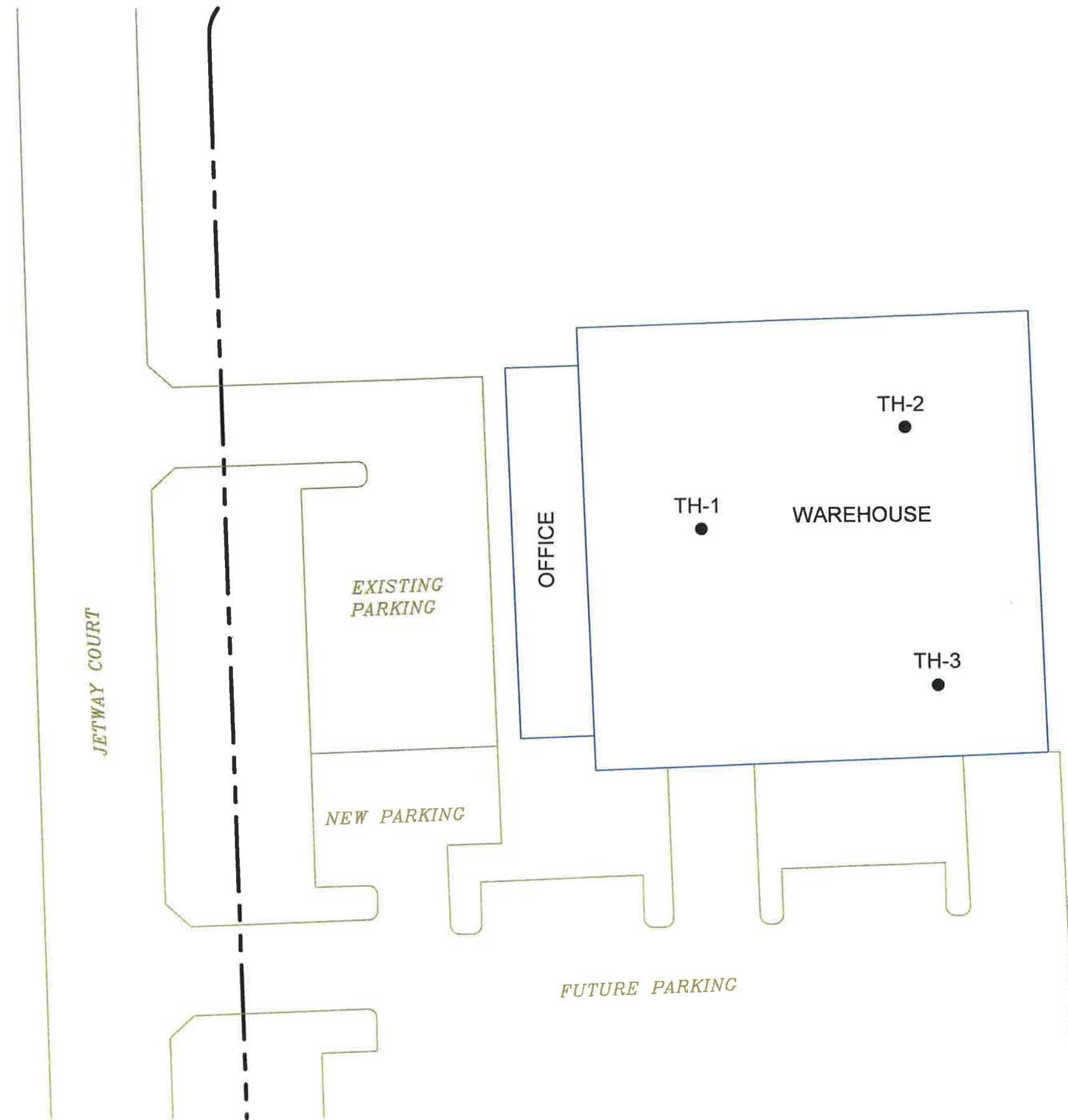
Michael N. Lemons, P.E.  
Associate Engineer

Reviewed by:

David A. Glater, P.E., C.P.G.  
Principal Geological Engineer

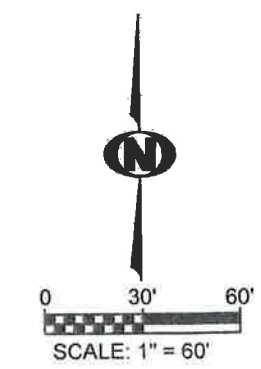
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**VICINITY MAP**  
(NO SCALE)

- LEGEND:
- TH-1 APPROXIMATE LOCATION OF EXPLORATORY BORING.
  - APPROXIMATE LOCATION OF PROPERTY BOUNDARY.
  - APPROXIMATE LOCATION OF EXISTING BUILDING.



NOTE:  
BASE DRAWING WAS PROVIDED BY HGF ARCHITECTS  
(DATED SEPTEMBER 30, 2004).

**Location of  
Exploratory  
Borings**

TH - 1

TH - 2

TH - 3



**LEGEND:**

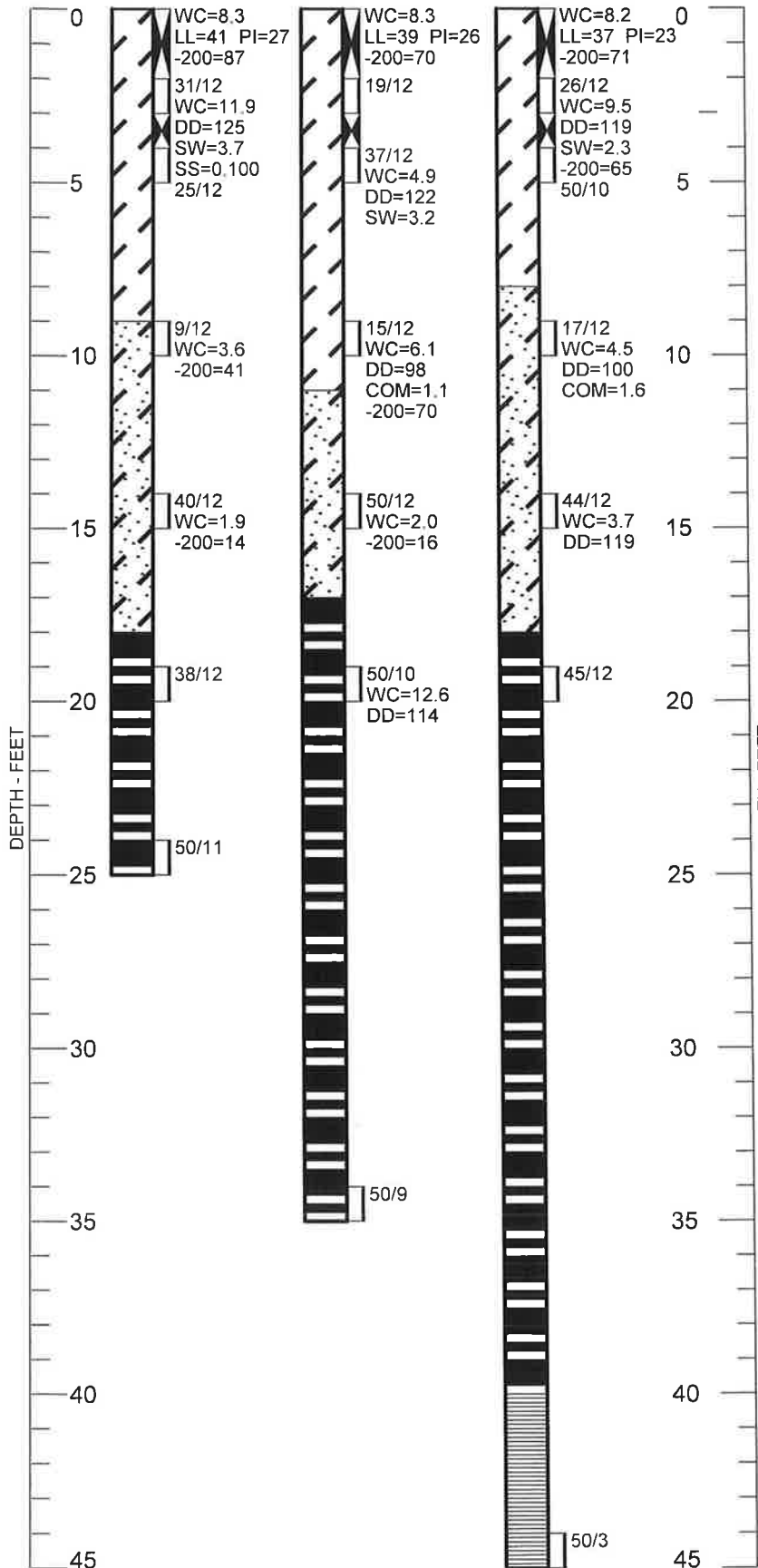
- CLAY, SLIGHTLY SANDY TO VERY SANDY, VERY STIFF, SLIGHTLY MOIST TO MOIST, LIGHT BROWN TO BROWN (CL).
- SAND, CLAYEY, GRAVELLY, MEDIUM DENSE TO VERY DENSE, SLIGHTLY MOIST TO MOIST, LIGHT BROWN TO BROWN (SC).
- CLAYSTONE, MEDIUM HARD TO HARD, SLIGHTLY MOIST TO MOIST, LIGHT BROWN TO BROWN.
- SHALE, VERY HARD, SLIGHTLY MOIST, DARK GRAY.

DRIVE SAMPLE. THE SYMBOL 26/12 INDICATES 26 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2.5-INCH O.D. SAMPLER 12 INCHES.

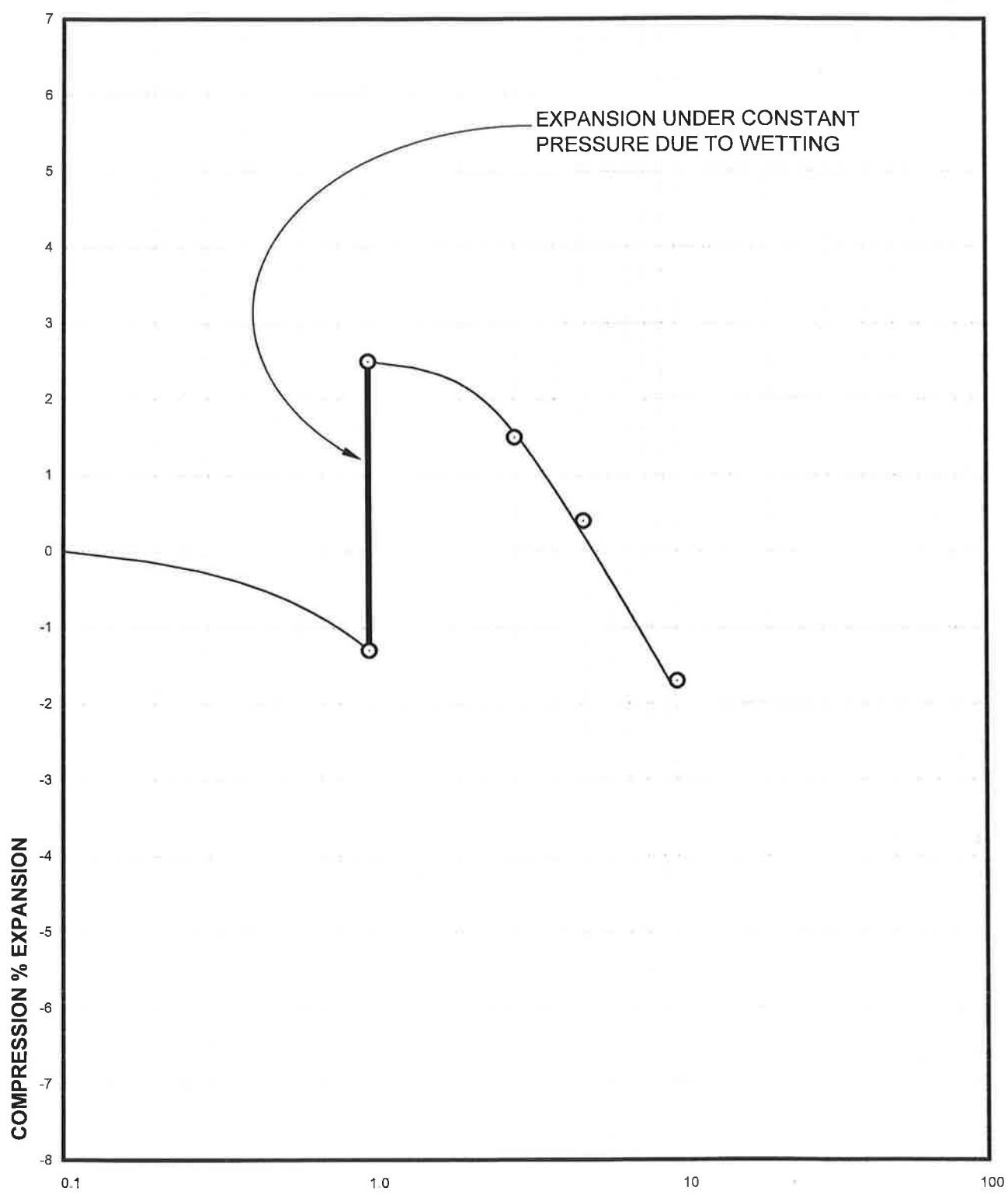
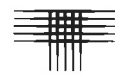
INDICATES BULK SAMPLE OBTAINED FROM AUGER CUTTINGS.

**NOTES:**

1. THE BORINGS WERE DRILLED AUGUST 26, 2011 USING A 4-INCH DIAMETER, CONTINUOUS-FLIGHT AUGER AND A CME-45, TRUCK-MOUNTED DRILL RIG.
2. THESE LOGS ARE SUBJECT TO THE EXPLANATIONS, LIMITATIONS, AND CONCLUSIONS AS CONTAINED IN THIS REPORT.
3. GROUND WATER WAS NOT ENCOUNTERED IN THE EXPLORATORY BORINGS DURING THIS INVESTIGATION.
4. WC - INDICATES MOISTURE CONTENT. (%)  
 DD - INDICATES DRY DENSITY. (PCF)  
 SW - INDICATES SWELL WHEN WETTED UNDER 1 KSF LOAD. (%)  
 COM - INDICATES COMPRESSION WHEN WETTED UNDER 1 KSF LOAD. (%)  
 LL - INDICATES LIQUID LIMIT. (%)  
 (NV : NO VALUE)  
 PI - INDICATES PLASTICITY INDEX. (%)  
 (NP : NON-PLASTIC)  
 -200 - INDICATES PASSING NO. 200 SIEVE. (%)  
 SS - INDICATES WATER-SOLUBLE SULFATE CONTENT. (%)



**Summary Logs of  
 Exploratory  
 Borings**

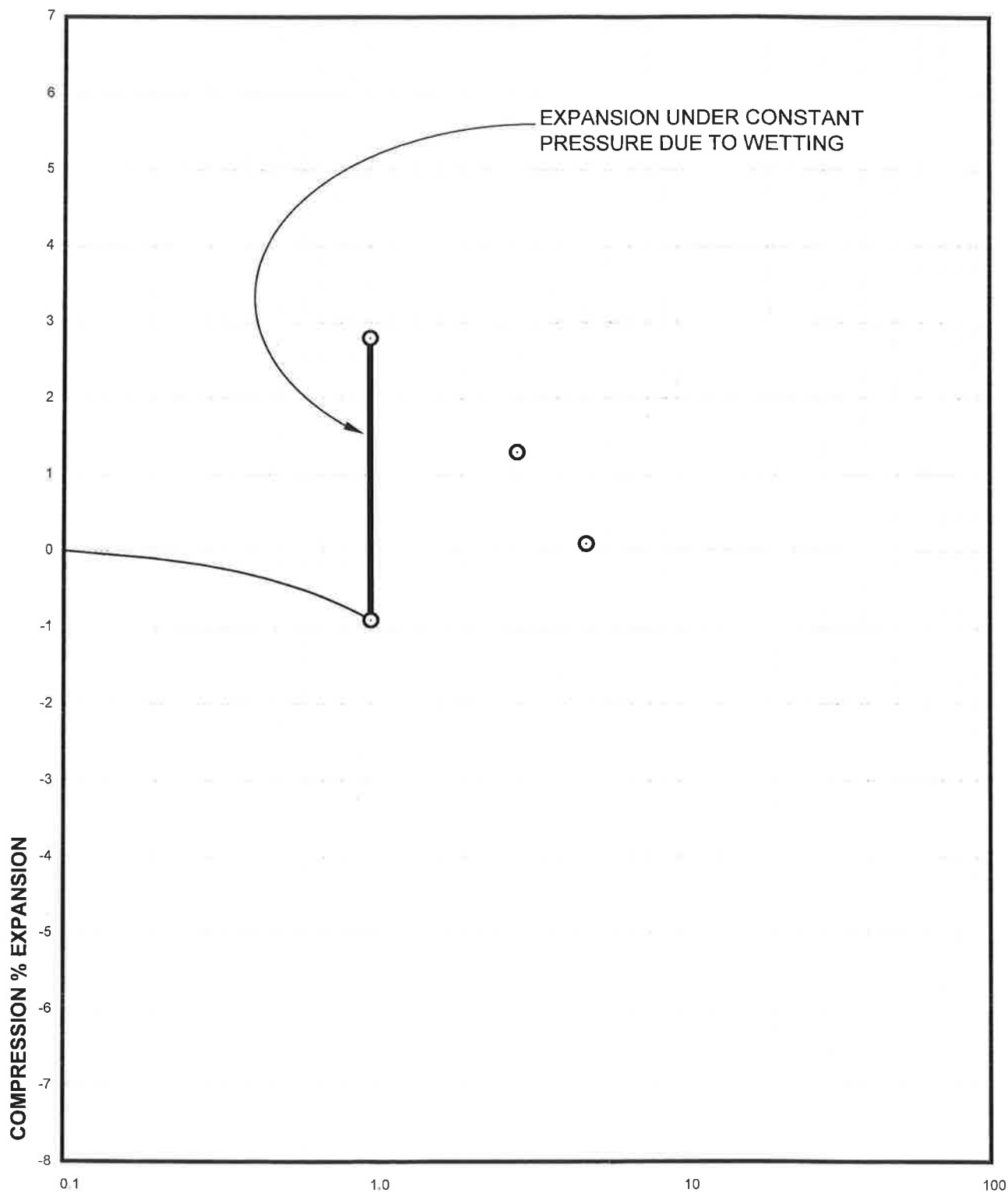
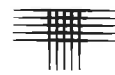


**APPLIED PRESSURE - KSF**  
Sample of CLAY, SLIGHTLY SANDY (CL)  
From TH-1 AT 2 FEET

DRY UNIT WEIGHT= 125 PCF  
MOISTURE CONTENT= 11.9 %

### Swell Consolidation Test Results

FIG. 3

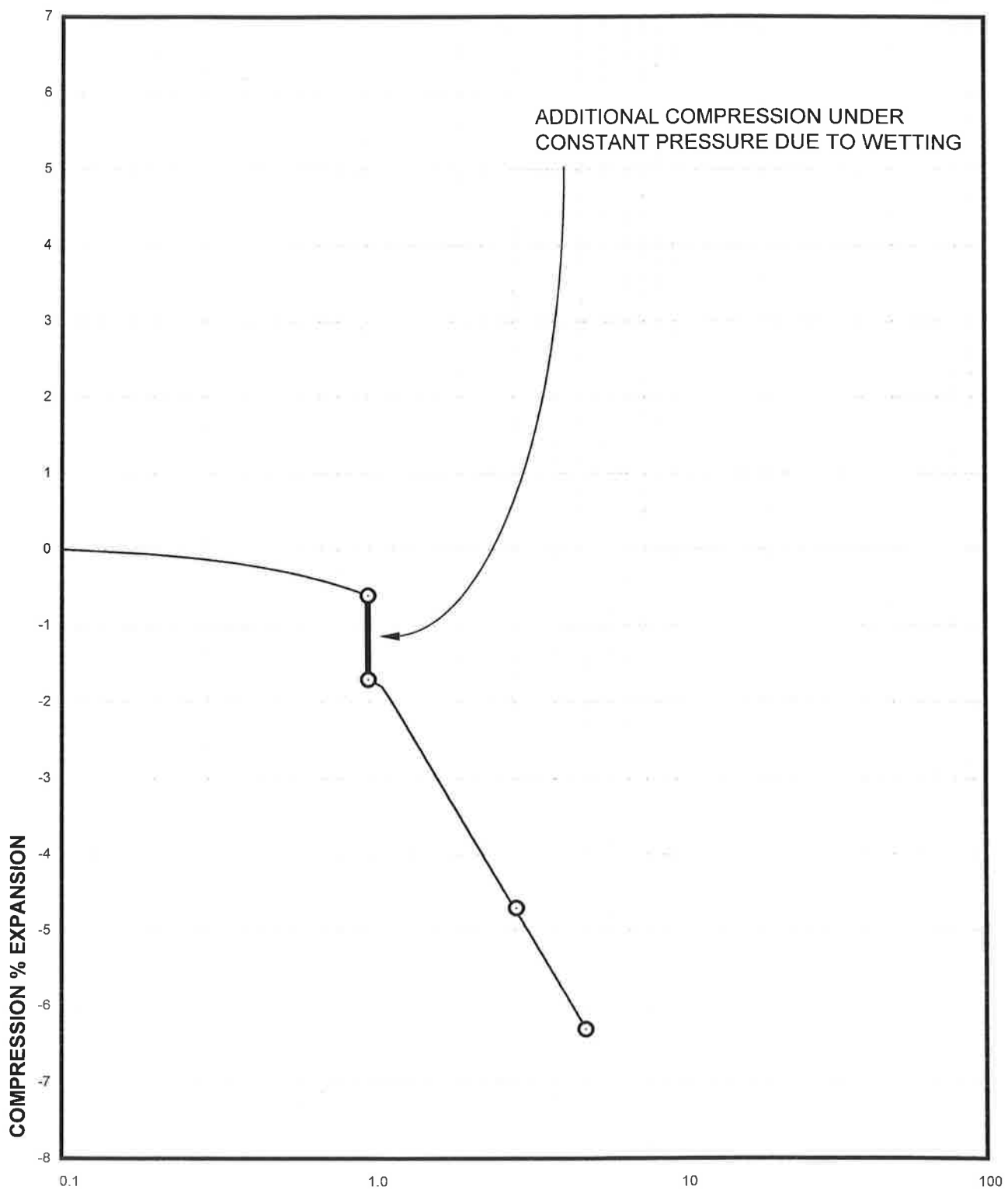
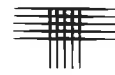


**APPLIED PRESSURE - KSF**  
Sample of CLAY, SANDY (CL)  
From TH-2 AT 4 FEET

DRY UNIT WEIGHT= 122 PCF  
MOISTURE CONTENT= 5.0 %

### Swell Consolidation Test Results

FIG. 4

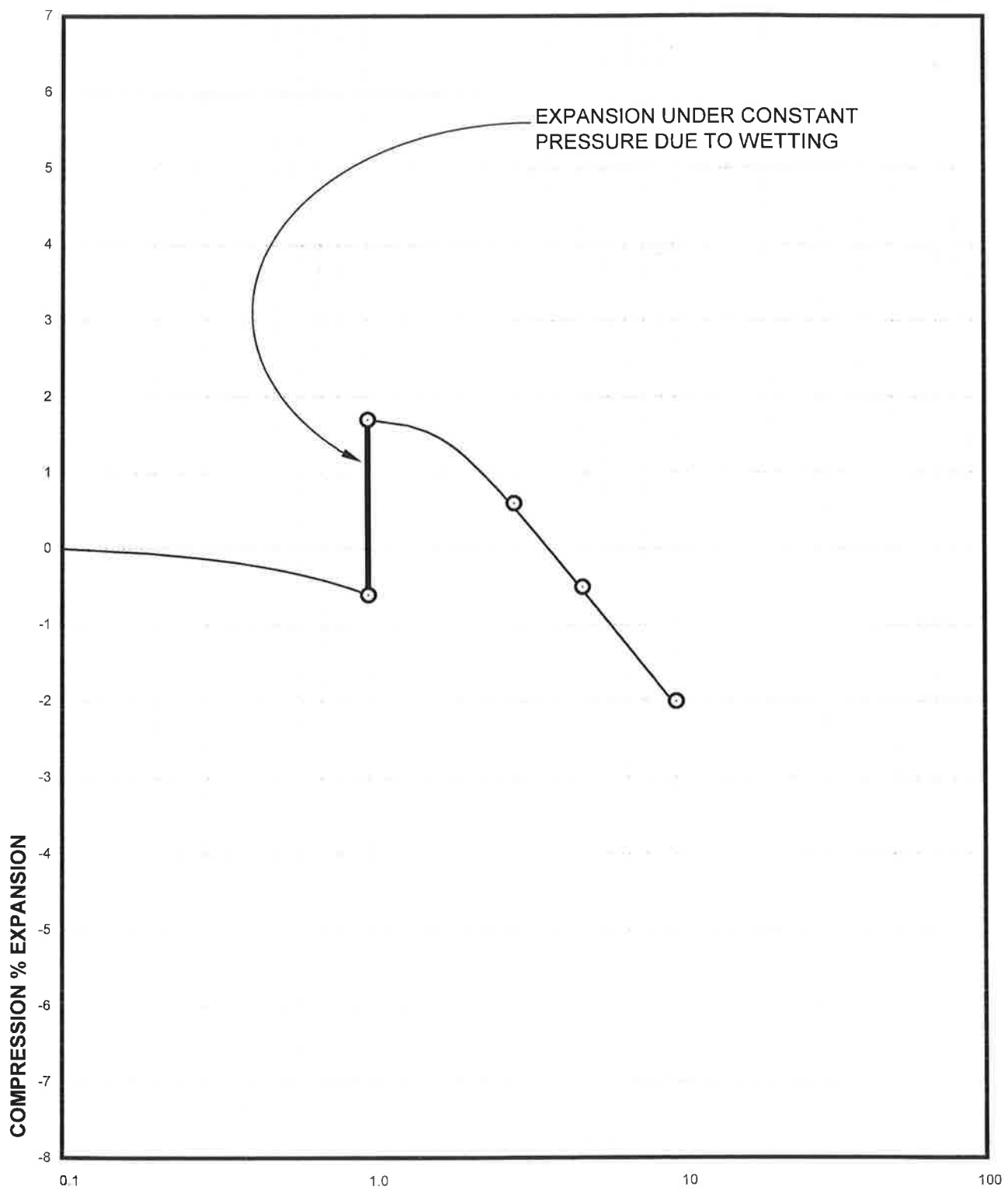
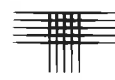


**APPLIED PRESSURE - KSF**  
Sample of CLAY, SANDY (CL)  
From TH-2 AT 9 FEET

DRY UNIT WEIGHT= 98 PCF  
MOISTURE CONTENT= 6.1 %

### Swell Consolidation Test Results

FIG. 5



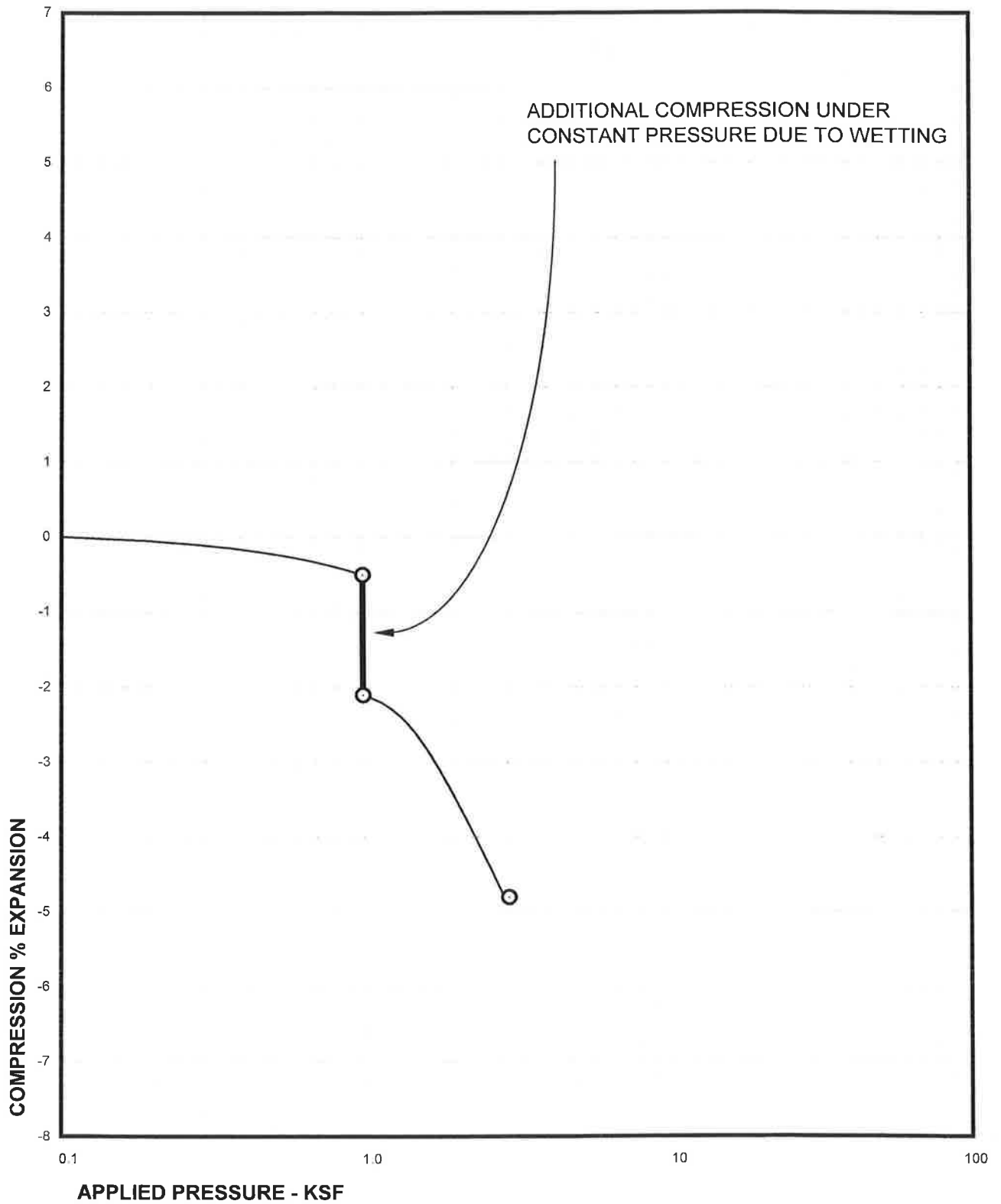
**APPLIED PRESSURE - KSF**

Sample of CLAY, SANDY (CL)  
From TH-3 AT 2 FEET

DRY UNIT WEIGHT= 119 PCF  
MOISTURE CONTENT= 9.5 %

### Swell Consolidation Test Results

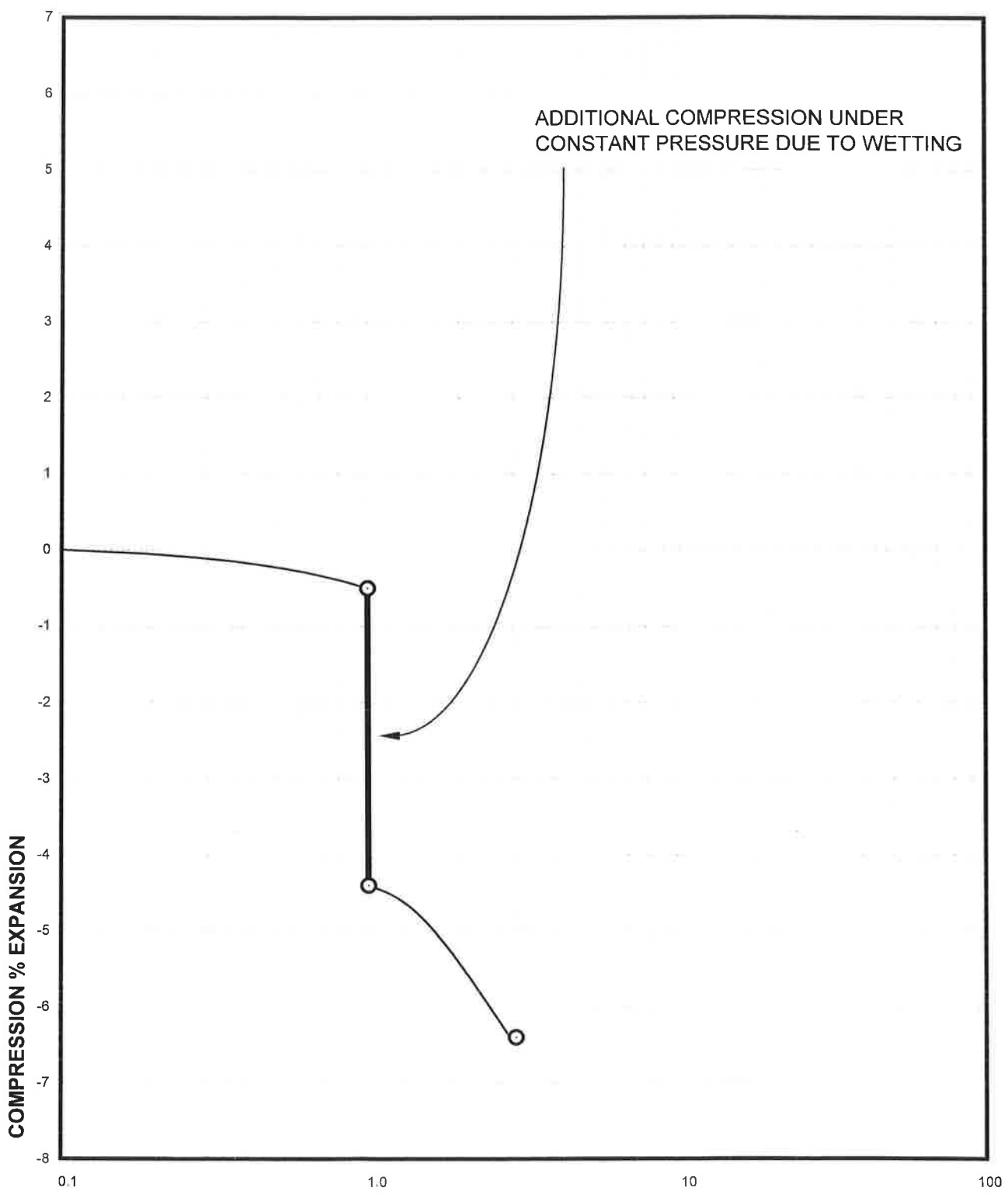
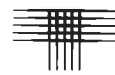
FIG. 6



Sample of SAND, CLAYEY (SC)  
From TH-3 AT 9 FEET

DRY UNIT WEIGHT= 100 PCF  
MOISTURE CONTENT= 4.5 %

### Swell Consolidation Test Results



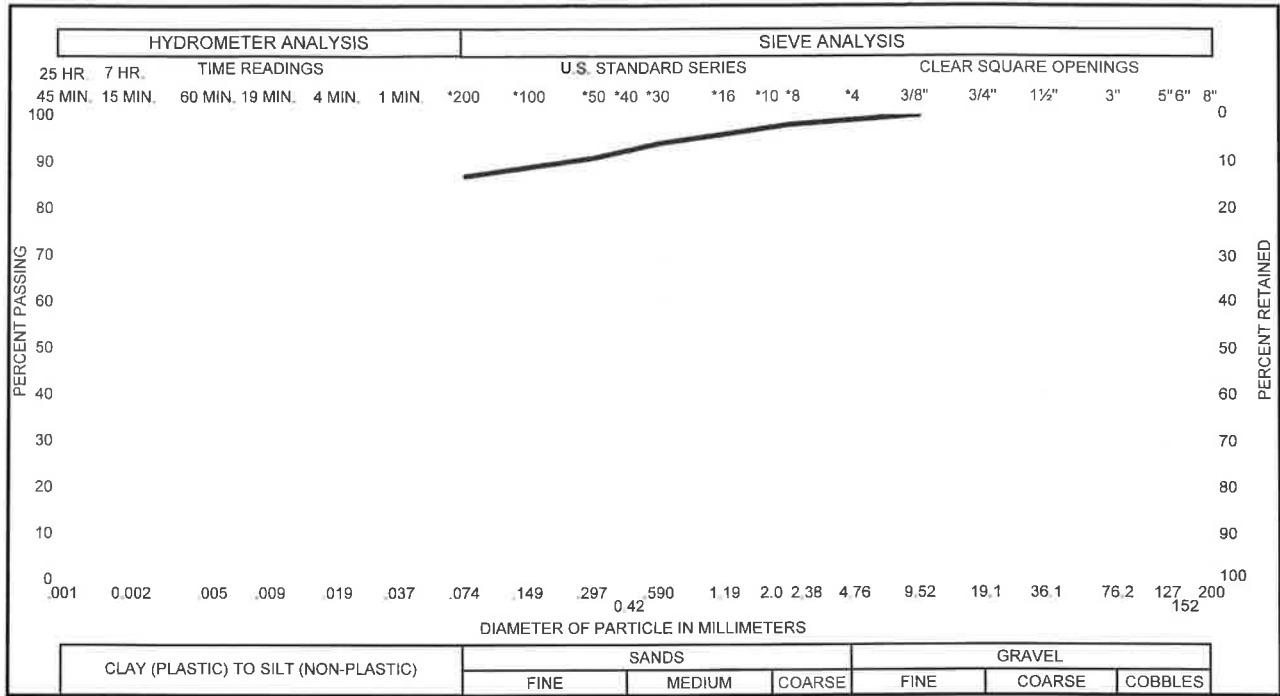
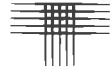
**APPLIED PRESSURE - KSF**

Sample of SAND, CLAYEY (SC)  
From TH-3 AT 14 FEET

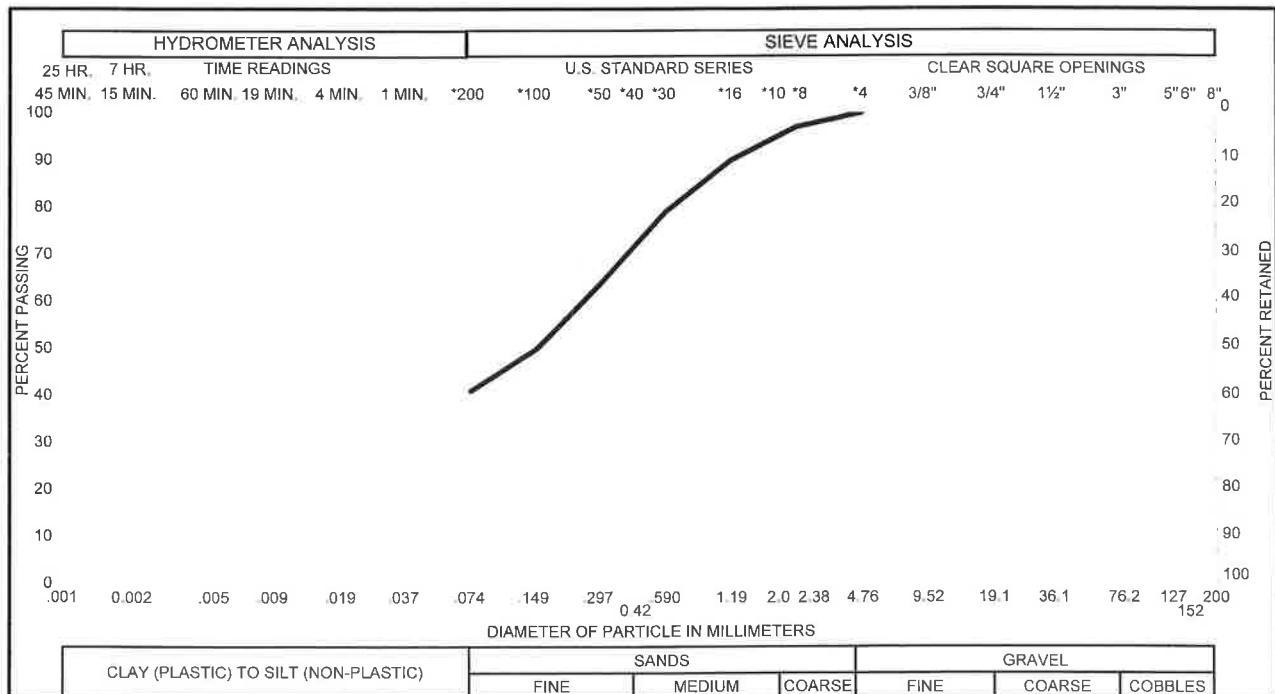
DRY UNIT WEIGHT= 119 PCF  
MOISTURE CONTENT= 3.7 %

### Swell Consolidation Test Results

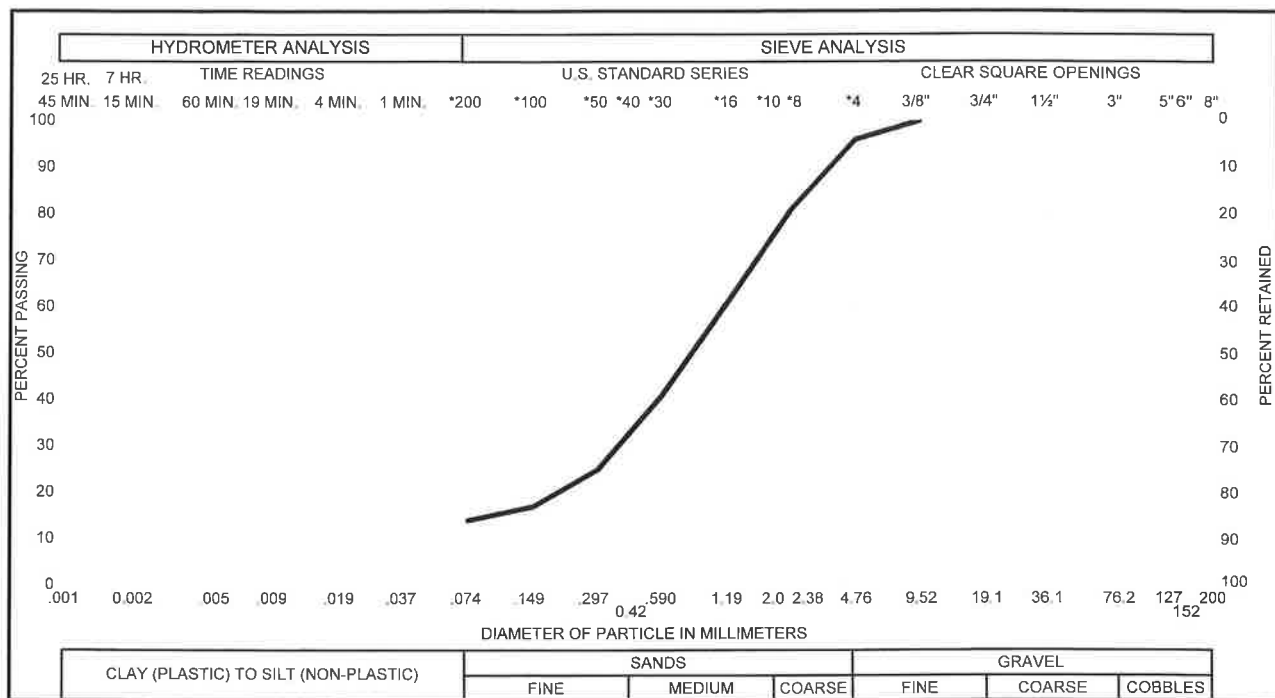
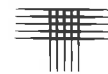
FIG. 8



Sample of CLAY, SLIGHTLY SANDY (CL) GRAVEL 1 % SAND 12 %  
 From TH - 1 AT 0-4 FEET SILT & CLAY 87 % LIQUID LIMIT 41 %  
 PLASTICITY INDEX 27 %

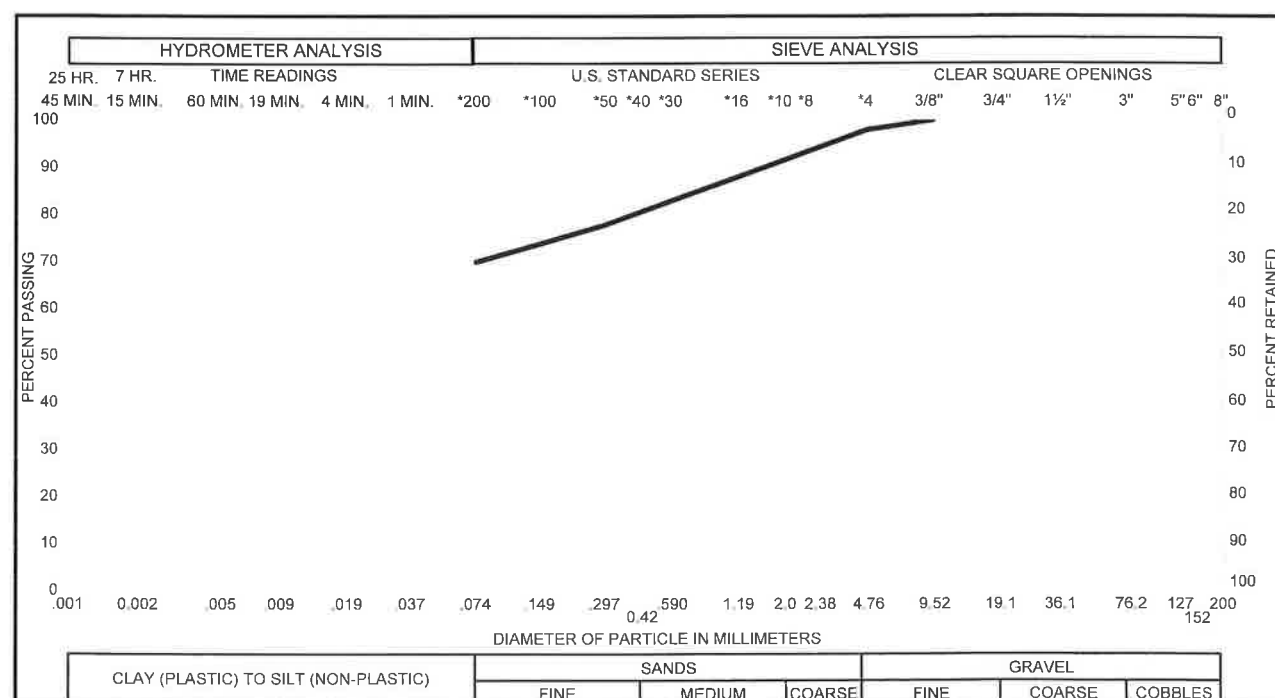


Sample of SAND, VERY CLAYEY (SC) GRAVEL 0 % SAND 59 %  
 From TH - 1 AT 9 FEET SILT & CLAY 41 % LIQUID LIMIT %  
 PLASTICITY INDEX %



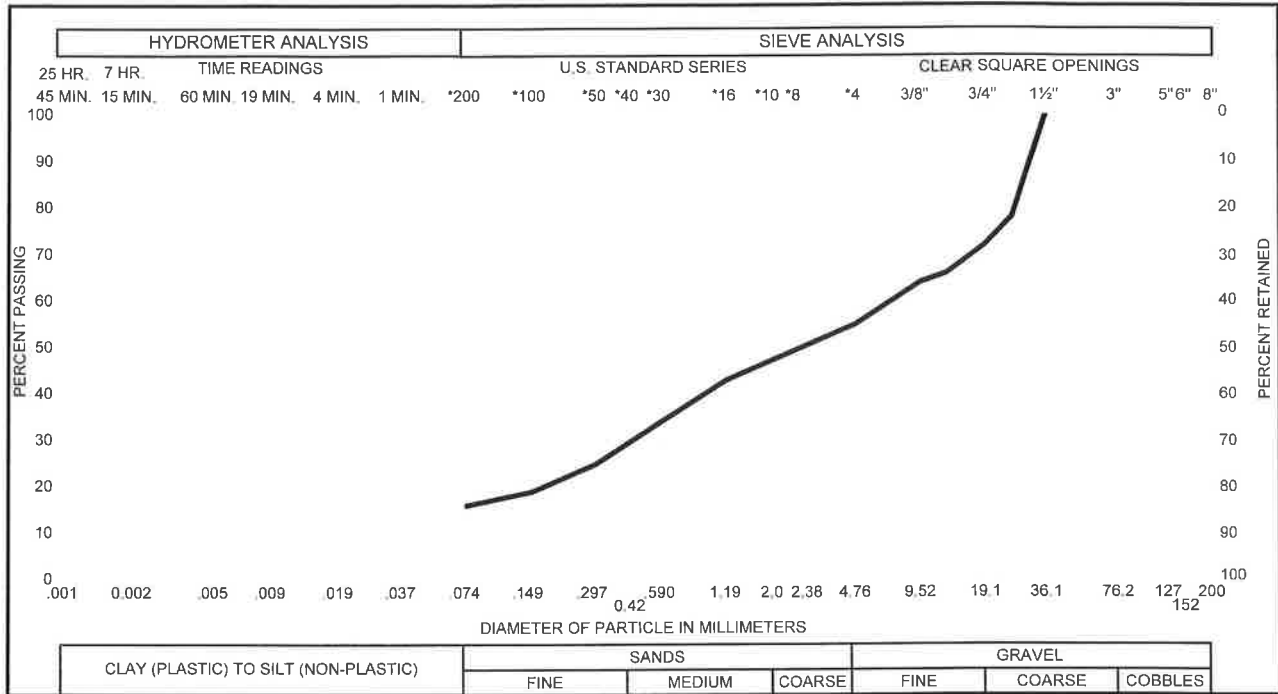
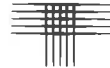
Sample of **SAND, CLAYEY (SC)** From **TH - 1 AT 14 FEET**

GRAVEL	4 %	SAND	82 %
SILT & CLAY	14 %	LIQUID LIMIT	39 %
PLASTICITY INDEX			26 %

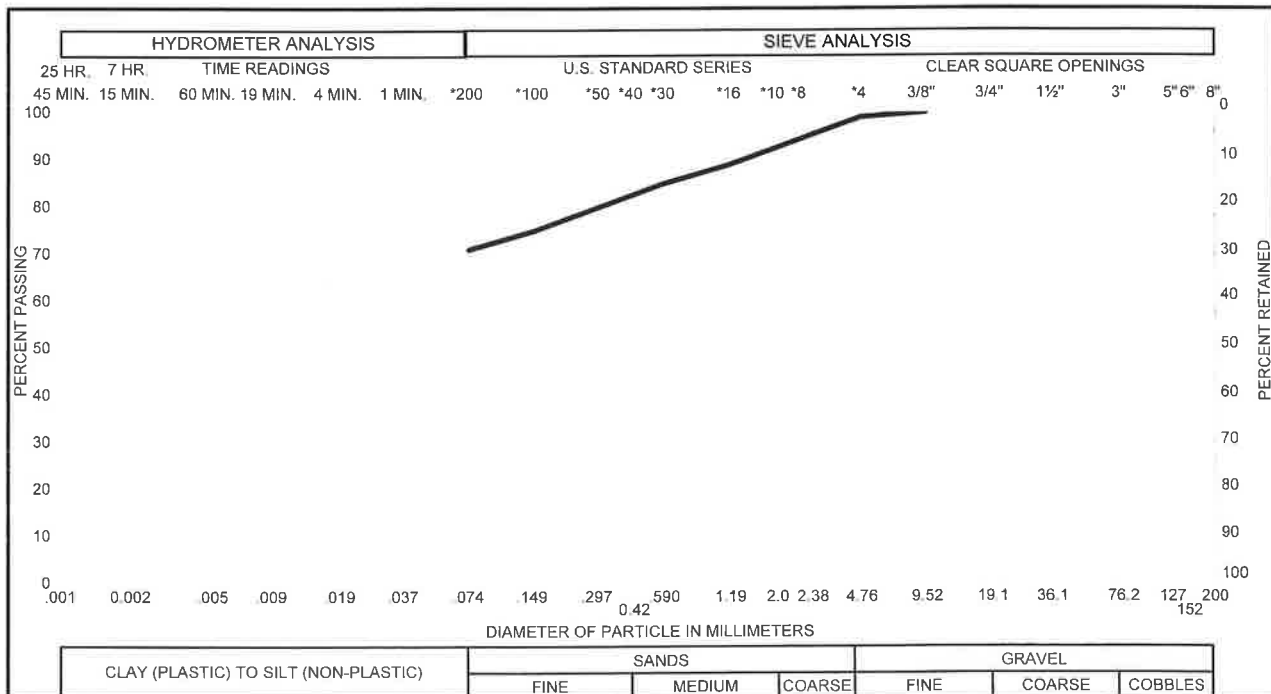


Sample of **CLAY, SANDY (CL)** From **TH - 2 AT 0-4 FEET**

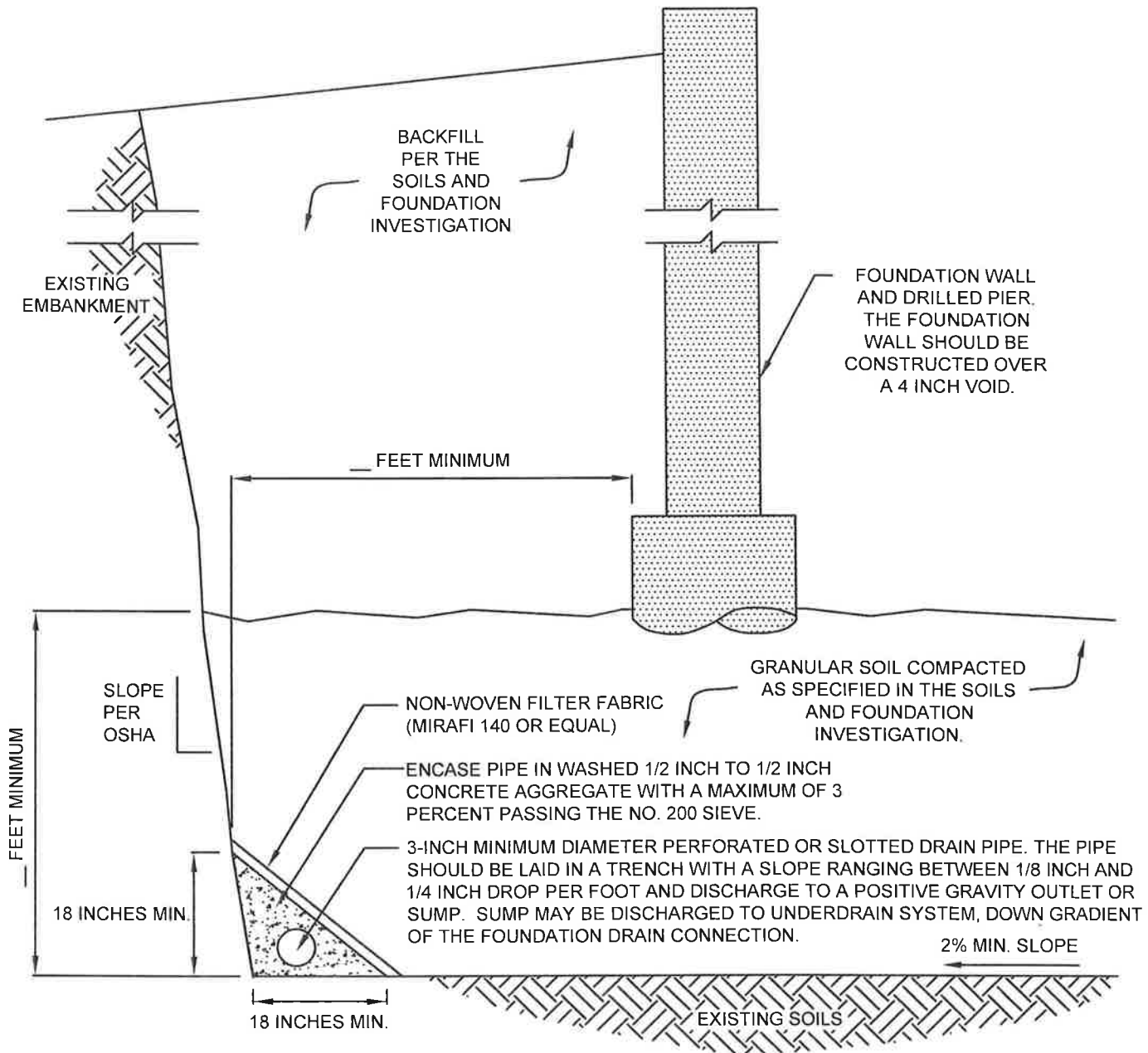
GRAVEL	2 %	SAND	28 %
SILT & CLAY	70 %	LIQUID LIMIT	%
PLASTICITY INDEX			%



Sample of **SAND, CLAYEY, GRAVELLY (SC)** GRAVEL **45 %** SAND **39 %**  
 From **TH - 2 AT 14 FEET** SILT & CLAY **16 %** LIQUID LIMIT **%**  
 PLASTICITY INDEX **%**



Sample of **CLAY, SLIGHTLY SANDY (CL)** GRAVEL **1 %** SAND **28 %**  
 From **TH - 3 AT 0-4 FEET** SILT & CLAY **71 %** LIQUID LIMIT **37 %**  
 PLASTICITY INDEX **23 %**



NOTES:

1. THE BELOW-FILL DRAIN IS APPLICABLE TO ALL FLOOR SYSTEMS. FLOOR SYSTEMS SHOULD BE DESIGNED AND CONSTRUCTED PER THE RECOMMENDATIONS PROVIDED IN THE SOILS AND FOUNDATION INVESTIGATION.
2. PERIMETER DRAINS AT THE FOUNDATION LEVEL SHOULD BE CONSTRUCTED PER THE DETAILS PROVIDED IN THE SOILS AND FOUNDATION INVESTIGATION.

