KOECHLEIN CONSULTING ENGINEERS, INC.
CONSULTING GEOTECHNICAL AND MATERIALS ENGINEERS

GEOTECHNICAL REPORT
BEAR CLAW III DEVELOPMENT
2420 SKI TRAIL LANE
STEAMBOAT SPRINGS, COLORADO

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SCOPE

This report presents the results of a geotechnical report for the proposed Bear Claw III development to be located at 2420 Ski Trail Lane in Steamboat Springs, Colorado. The approximate site location is shown on the Vicinity Map, Fig. 1. The purpose of this report was to provide geotechnical recommendations for the proposed development based on a review of our field and laboratory data obtained in May 2000 and the current proposed development plans.

This report includes descriptions of subsurface soil, bedrock and ground water conditions encountered in the exploratory borings in May 2000, the results of our laboratory data, foundation system recommendations, and recommended design and construction criteria for the current conceptual plans. This report was prepared from data developed during our field investigation in May 2000, laboratory testing of samples obtained in our field investigation, review of previous soil investigations and experience with similar projects and subsurface conditions.

The recommendations presented in this report are based on seven duplexes and two large condominium buildings being constructed on the subject property. We should be contacted to review our recommendations when the final structural plans for the structures have been developed. A summary of our findings and conclusions is presented in the following paragraphs.

EXECUTIVE SUMMARY

1. Based on the current topographic map of the subject site, the overall site topography has not significantly changed since our field investigation in May 2000. Therefore, it is our opinion that the subsurface conditions encountered in our field investigation in May 2000 will be representative of the current subsurface conditions. However, there is a 4 foot difference in overall elevation between the current topographic survey and the survey performed in 2000. Refer to the CURRENT SURVEY VS. PREVIOUS SURVEY section of this report for details.

2. Although we believe that the borings drilled in May 2000 were spaced in such a way as to provide a good overall cross section of the existing subsurface conditions beneath the subject site, no borings were drilled along the southern portion of the subject site. In order to verify that the subsurface conditions do not change in this area, an additional investigation could be performed. Refer to the ADDITIONAL INVESTIGATION section of this report for additional details.

3. The subsurface conditions encountered in the exploratory borings in May 2000 were similar. The subsurface conditions consisted of 6 inches to 6.0 feet of topsoil or existing fill underlain by a stiff to very stiff, sandy clay to varying depths of 2.0 to 10.0 feet. The existing fill was characterized by a medium stiff, sandy clay with some cobbles. Below the natural sandy clay, to the maximum depth explored of 45.0 feet, the subsurface conditions consisted of a medium hard to very hard, sandy claystone with interbedded layers of a fine to coarse grained sandstone. Laboratory tests indicated that the sandy claystone has a low to moderate swell potential. However, previous investigations indicate that the claystone has a low to high swell potential. Refer to the SUBSURFACE CONDITIONS section for details.

4. At the time of the field investigation in May 2000, ground water was not encountered in the exploratory borings during drilling. However, ground water was measured in exploratory borings TH-1 thru TH-11, 1 to 2 days after drilling was completed. Ground water was measured at various depths of 4.9 to 27.9 feet within the exploratory borings. Based on the subsurface conditions, it is our opinion that the ground water is travelling through seams of sandstone interbedded within the claystone.

5. Existing fill to a depth of 2.0 feet was encountered at this site in May 2000. The existing fill was characterized by a medium stiff, sandy clay with cobbles and pieces of concrete. Greater depths of existing fill could be encountered throughout the site.

6. Because the claystone has a low to high swell potential, special considerations should be given to the foundation system for the proposed buildings. Refer to the FOUNDATIONS section of this report for more details.

7. Because the claystone has a low to high swell potential, special considerations should be given to the floor system for the proposed buildings. Refer to the FLOOR SYSTEMS section of this report for more details.

8. Open cuts and excavations require precautions as outlined in this report in order to maintain the stability of slopes and sides of excavations. Refer to the EXCAVATIONS section of this report for additional details on cut and fill slope recommendations.

9. Because very hard claystone and sandstone was encountered in May 2000, it is our opinion that heavy-duty excavation equipment will be necessary to complete the required excavations.

10. Drainage around the structures should be designed and constructed to provide for rapid removal of surface runoff and avoid concentration of water adjacent to foundation walls. Refer to the SURFACE DRAINAGE section of this report for additional details.

11. The pavement subgrade soils classified as A-7-6 soils, as defined by the AASHTO Classification system. Pavement designs are based on the subgrade soils having an AASHTO classification of A-7-6 soils. Pavement sections are presented in the PAVEMENT DESIGN section of this report.

SITE CONDITIONS

The proposed Bear Claw III development will be located at 2420 Ski Trail Lane in

Steamboat Springs, Colorado. Existing Bear Claw condominiums are located immediately to the northeast of the proposed development. A ski lift borders the north side of the site while a gully borders the south side of the site. At the time of the preparation of this report in 2007, snow covered the site. However, several fill piles consisting of concrete pieces were observed on the site in May 2000. The portion of the site adjacent to the existing building is relatively flat with a slight slope down towards the south while the remainder of the site slopes down towards the southwest at an approximate grade of 10 to 30 percent. Vegetation on the site consists of bushes, grasses, and weeds.

PROPOSED CONSTRUCTION

We understand that the proposed development of the subject site has changed considerably since our original soils and foundation investigation in 2000. The current plans consist of the design and construction of seven duplexes and two large condominium buildings. The seven duplexes will be located southeast of the two large condominium buildings. The seven duplexes will most likely be two-stories in height with a walkout lower level. We anticipate that the duplexes (Buildings 1C, 2C, 3D, 4D, 5D, 6E, and 7E) will be of cast-in-place concrete and wood frame construction with slab-on-grade floors. Excavations between 3 to 45 feet may be necessary for the construction of the proposed duplexes. Maximum foundation loads for the proposed duplexes were assumed to be those normally associated with residential structures.

Based on the current plans, two large condominium buildings (Buildings A and B) will be constructed southwest of the existing Bear Claw Condominium building. The large condominium buildings will most likely be eight-stories in height with two levels of below grade parking. We anticipate that the condominium buildings will be of cast-in-place concrete (or pre-cast panel), structural steel and masonry construction with slab-on-grade floors. Excavations between 3 to 51 feet may be necessary for construction of the proposed condominium buildings. Maximum foundation loads for the proposed condominium buildings were assumed to be those normally associated with large commercial structures.

We anticipate that both flexible pavements and rigid pavements will be used at this site. Rigid pavement will most likely be used for the entrances and heavy traffic areas while the flexible pavement may be used for the remainder of the site.

PREVIOUS INVESTIGATIONS

Three previous geotechnical investigations were performed for this site in the early 1980’s, early 1990’s, and in 2000. Information from these reports was used in the compilation of this geotechnical report. The following previous investigations were reviewed prior to compilation of this report.

1. Soils and Foundation Investigation, Bear Claw III Condominiums, 2420 Ski Trail Lane, Steamboat, Colorado, July 14, 2000, Job No. 00-074, prepared by Koechlein Consulting Engineers, Inc.

2. Preliminary Geotechnical Investigation, Bear Claw III Condominiums, Section 22, Township 6 North, Range 84 West, Steamboat Springs, Colorado, dated December 31, 1981, Project Number 1-1103-5243-00, prepared by Fox Consultants, Inc.

3. Final Geotechnical Evaluation, Bear Claw III Condominiums, Steamboat Springs, Colorado, dated January 15, 1990, Job No. 1-1103-8225-00, prepared by Fox Consultants, Inc.

CURRENT SURVEY VS. PREVIOUS SURVEY

Based on the current topographic map of the subject site, the overall site topography has not significantly changed since our field investigation in May 2000. However, there is a 4 foot difference in overall elevation between the current topographic survey and the survey performed in 2000. The elevation difference between the topographic surveys does not appear to be caused by a change in the topography of the site since 2000. It has been our experience that when this occurs, the survey benchmark used in one of the surveys is not correct. Because our previous investigation was performed using the topographic survey prepared in 2000 and the elevations of the borings surveyed in 2000, the information presented in this report will be based solely on the survey prepared in 2000. We recommend that the current survey be verified to confirm that the correct benchmark has been used. If it is determined that the current topographic survey is correct, the elevations presented in this report may need to be adjusted to account for the proper elevation.

SUBSURFACE INVESTIGATION

Based on the current topographic map of the subject site, the overall site topography has not changed significantly since our field investigation in May 2000. Therefore, it is our opinion that the subsurface conditions encountered in our field investigation in May 2000 will be representative of the current subsurface conditions. However, there is an overall elevation difference between the current survey and the survey performed in 2000. Refer to the CURRENT SURVEY VS. PREVIOUS SURVEY section in this report for additional details.

Subsurface conditions were investigated at this site on May 2 thru 5, 2000 by drilling fifteen deep exploratory borings with a 4-inch diameter, continuous flight power auger, at the locations shown on the Locations of Exploratory Borings, Fig. 2. Ten pressuremeter tests were performed within five of the exploratory borings, TH-1 thru TH-5, during the investigation in May 2000. An engineer from our office was on the site to supervise the drilling of the exploratory borings and to visually classify and document the subsurface soils, bedrock and ground water conditions. A description of the subsurface soils and bedrock observed in the exploratory borings is shown on the Logs of Exploratory Borings, Figs. 3 thru 7 and on the Legend of Exploratory Borings, Fig. 8.

Representative soil and bedrock samples obtained from the exploratory borings were tested in our laboratory in order to determine their natural moisture content, dry density, Atterberg limits, gradation properties, pH properties, percent sulfate, resistivity, and swell-consolidation properties. The results of the laboratory tests are presented on the Logs of Exploratory Borings, Figs. 3 thru 7, on the Gradation Test Results, Figs. 9 thru 14, on the Swell-Consolidation Test Results, Figs. 15 thru 18, on the Moisture-Density Relationship Test, Fig. 19 and in the Summary of Laboratory Test Results, Table I.

ADDITIONAL INVESTIGATION

As previously mentioned, based on the current topographic map of the subject site, the overall site topography has not significantly changed since our field investigation in May 2000. Therefore, it is our opinion that the subsurface conditions encountered in our field investigation in May 2000 will be representative of the current subsurface conditions. In addition, based on the current site plan and the locations of the proposed buildings, it is our opinion that our borings drilled in May 2000 were spaced in such a way as to provide a good overall cross section of the existing subsurface conditions beneath the subject site.

However, no borings were drilled along the southern portion of the subject site. In particular in the area of the southern portion of Building B and Buildings 6E and 7E. It is possible that subsurface conditions could change in this area, but based on the consistency of the subsurface conditions encountered within our borings and the topography of the area, it is our opinion that the risk of the subsurface conditions changing is low. In order to verify that the subsurface conditions do not change in this area, an additional investigation could be performed. Due to the topography of this area, however, a field investigation would be difficult to perform without first cutting benches into the hillside in order to provide access for a drill rig. If the owner would like to perform an additional field investigation in this area, we can be contacted to provide this service.

SUBSURFACE CONDITIONS

The subsurface conditions encountered in the exploratory borings drilled in May 2000 were similar. The subsurface conditions consisted of 6 inches to 6.0 feet of topsoil or existing fill underlain by a stiff to very stiff, moist, sandy clay to varying depths of 2.0 to 10.0 feet. The existing fill was characterized by a dry to moist, medium stiff, sandy clay with some cobbles. Below the natural sandy clay, to the maximum depth explored of 45.0 feet, the subsurface conditions consisted of a medium hard to very hard, sandy claystone with interbedded layers of a fine to coarse grained sandstone. Laboratory tests indicated that the sandy claystone has a low to moderate swell potential. However, previous investigations prepared by others indicated that the claystone has a low to high swell potential.

At the time of the investigation in May 2000, ground water was not encountered in the exploratory borings during drilling. However, ground water was measured in exploratory borings TH-1 thru TH-11, 1 to 2 days after drilling was completed. Ground water was measured at various depths of 4.9 to 27.9 feet within the exploratory borings. Based on the subsurface conditions, it is our opinion that the ground water is travelling through seams of sandstone interbedded within the claystone.

RADON

In recent years, radon gas has become a concern. Radon gas is a colorless, odorless gas that is produced by the decay of minerals in soil and rock. The potential for radon gas in the subsurface strata of mountain terrain is likely. Because we anticipate that the proposed buildings will be constructed with below grade areas, we suggest that the buildings be designed with ventilation for all below grade areas.

MOLD

Mold has become a concern with new construction. Mold tends to develop in dark or damp areas such as below grade areas, under floor spaces, or bathrooms. Recommendations for the prevention, remediation, and/or mitigation of mold is outside the scope of this report. We recommend that the owner contact a Professional Industrial Hygienist for recommendations for the prevention, remediation, and/or mitigation of mold.

GROUND WATER

At the time of the field investigation in May 2000, ground water was not encountered in the exploratory borings during drilling. However, ground water was measured in exploratory borings TH-1 thru TH-11, 1 to 2 days after drilling was completed. Ground water was measured at various depths of 4.9 to 27.9 feet within the exploratory borings. No ground water was measured in borings TH-12 thru TH-15 after drilling. Based on the subsurface conditions, it is our opinion that the ground water is travelling through seams of sandstone interbedded within the claystone. Because, excavations up to 51 feet may be required for construction of the proposed buildings, we anticipate that ground water will be encountered within the proposed excavations. Refer to the DEWATERING section of this report for details on controlling ground water within the excavations.

PRESSUREMETER TESTS

In addition to standard California drive samples, ten pressuremeter tests were performed in five exploratory borings, TH-1 thru TH-5, to determine the strength of the bedrock. The pressuremeter tests were performed in May 2000 at shallow depths of 11.5 to 16.0 feet, and at deeper depths of 29.8 to 33.0 feet, to determine the strength of the bedrock at potential spread footing and drilled pier foundation depths. In general, most of the pressuremeter tests were performed within the claystone. However, some of the tests may have been performed with portions of the probe within a sandstone lens. For a complete description of the test and test results, refer to the Pressuremeter Tests, Appendix A.

The results of the pressuremeter tests were used to calculate the strength properties of the bedrock. Based on the pressuremeter tests, the allowable bearing pressure for spread footings and the end bearing and skin friction for drilled piers at this site were increased by 43 percent. Our specific foundation design and construction considerations are presented in the FOUNDATION section of this report.

EXCAVATIONS

We anticipate that excavations of up to 51 feet may be required for construction of the proposed buildings. Because very hard claystone and sandstone was encountered in the exploratory borings, it is our opinion that heavy-duty excavation equipment will be necessary to complete the required excavations. Based on the condition of the bedrock encountered during our field investigation, we anticipate that the rock may be excavated by ripping with heavy-duty equipment. However, it is possible that isolated areas of very hard bedrock may be encountered during excavating, which may require blasting. If blasting is required, the owner and design team needs to evaluate the potential damage to adjacent buildings.

Care needs to be exercised during construction so that the excavation slopes remain stable. The near surface soil, which consisted of an existing clay fill, sandy clay or weathered claystone classifies as Type B soils in accordance with OSHA regulations. The non-weathered claystone and sandstone classifies as Type A soils in accordance with OSHA regulations. OSHA regulations should be followed in all excavations and cuts.

Utilities

We anticipate that utilities will be constructed in the existing clay fill, sandy clay, claystone, and sandstone. The above paragraphs present excavation conditions that may be encountered during construction of utilities for the proposed project. A resistivity test and pH test was performed on a bulk sample obtained during our field investigation in May 2000, in order to determine corrosivity of the soils. The tests indicated that the soils have a pH of 8.3 and a resistivity of 4008 Ohm-cm. Based on the test results, the on-site soils are moderately corrosive.

Permanent Cut Slopes

We anticipate that cut slopes could be excavated in the sandy clay, claystone and sandstone. Temporary cut slopes should follow OHSA regulations as presented in the previous paragraphs. Permanent cut slopes within the sandy clay or weathered claystone could be safely excavated to 2 to 1 (Horizontal to Vertical) slopes. Permanent cut slopes within the non-weathered claystone and sandstone may be safely excavated to 2 to 1 (H to V) slopes. It may be possible to permanently excavate the non-weathered claystone and sandstone to steeper slopes, such as a 1 to 1 (H to V) or a 0.75 to 1 (H to V). However, steep slopes such as these may require shotcrete to prevent excessive sloughing from weathering of the bedrock. If the owner wishes to excavate the non-weathered bedrock to a 1 to 1 (H to V) or greater slope, a representative from our office must observe the condition of the bedrock during excavating in order to determine the bedrock competency.

Permanent Fill Slopes

Based on the proposed site plans, we do not anticipate that large permanent fill slopes will be constructed. However, if fill slopes are constructed, we anticipate that these fill slopes will be constructed with the on-site sandy clay or crushed claystone or sandstone fragments. In our opinion, fill slopes constructed to a height of 20 feet, with the on-site soils and rock, may be safely constructed at a 2 to 1 (H to V) slope.

SHORING

Due to the depth of the proposed excavations and the proximity of the existing Bear Claw Condominium Building to Building A, it may not be possible to slope all of the excavation sides as required by OSHA regulations. The ability to complete the excavation within the site constraints and the need for shoring systems, including the type of system, should be evaluated during the design phase of the project.

Because the bedrock is very hard, it may be possible to complete the excavations without shoring. However, the need to shore the bedrock will be dependent on the competency of the bedrock. A method to determine the competency of the bedrock would be to excavate a deep test pit in the area of the proposed development using a trackhoe. An engineer from our office could observe the exposed bedrock at that time and determine the competency of the rock.

If it is determined that shoring will be required, based on the subsurface conditions encountered during the investigation in May 2000, the shoring system may be designed using the following engineering soil characteristics for the natural sandy clay: $\phi' = 0°$, $\gamma = 112$ pcf, c = 10.0 psi. The claystone/sandstone will have engineering soil characteristics of: $\phi' = 0°$, $\gamma = 120$ pcf, c = 60.0 psi. We recommend a contractor specializing in shoring design and construction be contacted for design recommendations and construction of the shoring.

DEWATERING

Although ground water was not encountered within the exploratory borings during drilling in May 2000, ground water was measured in exploratory borings TH-1 thru TH-11, 1 to 2 days after drilling was completed. Ground water was measured at various depths of 4.9 to 27.9 feet within the exploratory borings. Based on the subsurface conditions, it is our opinion that the ground water is travelling through seams of sandstone interbedded within the claystone. Because ground water was measured in the exploratory borings above the proposed lowest level of the proposed buildings, we anticipate that a temporary dewatering systems will be required during construction of the buildings. In our opinion, the ground water flow within the excavations should be low and can be controlled during excavation by construction of a trench at the bottom of the excavation that drains down to a gravity outlet or a sump pit where the water can be removed by pumping.

Isolated areas of low to moderate ground water seepage through the excavation slopes may require localized stabilization of these areas. These areas may be stabilized by using a filter fabric covered with a free draining gravel, as shown in the Typical Slope

Stabilization, Fig. 20. If ground water is encountered during excavating, we should be contacted to provide specific on-site recommendations at that time.

SEISMICITY

The subsurface soil, bedrock, and ground water conditions encountered within the exploratory borings in May 2000 indicate that the soil profile classifies as a very dense soil profile. Based on this classification and the International Building Code (IBC), it is our opinion that the subject site has a seismic site classification of Site Class C.

FOUNDATIONS

We anticipate that the subsurface conditions at the foundation elevations for the proposed buildings will consist of primarily of the sandy claystone with interbedded lenses of sandstone. Laboratory test results indicated that the sandy claystone has a low to moderate swell potential. The previous investigations performed by others in 1980's and 1990's indicated that the claystone has a low to high swell potential. Although expansive claystone was encountered on the site, because it is interbedded with non-expansive sandstone seams we evaluated the use of spread footings to support the proposed structures.

Spread footings bearing on the expansive claystone could experience 1.0 to 2.5 inches of differential movement. If the owner is willing to accept the risk of foundation movement, spread footings could be used to support the proposed structures. If the owner is not willing to accept the risk of movement, the proposed structures should be supported by deep foundation systems consisting of drilled piers bearing in the claystone and sandstone bedrock. The following sections present design and construction criteria for spread footing foundation systems and drilled pier foundation systems.

Spread Footing Foundation System

If the owner is willing to accept the risk of foundation movement as outlined in the previous paragraph, spread footings may be used to support the proposed structures. We recommend that the spread footing foundation systems be designed and constructed to meet the following criteria:

1. Footings should be supported by the undisturbed claystone, sandstone, or properly moisture conditioned and compacted fill, as described below in Items 9 and 10.

2. Footings should extend below topsoil or soft surface soils and should be supported by the undisturbed claystone bedrock. On this site, we recommend that the footings be constructed at a minimum depth of 5 feet from the existing ground surface.

3. Because the bedrock has a low to moderate swell potential, we recommend that the wall and column footings be designed for a maximum allowable soil bearing pressure of 8,000 psf with a minimum dead load of 1,500 psf. Interrupted spread footings may be necessary to achieve the necessary dead load.

4. Excavation for foundations adjacent to existing structures should be performed with care. The excavations should be made so that existing

foundations and floor slabs are not undermined. Excavations adjacent to existing structures should be excavated at a 1 to 1 slope (Horizontal to Vertical) from the existing foundations.

5. Foundations should be designed to span a distance of at least 10.0 feet in order to account for anomalies in the bedrock or fill.

6. Foundation wall backfill should not be considered for support of load bearing footings. Footings should be stepped and supported by undisturbed bedrock and should not be constructed on foundation wall backfill. Foundation walls or grade beams should be designed to span across an excavation backfill zone and should not be constructed with footings within this zone.

7. The base of the exterior footings should be established at a minimum depth below the exterior ground surface, as required by the local building code. We believe that the depth for frost protection in the local building code in this area is 4 feet.

8. Column footings should have a minimum dimension of 24 inches square and continuous wall footings should have a minimum width of 16 inches. Footing widths may be greater to accommodate structural design loads.

9. Pockets or layers of soft soils, fill, and/or bedrock may be encountered in the bottom of the completed footing excavations. These materials should be removed to expose the undisturbed bedrock. The foundations should be constructed on the natural bedrock or compacted fill. Refer to the COMPACTED FILL section of this report for backfill requirements.

10. No more than 1 foot of fill should be placed and compacted below the proposed spread footings. If greater amounts of fill are necessary, we recommend that the excavations be filled with a lean concrete. Fill should be placed and compacted as outlined in the COMPACTED FILL section of this report. We recommend that a representative of our office observe and test the placement and compaction of structural fill used in foundation construction. It has been our experience that without engineering quality control, inappropriate construction

techniques occur which result in unsatisfactory foundation performance.

11. A representative from our office must observe the completed foundation excavations. Variations from the conditions described in this report, which were not indicated by our borings, can occur. The representative can observe the excavation to evaluate the exposed subsurface conditions and make the necessary recommendations.

Drilled Pier Foundation System

If the owner is not willing to accept the risk of foundation movement, it is our opinion that the proposed structures should be constructed on a drilled pier foundation system bearing within the claystone with sandstone seams. We recommend that the drilled pier foundation be designed and constructed to meet the following criteria:

1. Drilled piers should extend into the claystone/sandstone bedrock. The claystone/sandstone bedrock was encountered at various depths of 2.0 to 10.0 feet in the exploratory borings.

2. Very hard sandstone layers of varying thickness were encountered within the exploratory borings. Because these layers are very hard, special drilling equipment may be required.

3. Based on results of the pressuremeter test, it is our opinion that piers may be designed for a maximum allowable end bearing pressure of 50,000 psf and an allowable skin friction value of 5,000 psf for the portion of the pier in bedrock. The top 2.0 feet of the bedrock may be disturbed and should not be included in the design calculations.

4. Due to the expansive potential of the existing fill, we recommend that the drilled piers be designed with a minimum dead load of:

$$D_{L(min)} = 35d$$

Where: $D_{L(min)}$ = minimum dead load in kips, and
d = the diameter of the pier in feet

Load factors or a factor of safety should not be applied to the minimum dead load calculated from the preceding equation. Dead load from the buildings may be used to resist the uplift force. In addition, a skin friction of 5,000 psf can also be used to resist the uplift force for the portion of the piers in bedrock.

5. Because the claystone bedrock has a low to high swell potential, the piers should penetrate at least 8 feet into the unweathered zone of the claystone bedrock or be a minimum length of 18 feet.

6. There should be a 6-inch continuous void beneath all grade beams or foundation walls between the piers to concentrate the dead load of the structure and reduce the risk of uplift forces, if any occur on the grade beams.

7. If LPILE is used to design the drilled piers, the following table presents criteria, which may be used as input.

Type	Description	γ (pci)	κ (pci)	ϕ (°)	c (psi)	ε_{50} (%)
1	CLAY, Sandy	0.065	400	0	10.0	-
2	CLAYSTONE	0.069	1,500	0	60.0	0.005

8. The horizontal subgrade reaction k_h depends on the type of soil and rock. For the sandy clays, the horizontal subgrade reaction k_h may be taken as 85 kcf. For the claystone bedrock, the horizontal subgrade reaction k_h may be taken as 500 kcf.

9. All pier reinforcement should be designed and specified by the structural engineer. Reinforcement should extend into the grade beams or foundation walls.

10. Piers should be spaced center to center a distance of at least 3 pier diameters. Piers closer than 3 pier diameters should be designed as a

group. Special installation techniques will be required for piers spaced closer than 3 pier diameters.

11. Ground water was measured in exploratory borings TH-1 thru TH-11, 1 to 2 days after drilling. Ground water was measured at various depths of 4.9 to 27.9 feet within the exploratory borings. Although the amount of ground water was not significant immediately after drilling, we anticipate that casing may be required for installation of piers. If casing is not used, it may be necessary to pump concrete into the drilled piers to displace any water. Concrete should not be poured if more than 3 inches of water is present within the pier holes, unless concrete is pumped and the water is displaced.

12. A representative from our office should be on-site to observe the installation of the drilled piers. Our representative will be able to observe the conditions exposed by the installation of the drilled piers, to check the pier construction procedures for proper cleaning of the pier holes and to observe and test concrete placement.

FLOOR SYSTEMS

The near surface material at the anticipated floor elevations consisted of expansive claystone. Because the claystone has a low to high swell potential, we anticipate that slabs-on-grade constructed on the existing claystone could experience up to 2 inches of movement. If the owner is not willing to accept the risk of slab movement, we recommend that the floor systems be constructed as a structural floor with at least 12 inches of air space beneath the floor.

If the owner is willing to accept the risk of slab movement but would like to reduce the risk of movement to a lower level, slabs-on-grade may constructed on 3.0 feet of properly moisture conditioned and compacted non-expansive structural fill.

We recommend the following precautions for the construction of slab-on-grade floors. These precautions will not prevent floor slab movement; however, they tend to reduce damage, if movement occurs.

1. Slabs could be placed on 3.0 feet of properly moisture conditioned and compacted, non-expansive structural fill.

2. Slabs should be separated from exterior walls and interior bearing members. Vertical movement of the slabs should not be restricted.

3. Slab-bearing partitions should be minimized. Where such partitions are necessary, a slip joint should be constructed to allow free vertical movement of the partitions

4. Exterior slabs should be separated from the buildings. These slabs should be reinforced to function as independent units. Movement of these slabs should not be transmitted directly to the foundations or walls of the structures.

5. Underslab plumbing should be eliminated where feasible. Where such plumbing is unavoidable it should be thoroughly pressure tested during construction. Plumbing and utilities, which pass through the slabs, should be isolated from the slabs.

6. Heating and air conditioning systems supported by slabs should be provided with flexible connections so that slab movement is not transmitted to duct work.

7. Frequent control joints should be provided in all slabs to reduce problems associated with shrinkage.

8. Fill beneath slabs-on-grades may consist of on-site soil, crushed sandstone bedrock or approved fill. Fill should be placed and compacted as recommended in the COMPACTED FILL section of this report. Placement and compaction of fill beneath slabs should be observed and tested by a representative of our office.

FOUNDATION DRAINAGE

Surface water tends to flow through relatively permeable backfill typically found adjacent to foundations. The water that flows through the fill collects on the surface of relatively impermeable soils occurring at the foundation elevation. Both this surface water and possible ground water can cause wet or moist below grade conditions after construction.

Since we anticipate that below grade levels will be constructed for the proposed buildings, we recommend the installation of an exterior drain along the below grade foundation walls. The drain should consist of a 4-inch diameter perforated pipe encased in free draining gravel. The drain should be sloped so that water flows to a sump where the water can be removed by pumping or to a positive gravity outlet. Recommended details for typical foundation wall drains are presented in the Typical Spread Footing Wall Drain Detail, Fig. 21; and Typical Drilled Pier Wall Drain Detail, Fig. 22.

UNDERSLAB DRAINAGE

Because the proposed structures will be constructed with below grade areas and water was encountered in the exploratory borings, it may be necessary to design the proposed buildings with underslab drains. We recommend that the need for underslab drains be evaluated during construction of the proposed buildings. During construction, the amount of ground water being controlled will help to evaluate the need of an underslab drain system for the proposed buildings.

The underslab drains may consist of 4 to 8-inch diameter, perforated PVC and solid PVC pipe embedded within gravel. The 4-inch diameter perforated PVC pipe is used as laterals to collect the water into either 6 or 8-inch diameter solid PVC pipe. The solid pipes are then daylighted. The top of the pipes should be at least 4-inches below the proposed floor slabs and should be laid on a slope ranging between 1/8 to 1/4-inch drop per foot of drain. The pipes should be encased in a washed gravel. The free draining gravel used for the foundation drain may be used for the underslab drain system.

LATERAL WALL LOADS

Below grade walls are planned which must resist lateral earth pressures. Lateral earth pressures depend on the type of backfill and the height and type of wall. Walls, which are free to rotate sufficiently to mobilize the strength of the backfill, should be designed to resist the "active" earth pressure condition. Walls that are restrained should be designed to resist the "at rest" earth pressure condition. Basement walls are typically restrained. The following table presents the lateral wall pressures that may be assumed for design.

Earth Pressure Condition	Equivalent Fluid Pressure[1] (pcf)
Active	40
At-rest	55
Passive	300

Notes:
1. Equivalent fluid pressures are for a horizontal backfill condition with no hydrostatic pressures or live loads.
2. A coefficient of friction of 0.3 may be used at the base of spread footings to resist lateral wall loads.
3. Lateral earth pressure against walls where competent non-weathered bedrock has been encountered and excavated to a vertical slope may be taken as a uniform pressure of 250 psf. Lateral earth pressure from soil or weathered bedrock shall be taken as those values presented in the above table.

Backfill placed behind or adjacent to foundation walls and retaining walls should be placed and compacted as recommended in the COMPACTED FILL section of this report. Placement and compaction of the fill should be observed and tested by a representative of our office.

RETAINING WALLS

We understand that retaining walls will be constructed as part of the development of the subject site. Foundations for retaining walls may be designed and constructed as outlined in the FOUNDATIONS section of this report.

Lateral earth loads for retaining walls is presented in the LATERAL WALL LOADS section of this report. In order to reduce the possibility of developing hydrostatic pressures behind retaining walls, a drain should be constructed adjacent to the wall. The drain may consist of a manufactured drain system and gravel. The gravel should have a maximum size of 1.5 inches and have a maximum of 3 percent passing the No. 200 sieve. Washed concrete aggregate will be satisfactory for the drainage layer. The manufactured drain should extend from the bottom of the retaining wall to within 2 feet of subgrade elevation. The water can be drained by a perforated pipe with collection of the water at the bottom of the wall leading to a positive gravity outlet. A typical detail for a retaining wall drain is presented in the Typical Earth Retaining Wall Detail, Fig. 23.

SURFACE DRAINAGE

Reducing the wetting of structural soils below slabs-on-grade and pavements can be achieved by carefully planned and maintained surface drainage. We recommend the following precautions be observed during construction and maintained at all times during and after the construction is completed.

1. Wetting or drying of the open foundation excavations should be minimized during construction.

2. All surface water should be directed away from the top and sides of the excavations during construction.

3. The ground surface surrounding the exterior of the proposed buildings should be sloped to drain away in all directions. We recommend a slope of at least 12.0 inches in the first 10.0 feet for landscaped areas adjacent to the proposed structures.

4. Hardscape (concrete and asphalt) should be sloped to drain away from the buildings. We recommend a slope of at least 2 percent for all hardscape within 10.0 feet of the structures.

5. Backfill, especially around foundation walls, must be placed and compacted as recommended in the COMPACTED FILL section of this report.

6. Where landscaping is adjacent to the proposed buildings, roof drains should discharge at least 10.0 feet away from foundation walls with drainage directed away from the structures.

7. Surface drainage should be designed by a Professional Civil Engineer.

IRRIGATION

Sprinkler systems installed next to foundation walls or sidewalks could cause consolidation of non-expansive backfill beneath these areas or heaving of expansive claystone beneath these areas. This can result in settling or heaving of exterior steps and/or sidewalks. We recommend the following precautions be followed:

1. Do not install a sprinkler system next to foundation walls. The sprinkler system should be at least 10 feet away from the structures.

2. Sprinkler heads should be pointed away from the structures or in a manner that does not allow the spray to come within 10 feet of the buildings.

3. The landscape around the sprinkler system should be sloped so that no ponding occurs at the sprinkler heads.

4. If shrubs or flowers are planted next to the structures, these plants should be hand watered.

5. Install landscaping geotextile fabrics to inhibit growth of weeds and to allow normal moisture evaporation. We do not recommend the use of a plastic membrane to inhibit the growth of weeds.

6. Control valve boxes, for automatic sprinkler systems, should be periodically checked for leaks and flooding.

COMPACTED FILL

Structural fill for this project may consist of the on-site sandy clay, sandstone fragments, or approved imported fill. A standard Proctor (ASTM D-698) was performed on a sample of the crushed sandstone. The results of the Proctor test are presented in the Moisture-Density Relationship Test, Fig. 19. Non-structural fill may consist of claystone fragments. The imported fill should consist of non-expansive silty or clayey sands with at least 30 percent passing the No. 200 sieve and a maximum plasticity index of 10. We do not recommend that the claystone bedrock be used as structural fill for this project. No gravel or cobbles larger than 6.0 inches should be placed in fill areas. Fill areas should be stripped of all vegetation, existing fill, and loose soils, and then scarified, moisture treated, and compacted. Fill should be placed in thin loose lifts; moisture treated, and compacted as shown in the following table. The recommended compaction varies for the given use of the fill, as indicated in the following table.

Use of Fill	Recommended Compaction	
	Percentage of the Standard Proctor Maximum Dry Density (ASTM D-698)	Percentage of the Modified Proctor Maximum Dry Density (ASTM D-1557)
Below Structure Foundations	98	95
Below Slabs-on-Grade	95	90
Pavement Subgrade	100 (AASHTO T-99)	95 (AASHTO T-180)
Utility Trench Backfill	95	90
Backfill (Non-Structural)	90	90
Notes: 1. For clay soils the moisture content should be 0 to +3 percent of the optimum moisture content. 2. For granular soils the moisture content should be –2 to +2 of the optimum moisture content.		

We recommend that a representative of our office observe and test the placement and compaction of each lift placed for structural fill. Fill below slabs-on-grade is considered structural. It has been our experience that without engineering quality control, inappropriate construction techniques can occur which result in unsatisfactory foundation and slab-on-grade performance.

PAVEMENT DESIGN

We anticipate that both flexible pavement and rigid pavement will be used at this site for the proposed access roads. We recommend that rigid pavement be used in high traffic areas such as entrances or where heavy vehicles (trash trucks, delivery trucks, etc.) turn or maneuver. Two sections are presented for the flexible pavements. The following sections present design assumptions and recommended flexible and rigid pavement sections.

Flexible Pavement Design

The design of the flexible pavement was based upon an Equivalent Daily Load Application (EDLA), laboratory test results and the Colorado Department of Transportation pavement design manual. Design calculations were based on engineering soil characteristics from soil samples encountered in the exploratory borings to a depth of 4.0 feet. Subsurface conditions encountered within the borings, are presented in the SUBSURFACE CONDITIONS section of this report. Laboratory tests indicated that the soils encountered within the exploratory borings, to a depth of 4.0 feet, classify as A-7-6 soils, as defined by the AASHTO Classification system. Pavement designs are based on the subgrade soils having an AASHTO classification of A-7-6 soils. This soil type resulted in an estimated Hveem Stabilometer R-value of 19. The R-value was estimated from the AASHTO classification of the soil. The EDLA for this project was taken as 10. Two flexible pavement designs, based on the above method, are shown below in Table A. These flexible pavement designs include one full depth asphalt pavements and one aggregate base and asphalt pavements.

Table A
Summary of Flexible Pavement Alternatives

Traffic Volume	Full-Depth Asphalt (inches)	Asphalt & Base Course (inches)
EDLA = 10	6.5	4.5 + 6.0

Notes: We anticipate that the pavements will be constructed on the expansive claystone. If the owner is not willing to accept the risk of the pavements moving, we recommend that the pavement sections be constructed on 3.0 feet of properly moisture conditioned and compacted fill.

These designs assume that the asphalt component of the pavement has a 1500 pound Marshall stability (strength coefficient of 0.4). Normally, an asphalt aggregate should be relatively impermeable to moisture and should be designed as a well graded mix. These designs also assume that the base course has a minimum R-value of 77 (strength coefficient of 0.12). A Colorado Department of Transportation Class 5 or Class 6 base course will normally meet this requirement.

Rigid Pavement Design

A rigid pavement section was designed using the same values of the EDLA and R-value as those used in the flexible pavement design. The rigid pavement design is based on the working stress of the concrete, which is assumed to be 450 psi. The Colorado Department of Transportation pavement design manual, along with the above mentioned design values, were used to determine a rigid pavement section. The rigid pavement design resulted in a minimum design section of 6.0 inches of concrete.

Pavement Construction

In our opinion, pavements constructed over the expansive claystone could experience some movement and have a shortened useful life span. Therefore, if the owner is not willing to accept any risk of movement to the pavements, we recommend that 3.0 feet of the expansive claystone be removed and replaced with a properly moisture conditioned and compacted fill. If the owner is willing to accept the risk of movement, the pavements may be constructed on the expansive claystone.

If the owner assumes the risk of movement and chooses to construct the pavements on the expansive claystone, it still may be necessary to remove isolated areas of soft soil and rock prior to paving. Where soft soils and rock are removed the resulting surface may need to be stabilized with granular material before placing and compacting fill. Prior to placing fill the subgrade should be stripped of all soft soils, the resulting surface scarified, and the soils compacted. All fill should be compacted as recommended in the COMPACTED FILL section of this report. All asphalt should be compacted to 92 percent of the maximum Theoretical Density. For a more thorough description of our pavement construction recommendations, please refer to Appendix B.

LIMITATIONS

Although the exploratory borings were located to obtain a reasonably accurate determination of foundation conditions, variations in the subsurface conditions are always possible. Any variations that exist beneath the site generally become evident during excavation for the proposed buildings and installation of the proposed foundations. A representative from our office should observe the completed excavations and installation of all foundations in order to confirm that the soils and bedrock are as indicated by the exploratory borings and to verify our foundation and floor system design and construction recommendations. The placement and compaction of fill should also be observed and/or tested.

The design criteria and subsurface data presented in this report are valid for 3 years provided that a representative from our office observes the site at that time and confirms that the site conditions are similar to the conditions presented in the SITE CONDITIONS section of this report and that the recommendations presented in this report are still applicable. We recommend that final plans and specifications for proposed construction be submitted to our office for study, prior to beginning construction, to determine compliance with the recommendations presented in this report.

Routine maintenance, such as sealing and repair of cracks annually and overlays at 5 to 10 year intervals, is necessary to achieve long-term life of a pavement system. In addition, positive drainage must be maintained at all times in order to reduce the risk of pavement failure. Prior to paving, the pavement subgrade should be observed and tested by a representative of our firm.

If we can be of further assistance in discussing the contents of this report or in analyses of the proposed project from a geotechnical viewpoint, please contact our office.

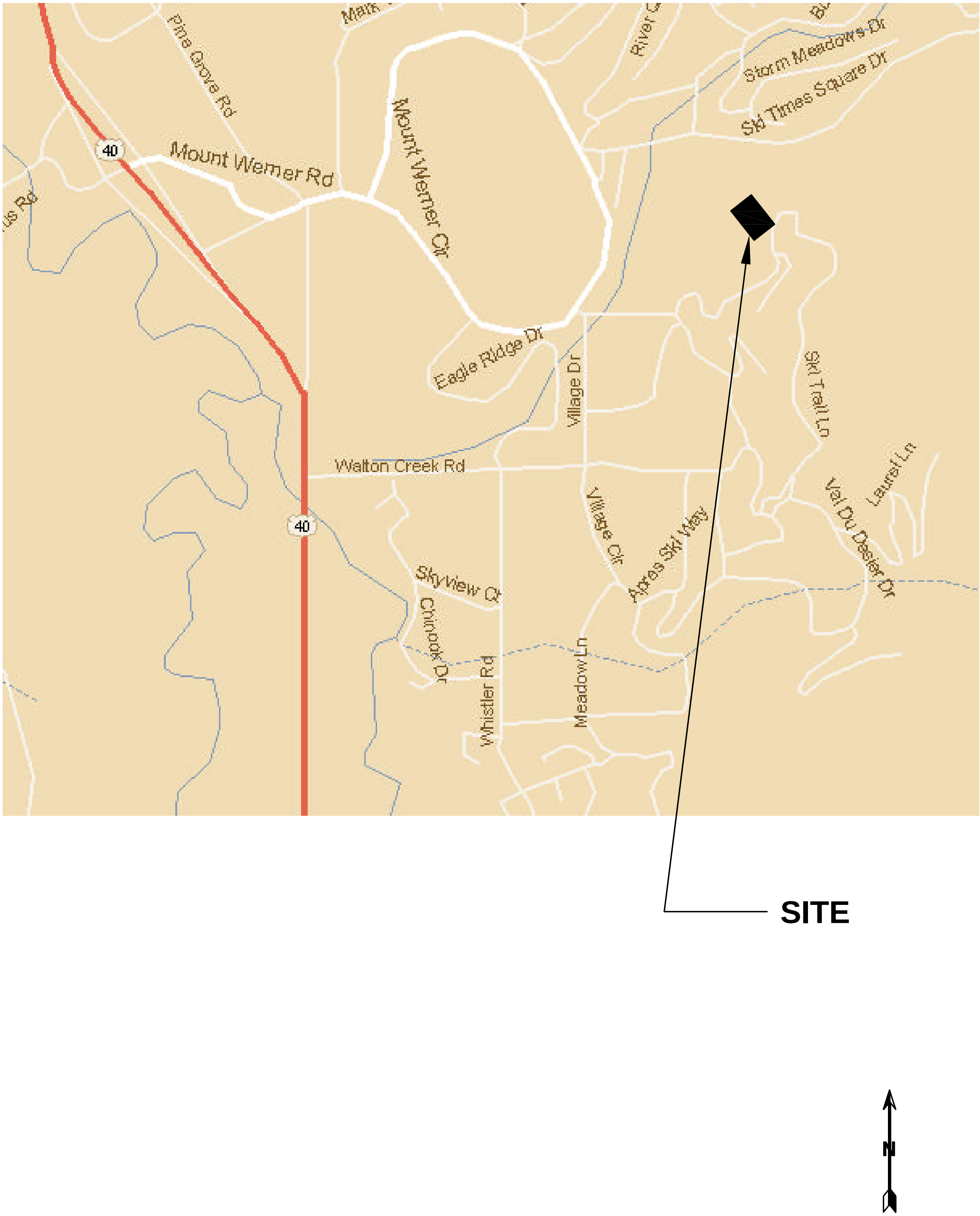
KOECHLEIN CONSULTING ENGINEERS, INC.

Scott B. Myers, P.E.
Senior Engineer

Reviewed by:

William H. Koechlein, P.E.
President

4 copies sent



VICINITY MAP

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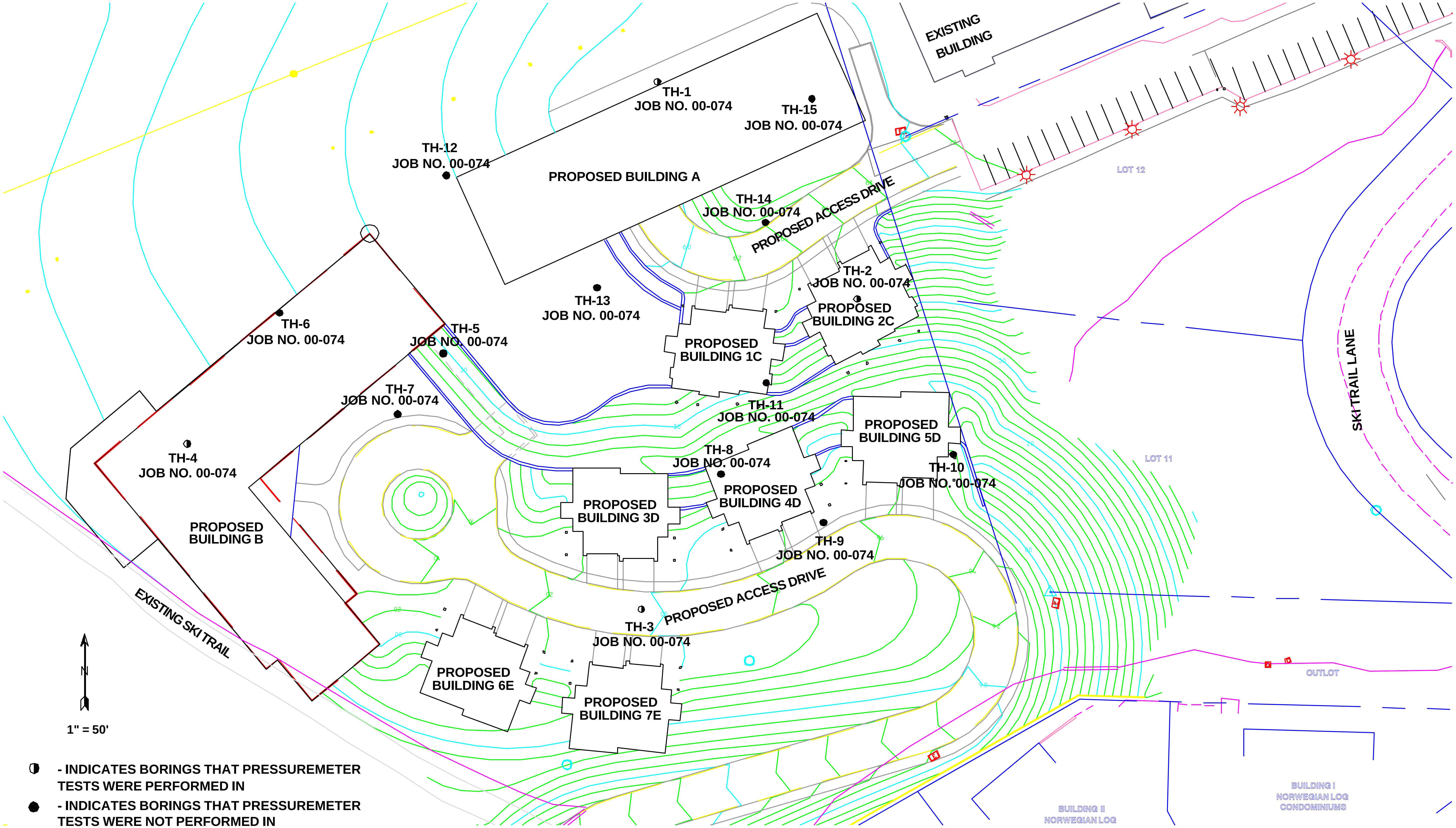
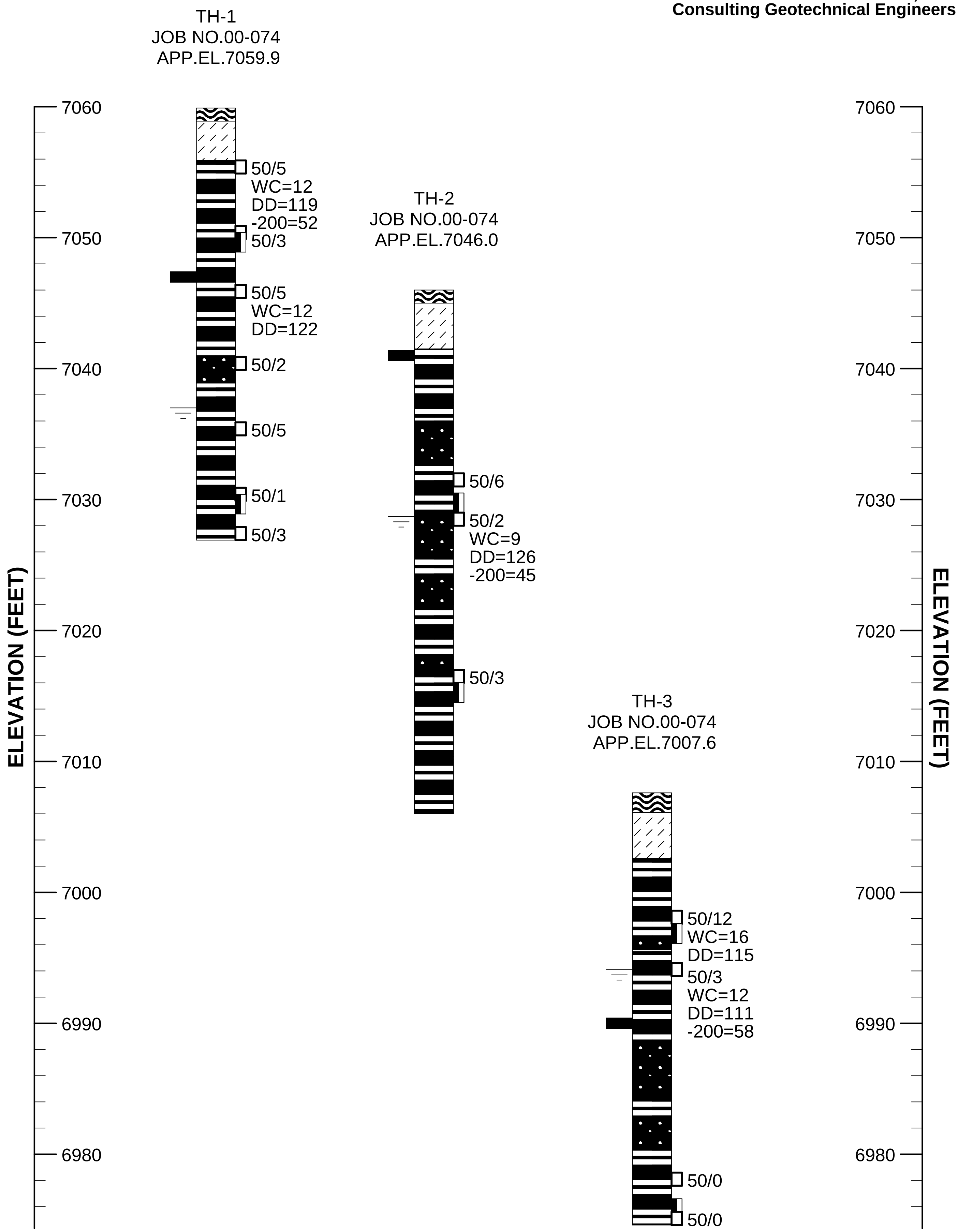
Consulting Geotechnical Engineers



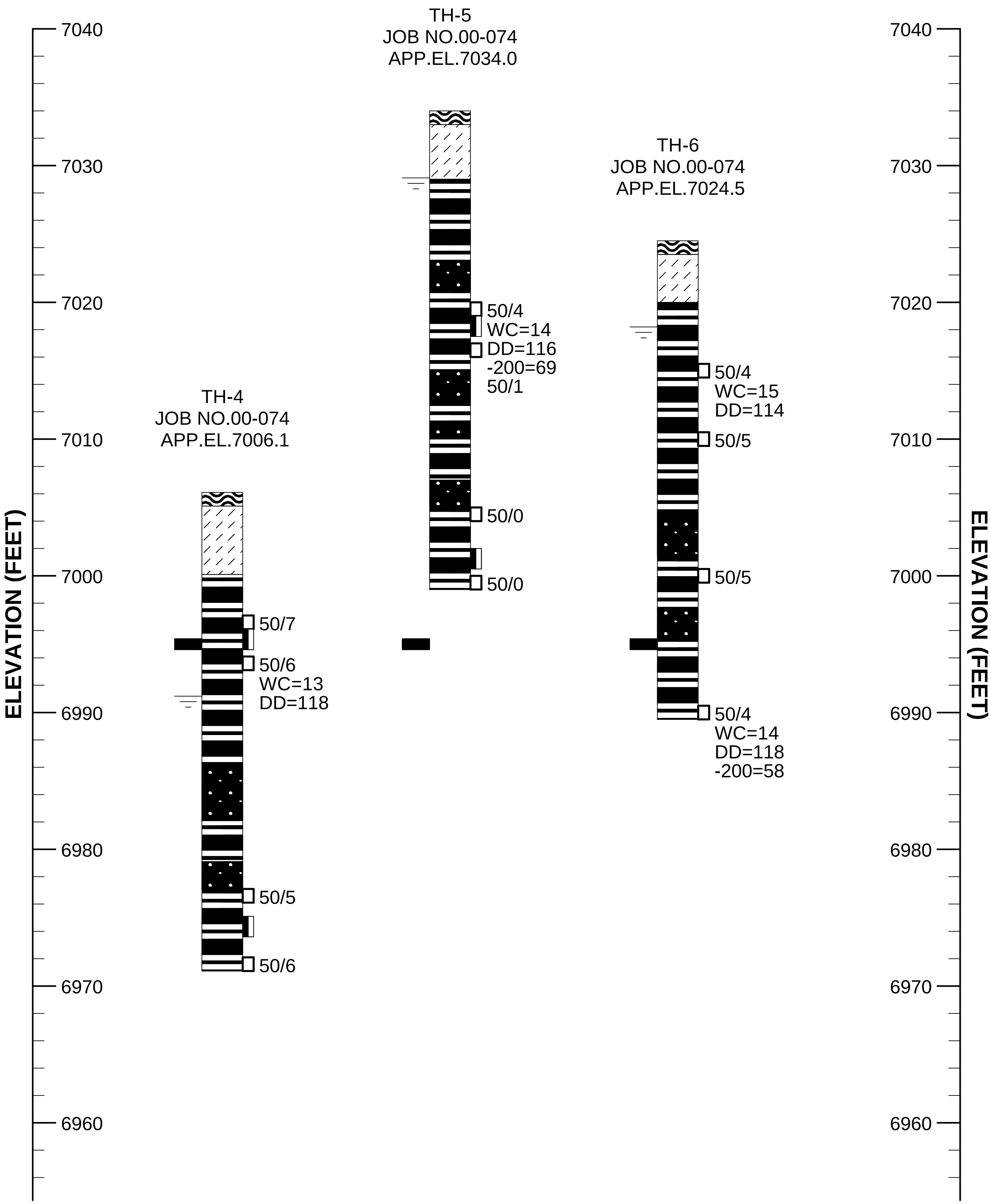
LOCATIONS OF EXPLORATORY BORINGS

JOB NO. 07-020

FIG. 2



LOGS OF EXPLORATORY BORINGS



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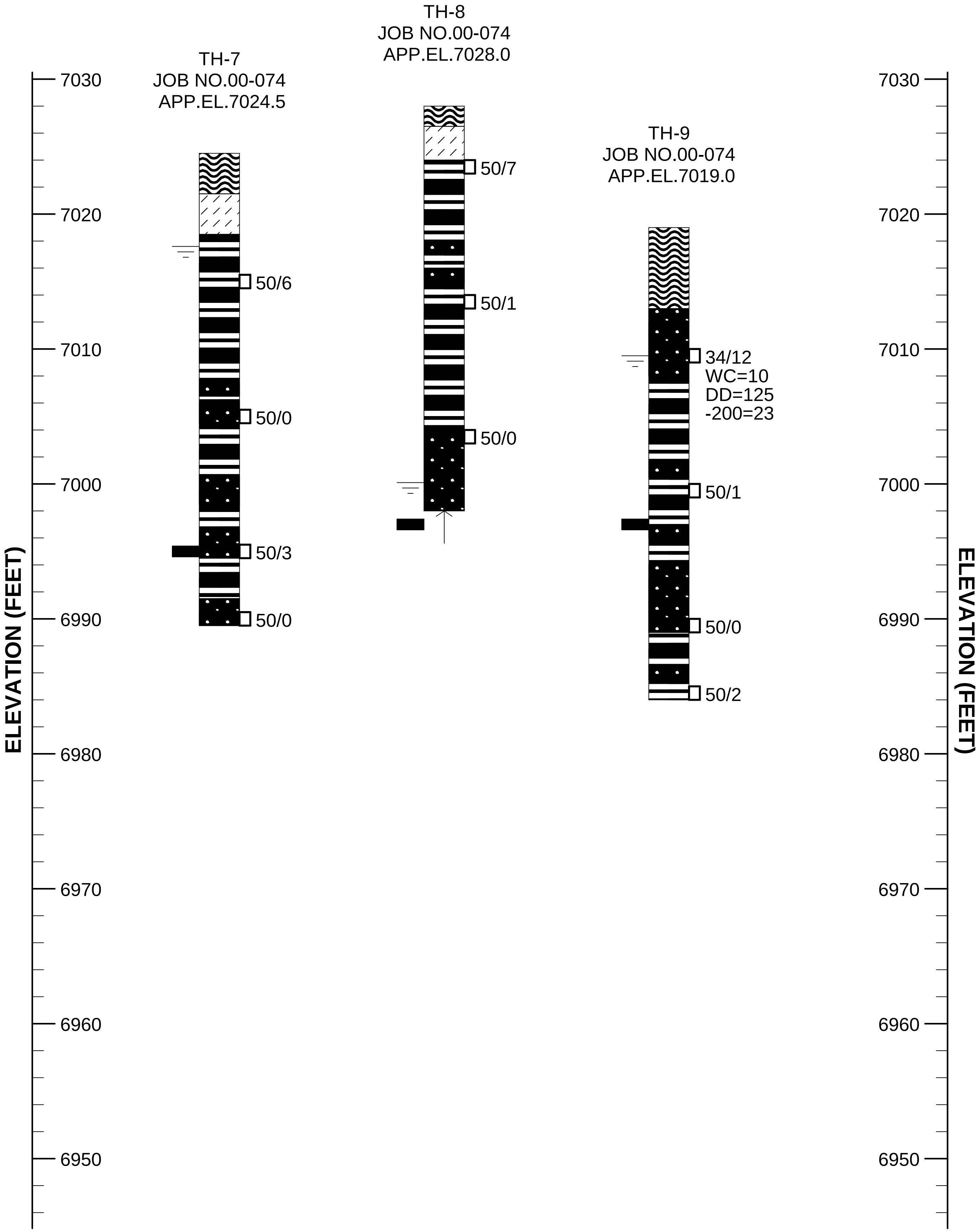
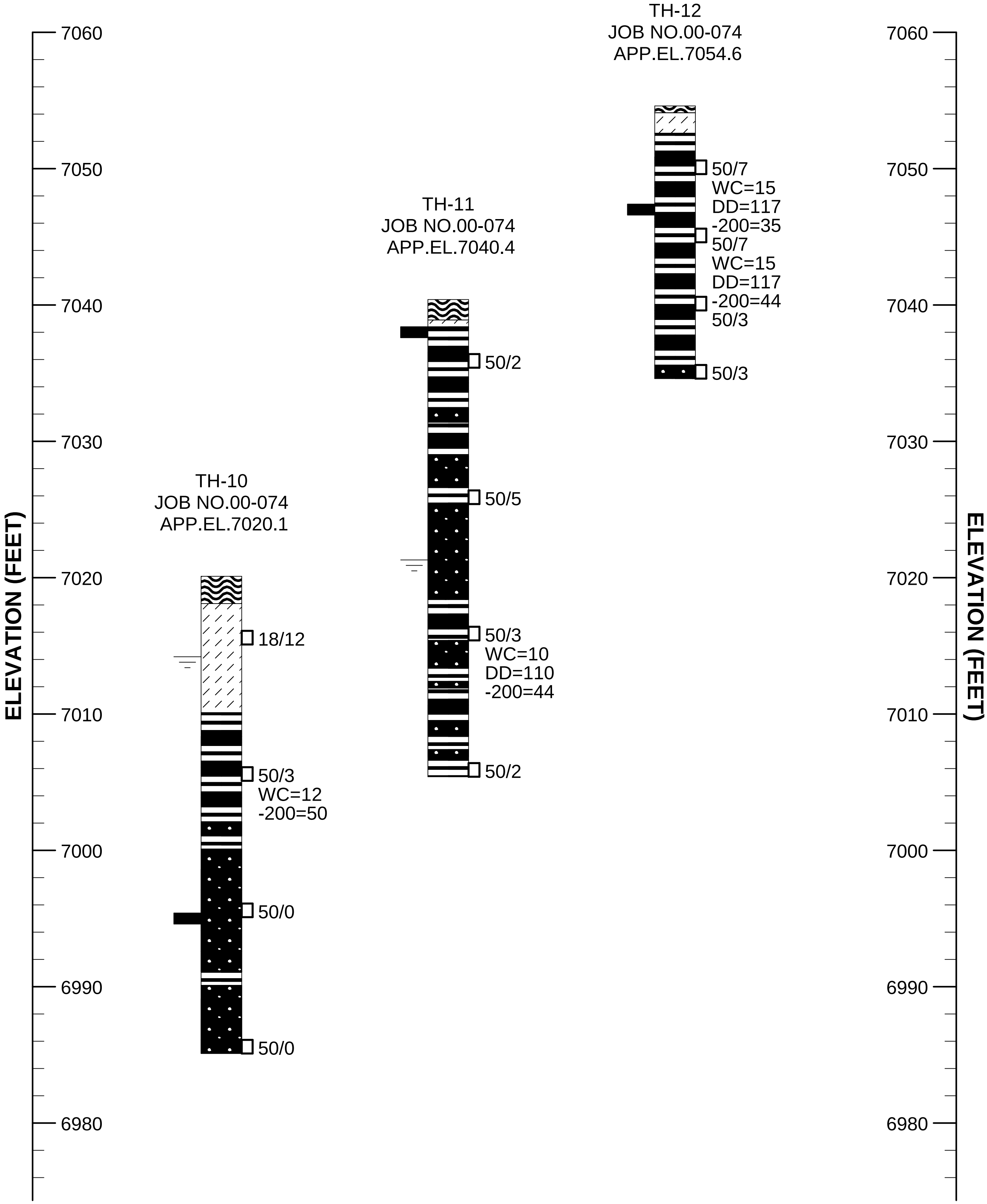
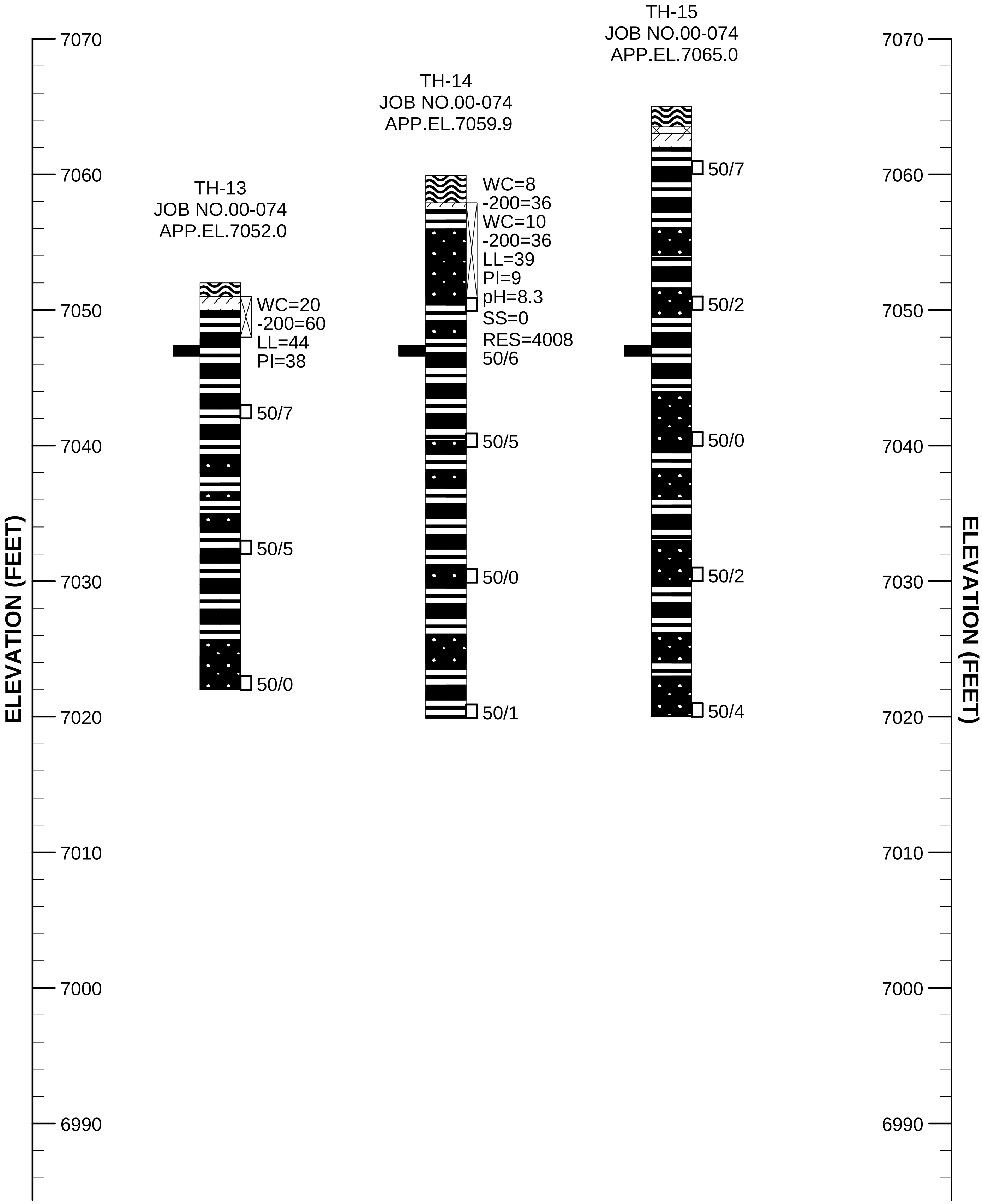


LOGS OF EXPLORATORY BORINGS



LOGS OF EXPLORATORY BORINGS



LOGS OF EXPLORATORY BORINGS

LEGEND:

TOPSOIL

CLAY, Sandy, Moist, Stiff to very stiff, Brown.

CLAYSTONE, Sandy, Dry to moist, Medium hard to very hard, Brown, Tan, Grey.

SANDSTONE, Coarse to fine grained, Gravelly, Dry, Very hard, Multi-colored, Tan, Grey.

FILL, Clay, Sandy, Cobbles, Dry to moist, Medium stiff, Brown.

WATER. Indicates depth of water measured 24 to 48 hours after drilling.

LOWEST FLOOR. Indicates approximate elevation of lowest floor for the closest building.

REFUSAL. Indicates practical drill rig refusal.

CALIFORNIA DRIVE SAMPLE. The symbol 50/5 indicates that 50 blows of a 140 pound hammer falling 30 inches were required to drive a 2.5 inch O.D. sampler 5 inches.

PRESSUREMETER TEST. Indicates that a pressuremeter test was performed.

BULK SAMPLE. Obtained from auger cuttings.

<u>Notes:</u>

1. Exploratory borings were drilled on May 2 thru 5, 2000 using a 4-inch diameter continuous flight power auger.

2. Ground water was measured at depths ranging from 4.9 feet (el. 7029.1) to 27.9 feet (el. 7000.1) 24 to 48 hours after drilling. No ground water was measured immediately after drilling in exploratory borings TH-12, TH-13, TH-14 and TH-15.

3. The Boring Logs are subject to the explanations, limitations, and conclusions as contained in this report.

4. Lowest levels are shown for the closest building on the exploratory borings.

5. Laboratory Test Results:
   - WC - Indicates natural moisture (%)
   - DD - Indicates dry density (pcf)
   - -200 - Indicates percent passing the No. 200 sieve (%)
   - LL - Indicates liquid limit (%)
   - PI - Indicates plasticity index (%)
   - pH - Indicates pH
   - SS - Indicates soluble sulfates (%)
   - RES - Indicates Resistivity (ohm/cm)

6. Elevations of the exploratory borings were surveyed in May 2000 and were checked from the existing topographic map of the site, at the time of this field investigation in May 2000. However, the current topographic map (2007) is approximately 4 feet off of the previous topographic survey completed in 2000. We recommend that the elevations presented on the current topographic map be verified.

LEGEND OF EXPLORATORY BORINGS

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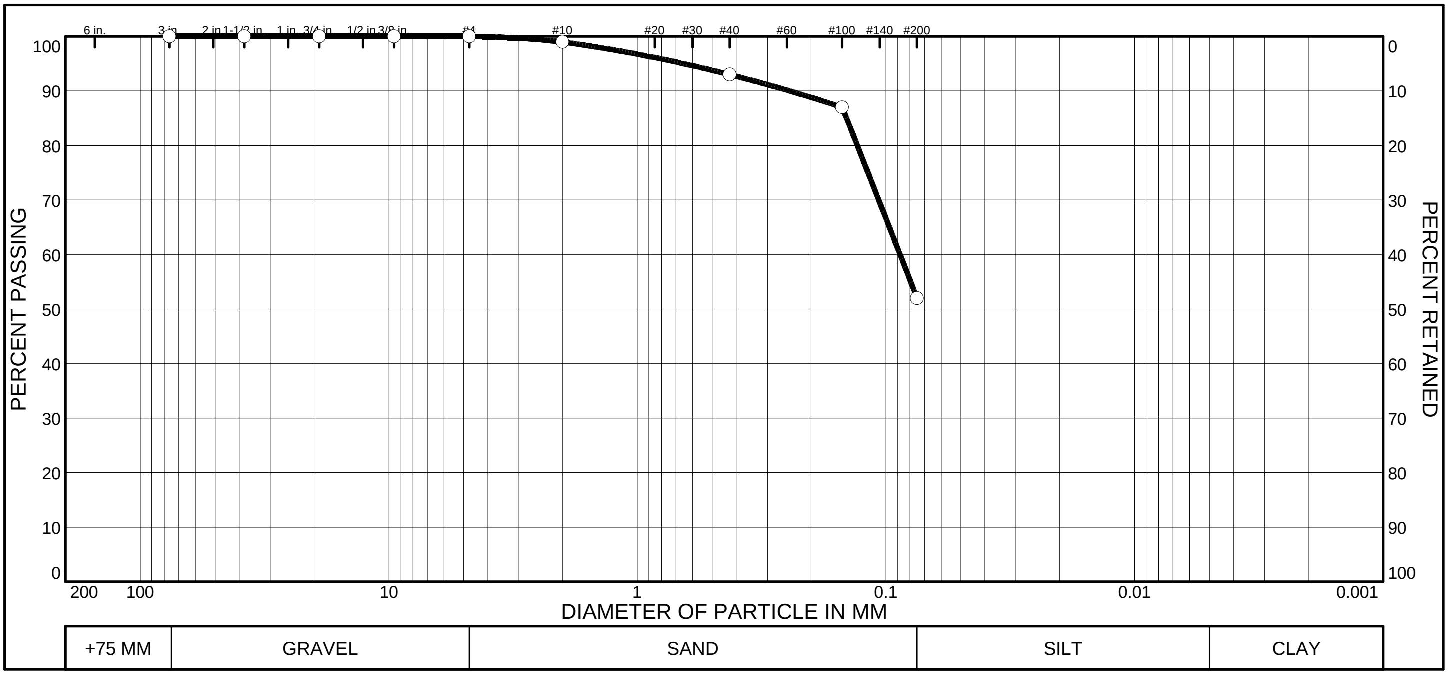
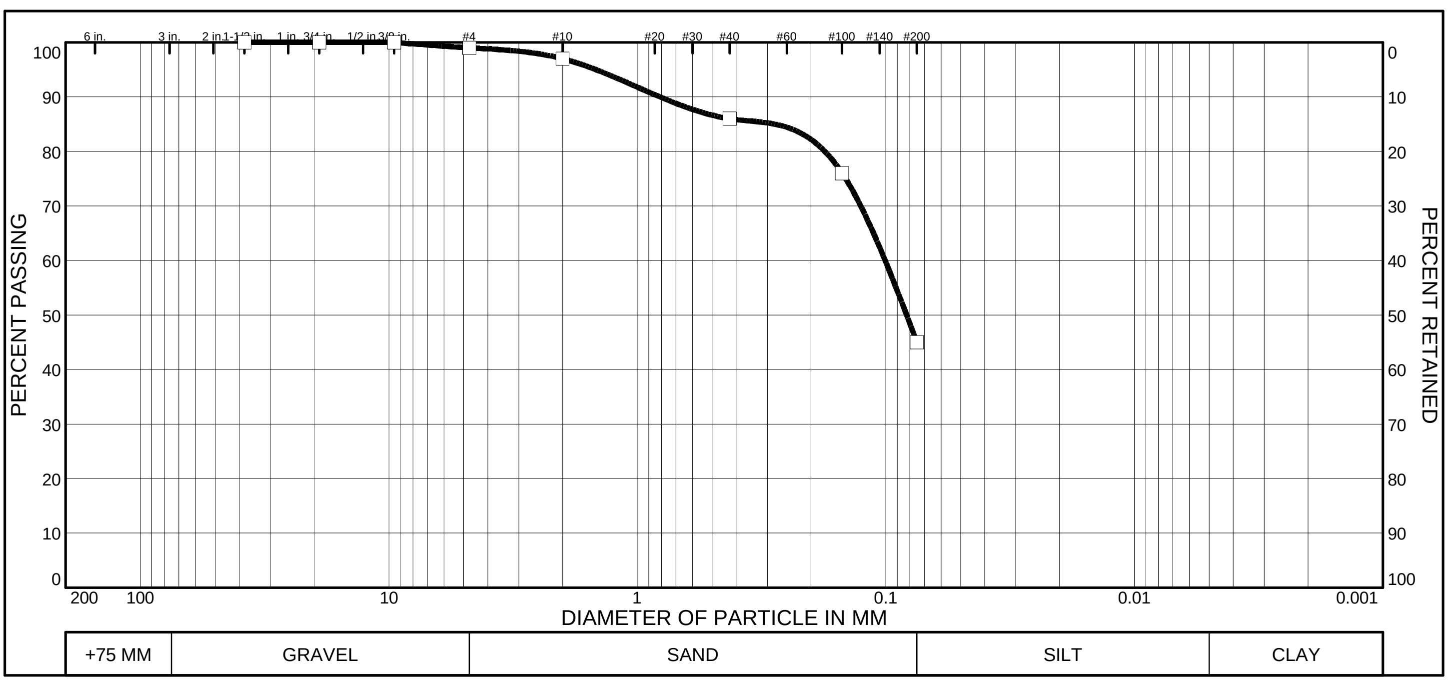


Sample of CLAYSTONE **GRAVEL** 0 **%** **SAND** 48 **%**

Source TH-1 (00-074) **Sample No.** ______ **Elev./Depth** 4.0 feet **SILT & CLAY** 52 **%** **LIQUID LIMIT** ______ **%**

PLASTICITY INDEX ______ **%**



Sample of CLAYSTONE/SANDSTONE **GRAVEL** 1 **%** **SAND** 54 **%**

Source TH-2 (00-074) **Sample No.** ______ **Elev./Depth** 17.0 feet **SILT & CLAY** 45 **%** **LIQUID LIMIT** ______ **%**

PLASTICITY INDEX ______ **%**

GRADATION TEST RESULTS

Job No. 07-020

FIG. 9

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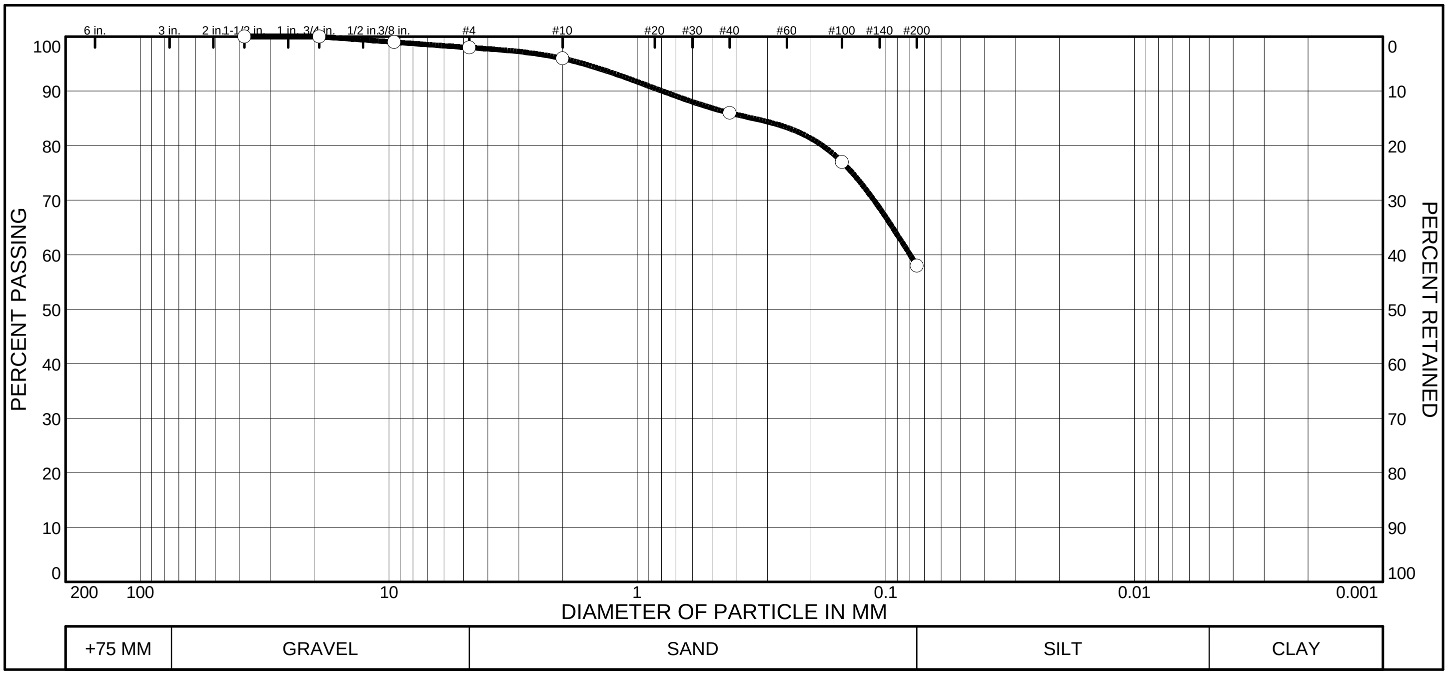
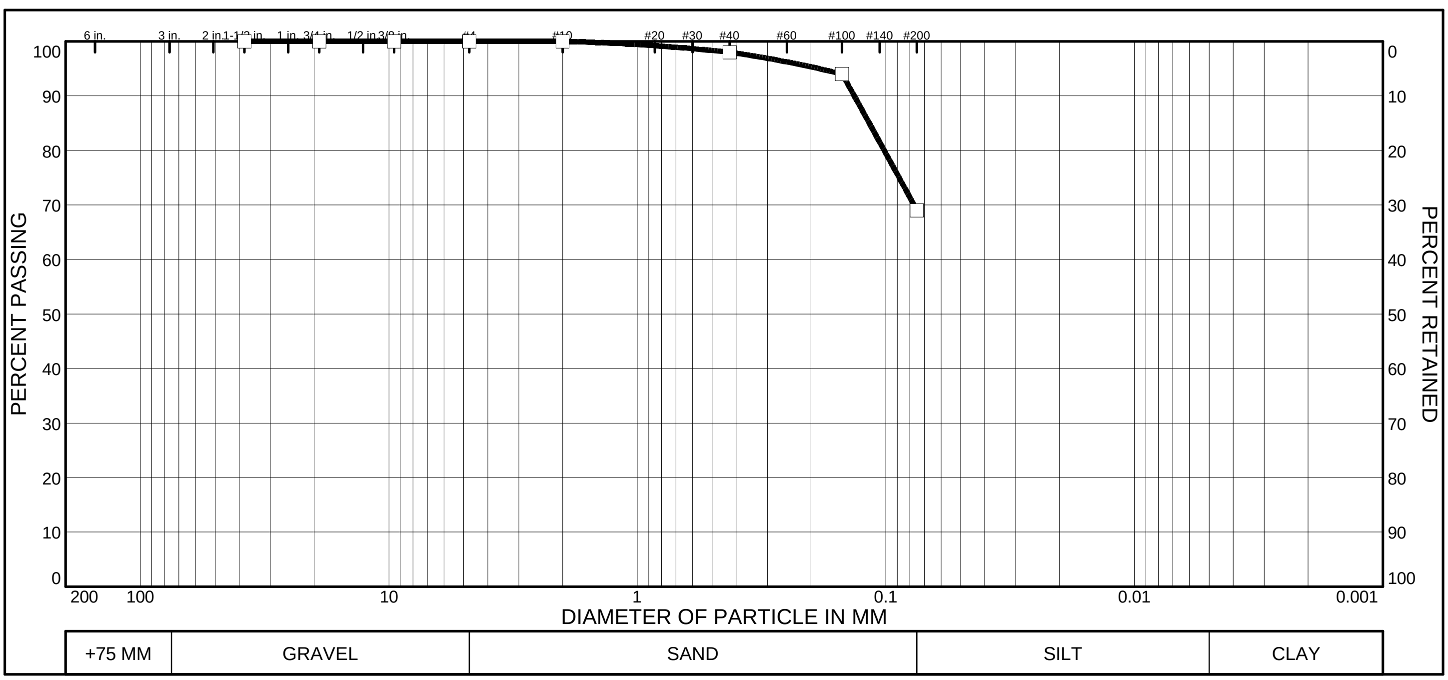


Sample of CLAYSTONE/SANDSTONE **GRAVEL** 2 **%** **SAND** 40 **%**

Source TH-3 (00-074) **Sample No.** ___ **Elev./Depth** 13.0 feet **SILT & CLAY** 58 **%** **LIQUID LIMIT** ___ **%**

PLASTICITY INDEX ___ **%**



Sample of CLAYSTONE, Sandy **GRAVEL** ___ **%** **SAND** 31 **%**

Source TH-5 (00-074) **Sample No.** ___ **Elev./Depth** 14.0 feet **SILT & CLAY** 69 **%** **LIQUID LIMIT** ___ **%**

PLASTICITY INDEX ___ **%**

GRADATION TEST RESULTS

Job No. 07-020

FIG. 10

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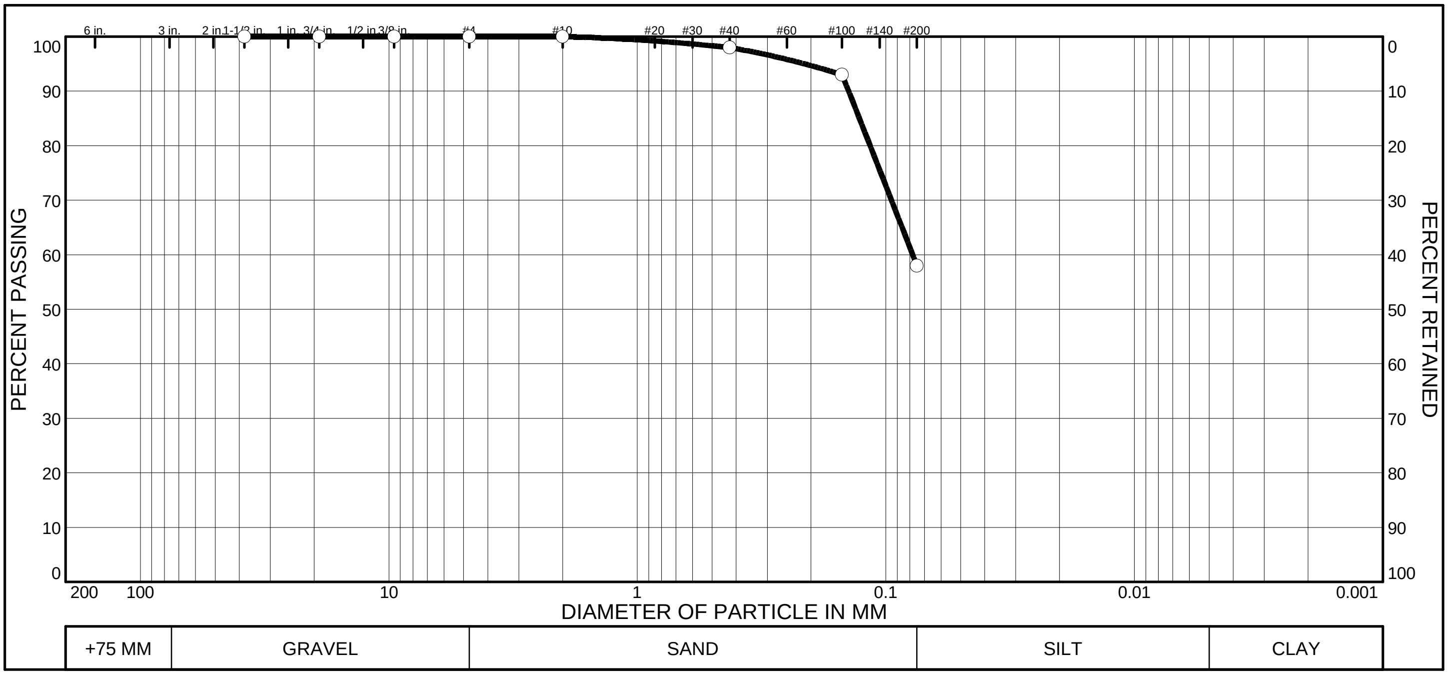
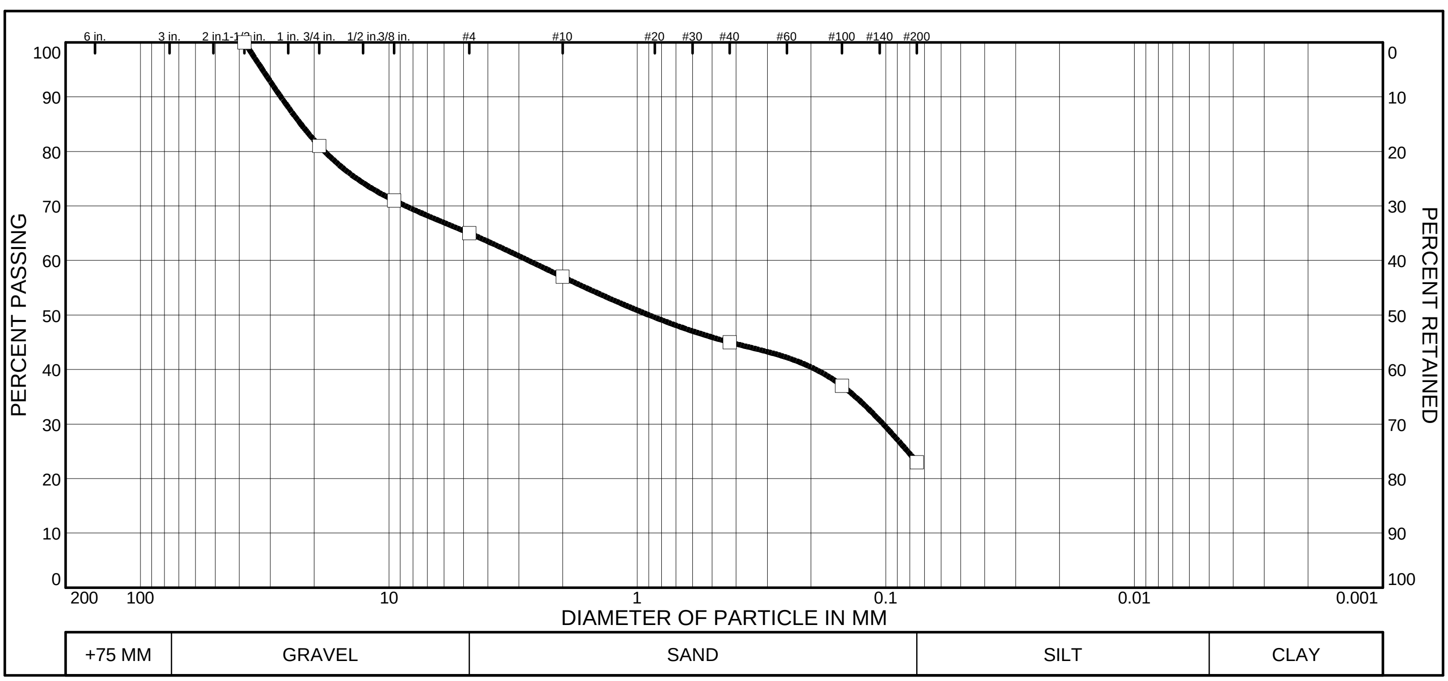


Sample of CLAYSTONE/SANDSTONE
Source TH-6 (00-074) **Sample No.** ______ **Elev./Depth** 34.0 feet

GRAVEL ______ % **SAND** 42 %
SILT & CLAY 58 % **LIQUID LIMIT** ______ %
PLASTICITY INDEX ______ %



Sample of SANDSTONE
Source TH-9 (00-074) **Sample No.** ______ **Elev./Depth** 9.0 feet

GRAVEL 35 % **SAND** 42 %
SILT & CLAY 23 % **LIQUID LIMIT** ______ %
PLASTICITY INDEX ______ %

GRADATION TEST RESULTS

Job No. 07-020 **FIG.** 11

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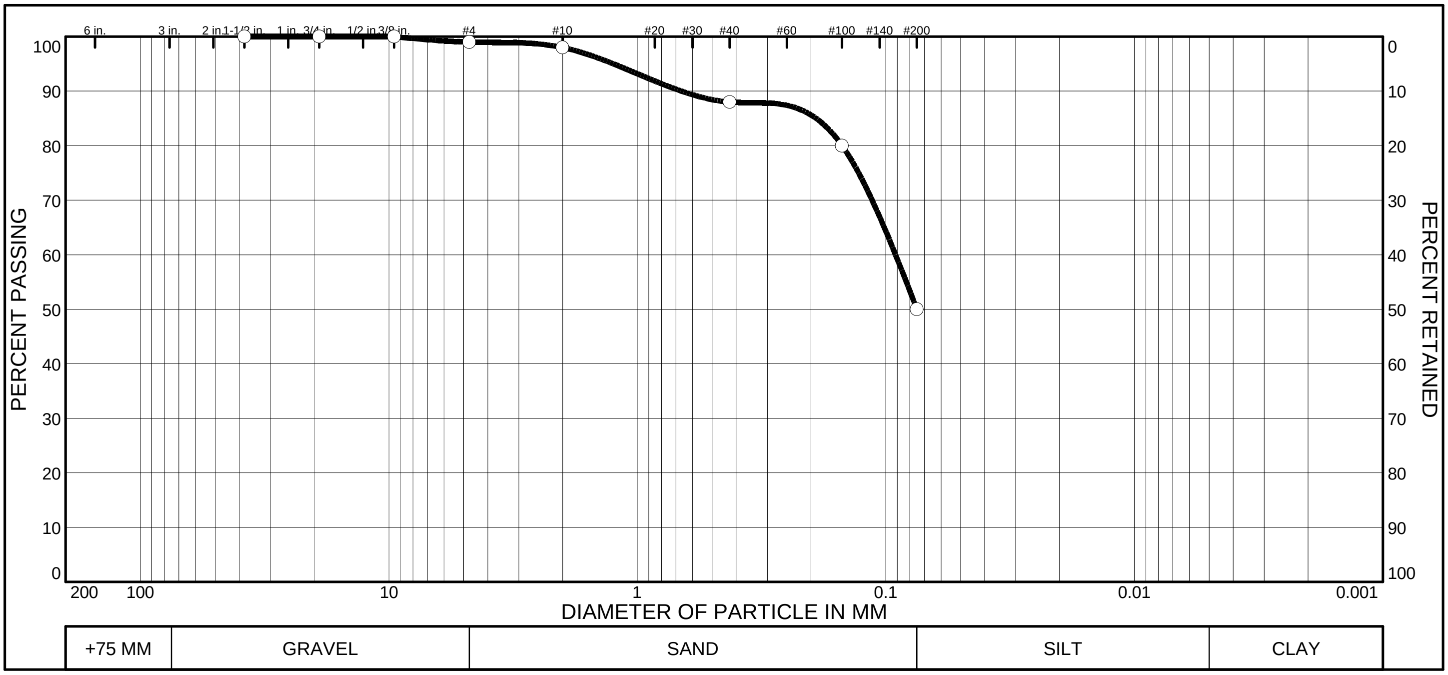
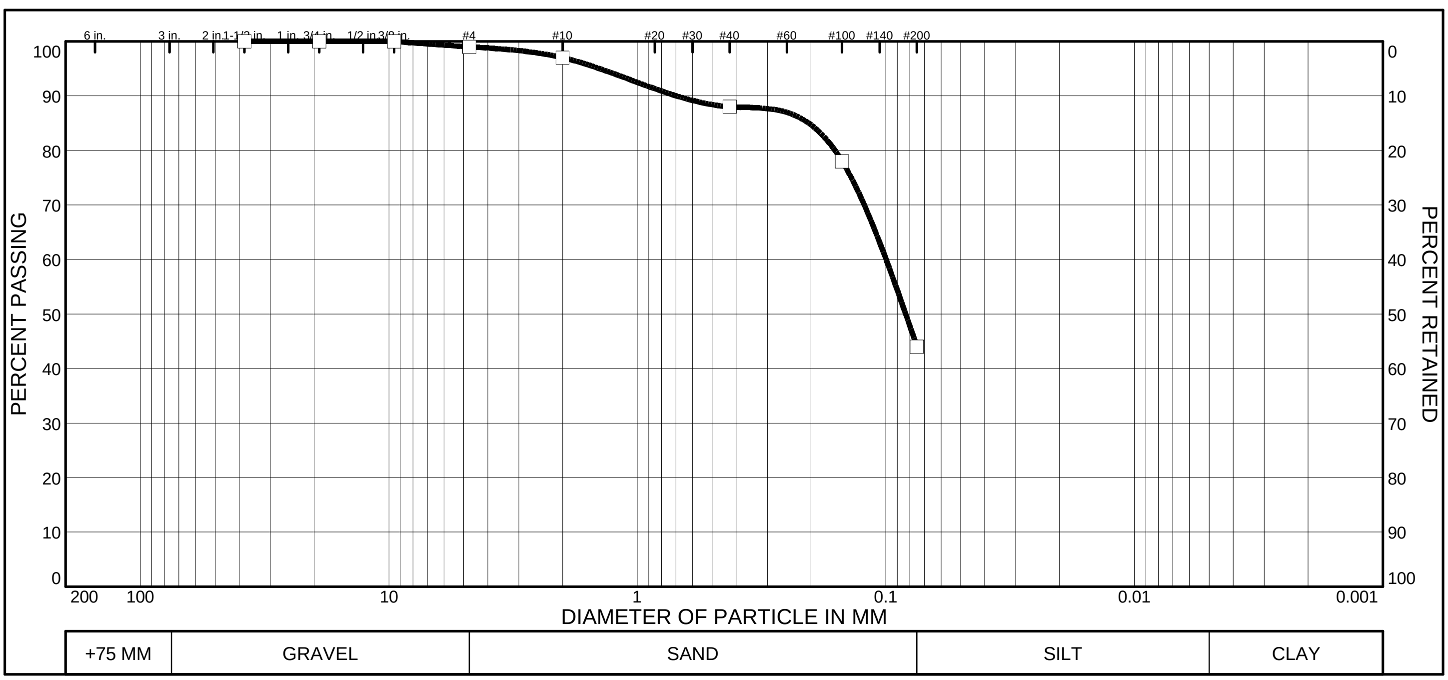


Sample of CLAYSTONE/SANDSTONE **GRAVEL** 1 **%** **SAND** 49 **%**

Source TH-10 (00-074) **Sample No.** ________ **Elev./Depth** 14.0 feet **SILT & CLAY** 50 **%** **LIQUID LIMIT** ____ **%**

PLASTICITY INDEX ________ **%**



Sample of CLAYSTONE/SANDSTONE **GRAVEL** 1 **%** **SAND** 55 **%**

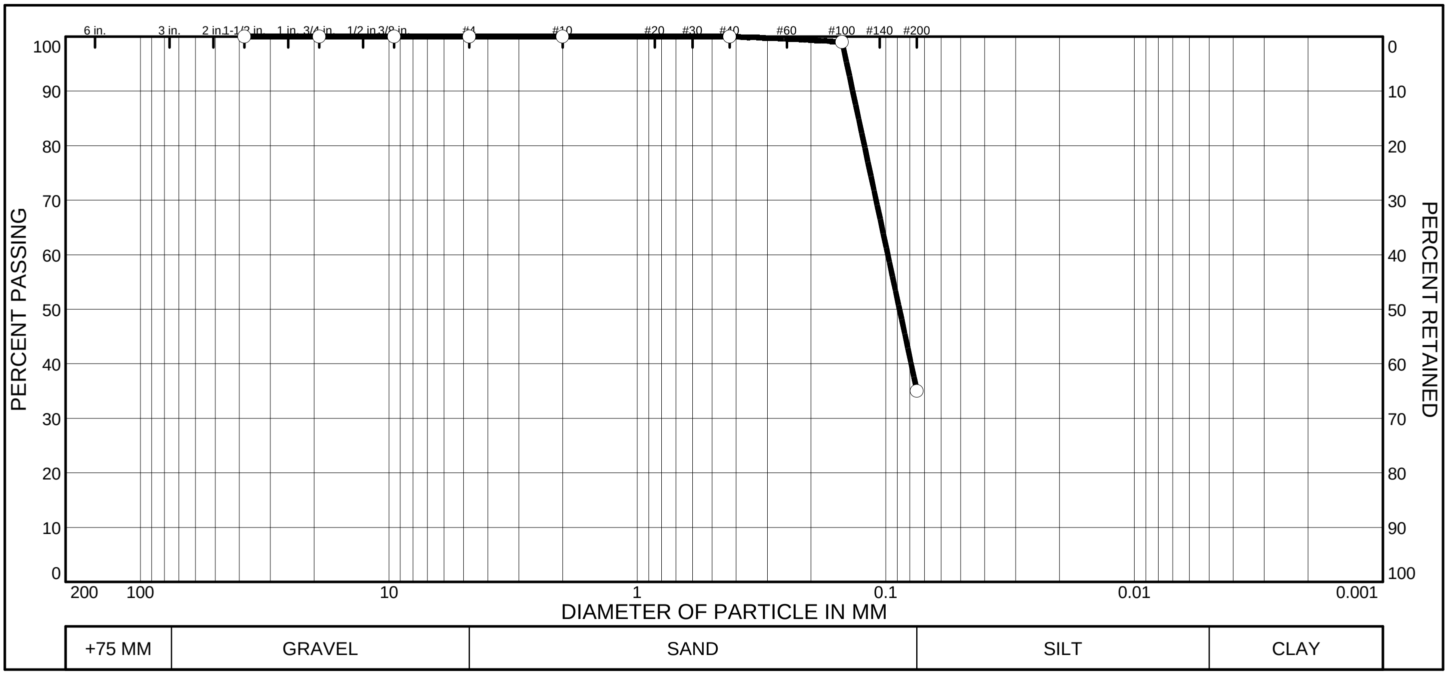
Source TH-11 (00-074) **Sample No.** ________ **Elev./Depth** 24.0 feet **SILT & CLAY** 44 **%** **LIQUID LIMIT** ____ **%**

PLASTICITY INDEX ________ **%**

GRADATION TEST RESULTS

Job No. 07-020 **FIG.** 12

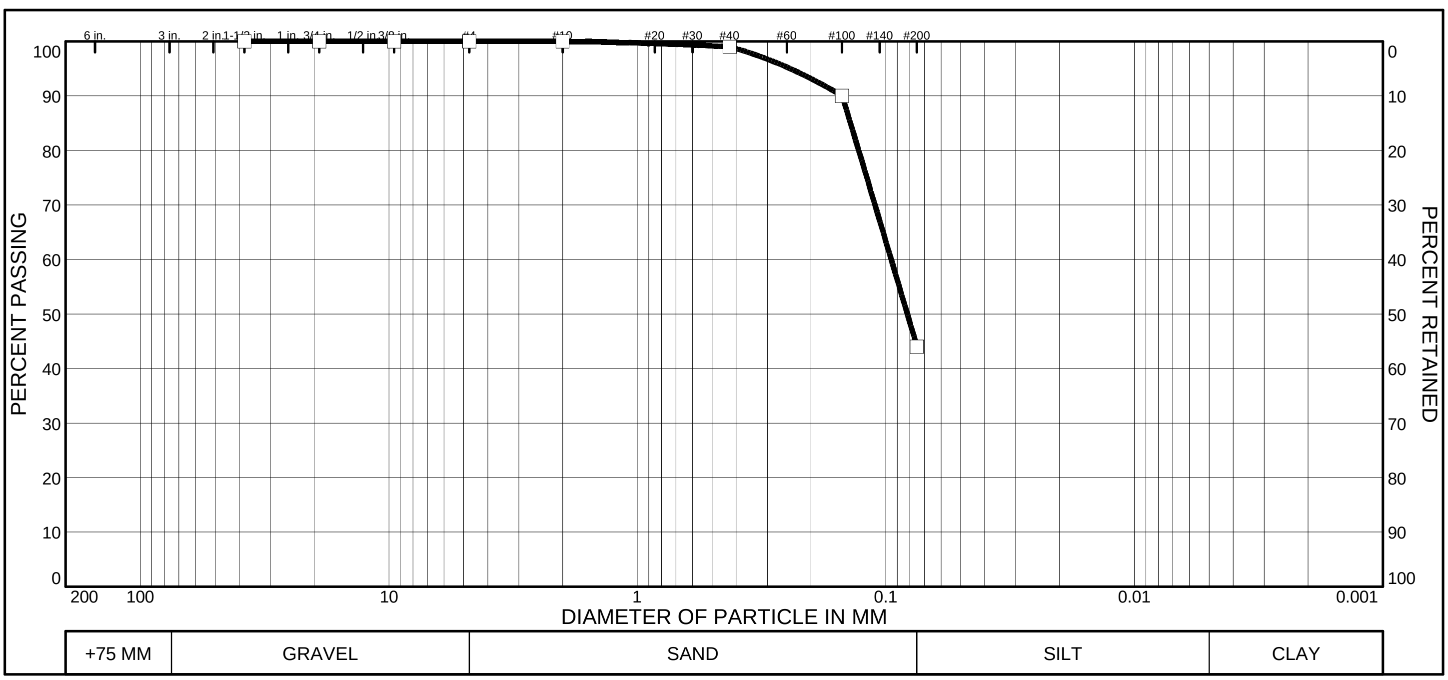
KOECHLEIN CONSULTING ENGINEERS



Sample of CLAYSTONE/SANDSTONE **GRAVEL** ____ **%** **SAND** 65 **%**

Source TH-12 (00-074) **Sample No.** ____ **Elev./Depth** 4.0 feet **SILT & CLAY** 35 **%** **LIQUID LIMIT** ____ **%**

PLASTICITY INDEX ____ **%**



Sample of CLAYSTONE/SANDSTONE **GRAVEL** ____ **%** **SAND** 56 **%**

Source TH-12 (00-074) **Sample No.** ____ **Elev./Depth** 9.0 feet **SILT & CLAY** 44 **%** **LIQUID LIMIT** ____ **%**

PLASTICITY INDEX ____ **%**

GRADATION TEST RESULTS

Job No. 07-020 **FIG.** 13

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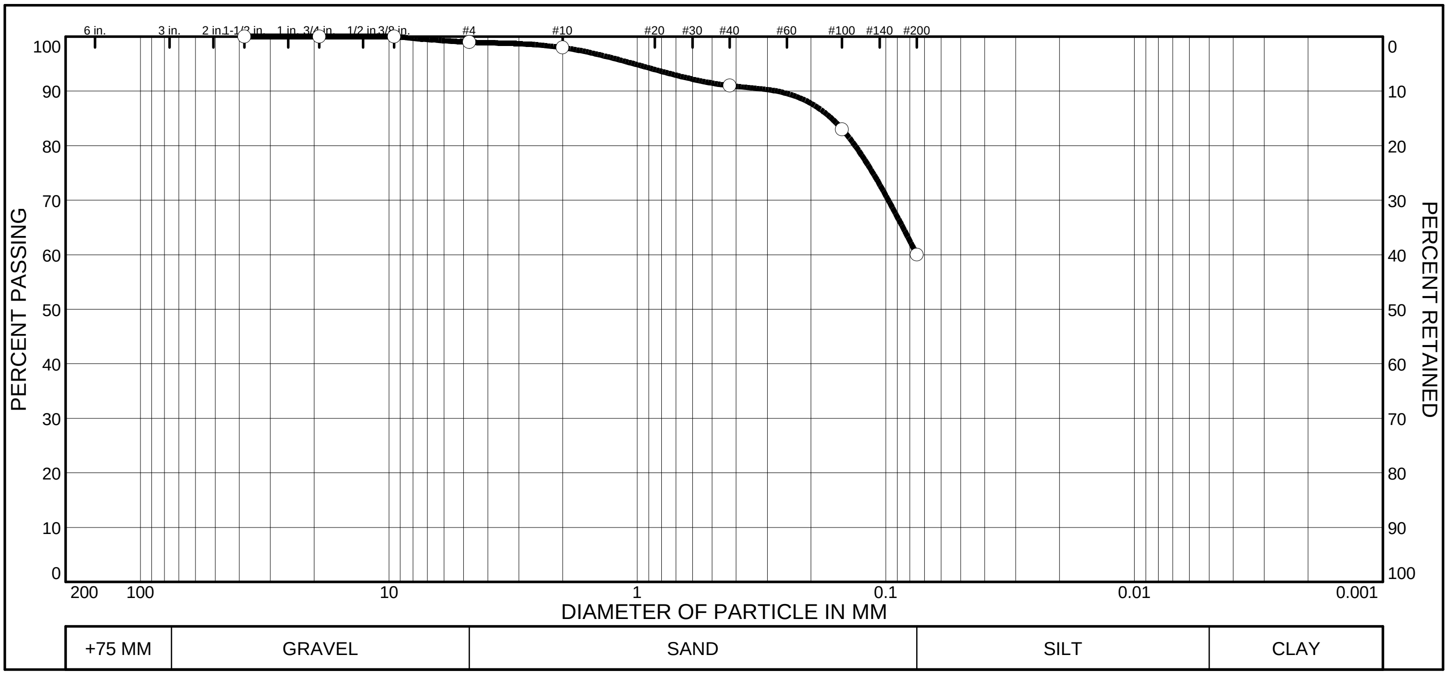
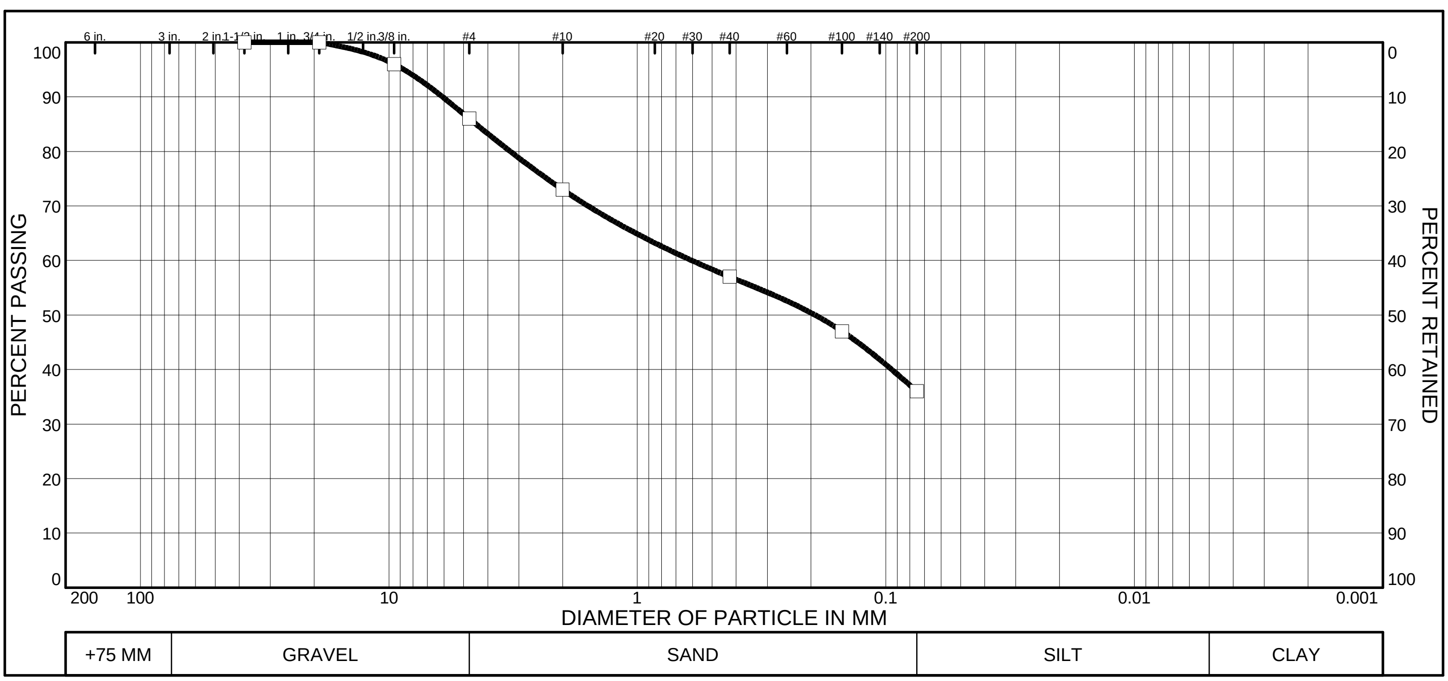


Sample of SILT, Sandy, Clayey **GRAVEL** 1 **%** **SAND** 39 **%**

Source TH-13 (00-074) **Sample No.** ____ **Elev./Depth** 0.0-4.0 feet **SILT & CLAY** 60 **%** **LIQUID LIMIT** ____ **%**

PLASTICITY INDEX ____ **%**



Sample of CLAYSTONE/SANDSTONE **GRAVEL** 14 **%** **SAND** 50 **%**

Source TH-14 (00-074) **Sample No.** ____ **Elev./Depth** 0.0-9.0 feet **SILT & CLAY** 36 **%** **LIQUID LIMIT** ____ **%**

PLASTICITY INDEX ____ **%**

GRADATION TEST RESULTS

Job No. 07-020 **FIG.** 14

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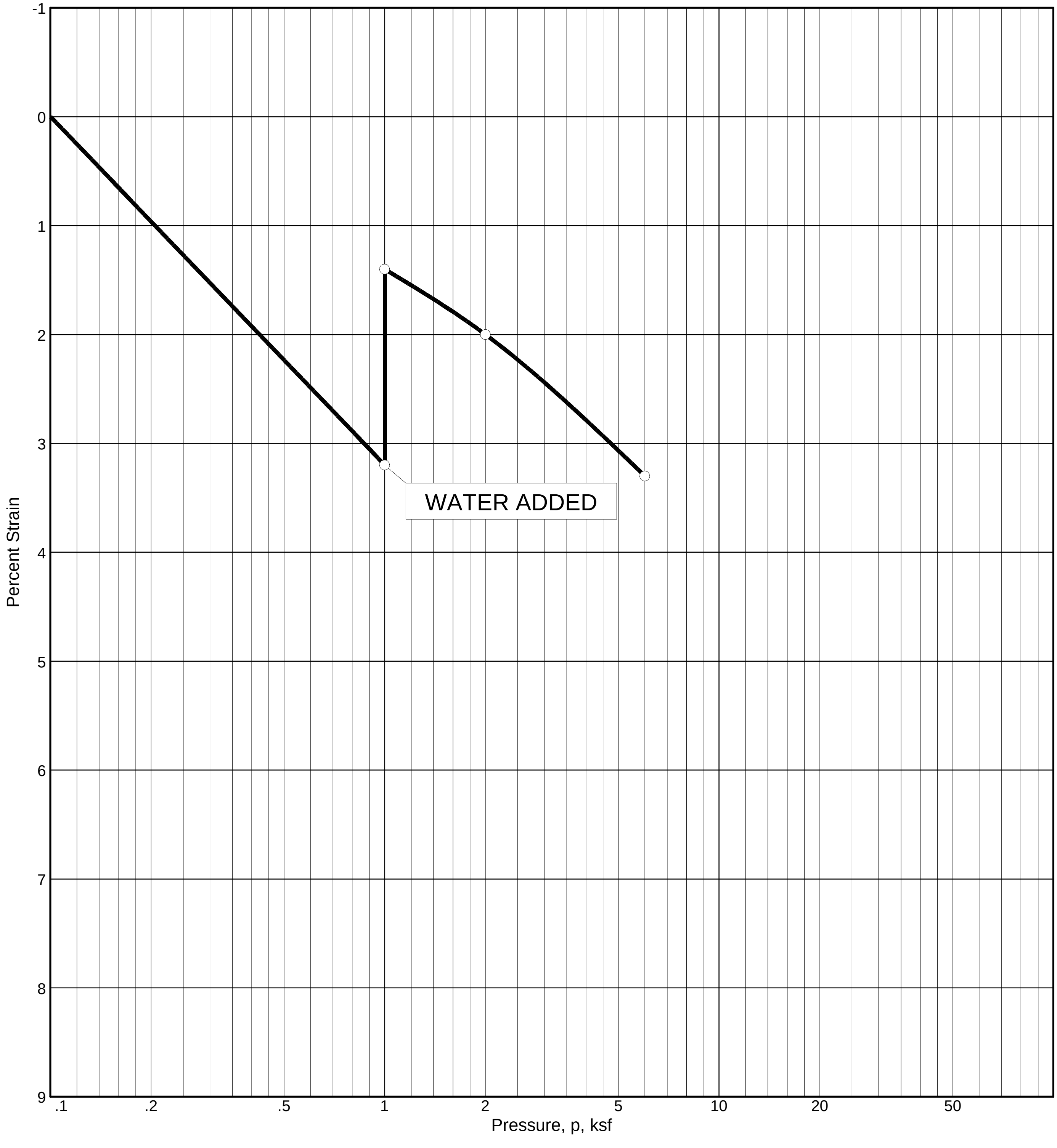

Sample of CLAYSTONE **Natural Dry Unit Weight=** 121.6 **(pcf)**

Source TH-1 (00-074) **Sample No.** ______ **Elev./Depth** 13.5 feet **Natural Moisture Content=** 12 **%**

SWELL-CONSOLIDATION TEST RESULTS

Job No. 07-020 **FIG.** 15

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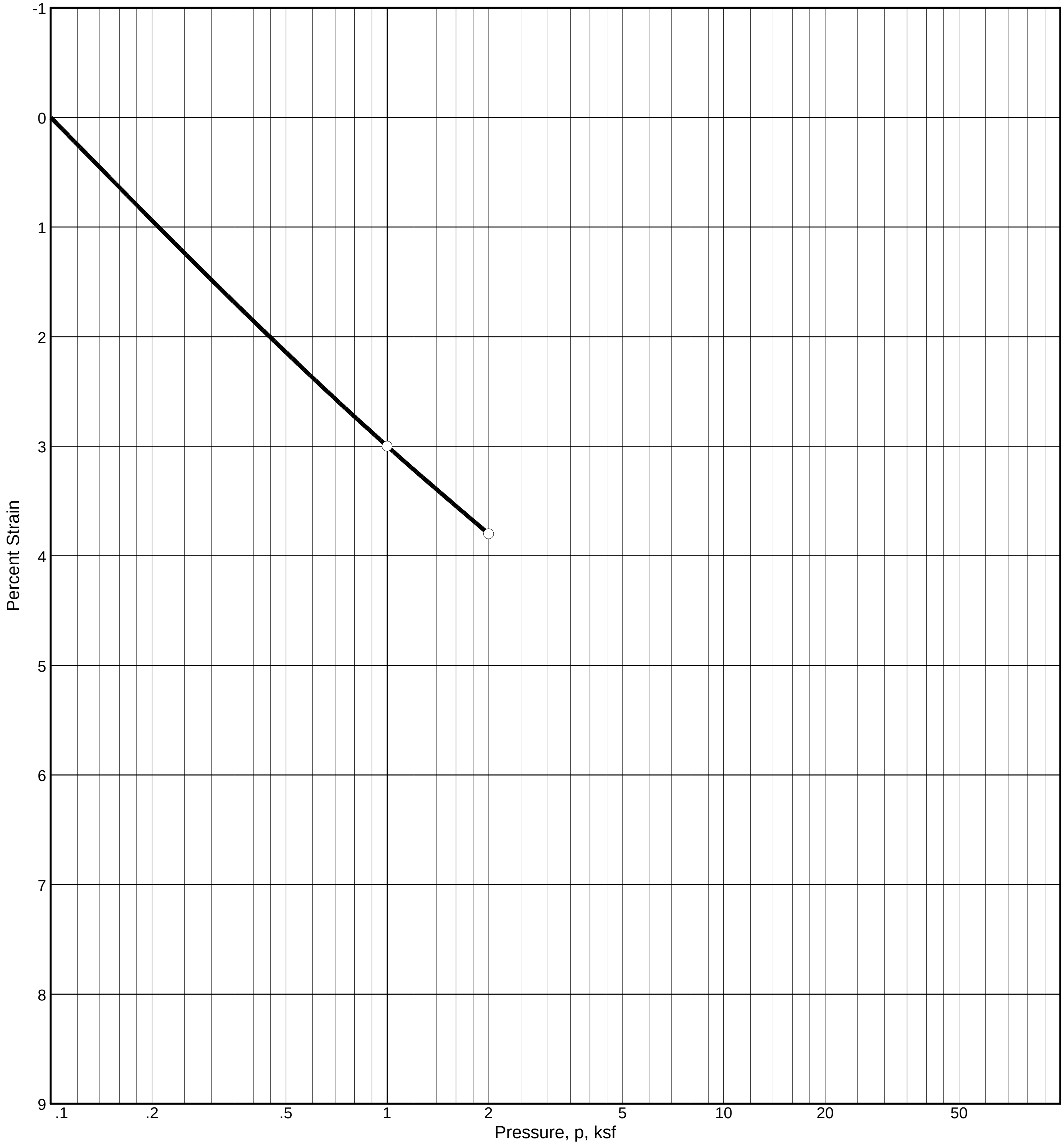

Sample of CLAYSTONE/SANDSTONE **Natural Dry Unit Weight=** 114.9 **(pcf)**

Source TH-3 (00-074) **Sample No.** ____ **Elev./Depth** 9.0 feet **Natural Moisture Content=** 16 **%**

SWELL-CONSOLIDATION TEST RESULTS

Job No. 07-020 **FIG.** 16

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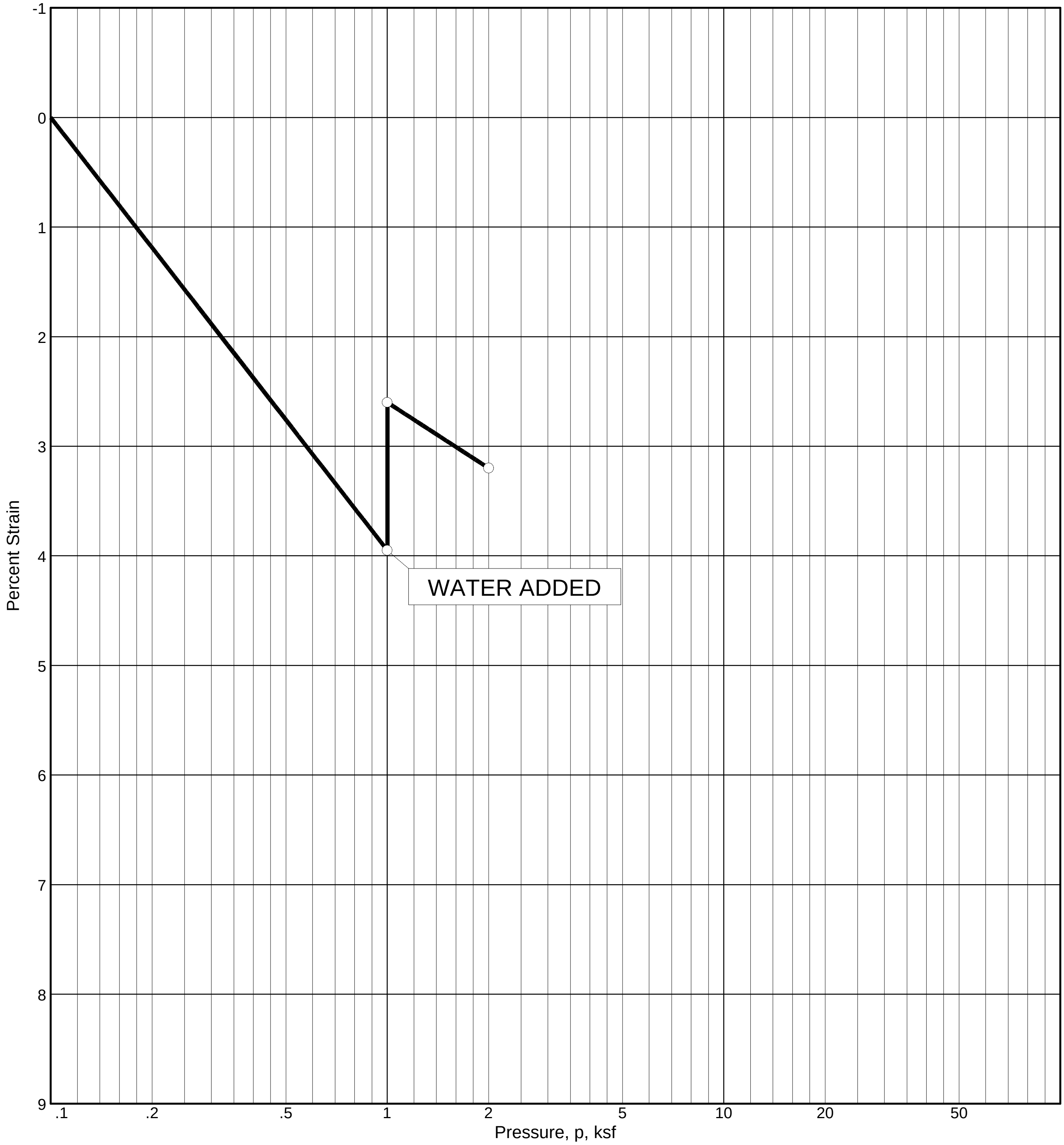

Sample of CLAYSTONE **Natural Dry Unit Weight=** 117.8 **(pcf)**

Source TH-4 (00-074) **Sample No.** ____ **Elev./Depth** 12.0 feet **Natural Moisture Content=** 13 **%**

SWELL-CONSOLIDATION TEST RESULTS

Job No. 07-020 **FIG.** 17

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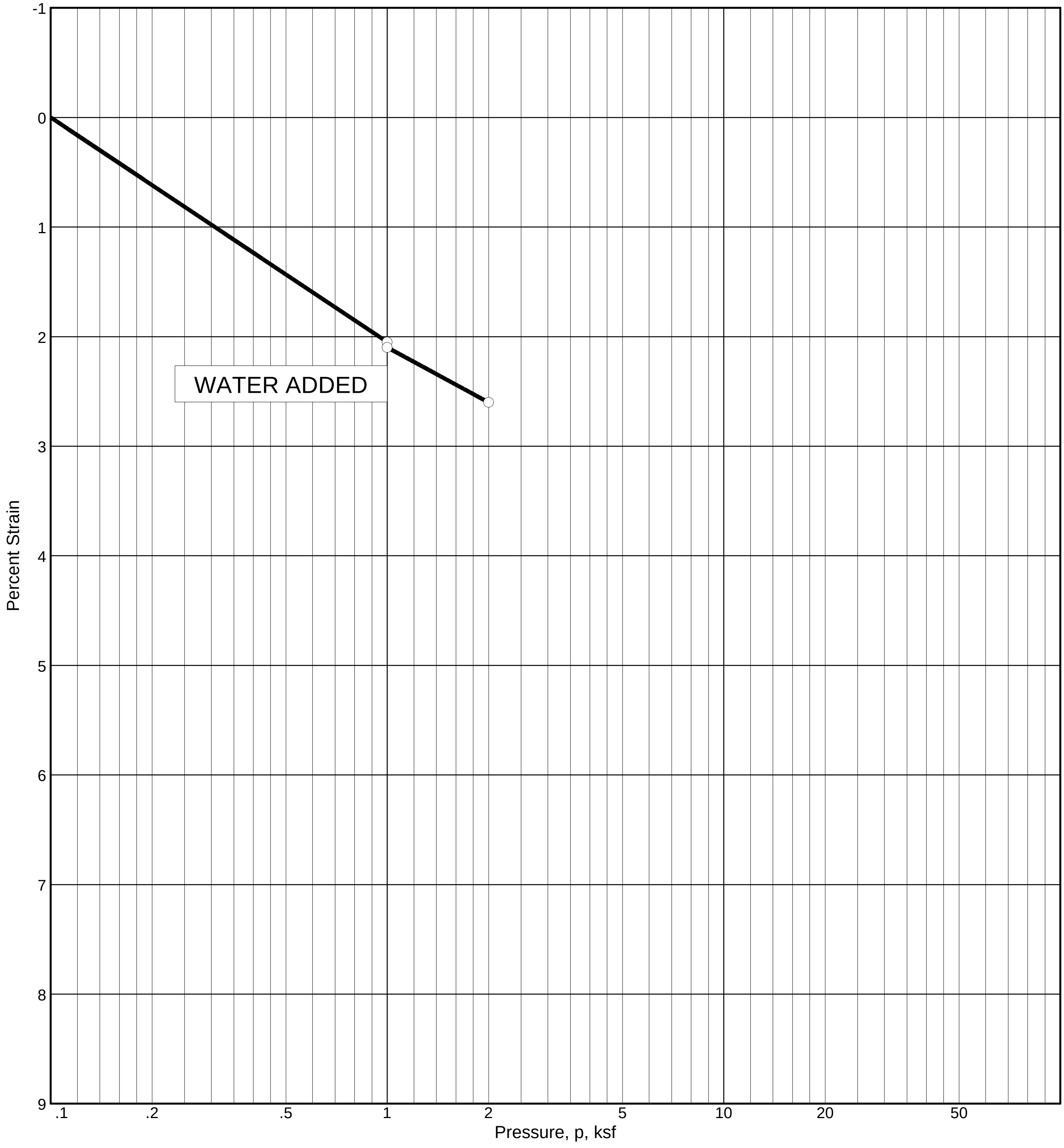


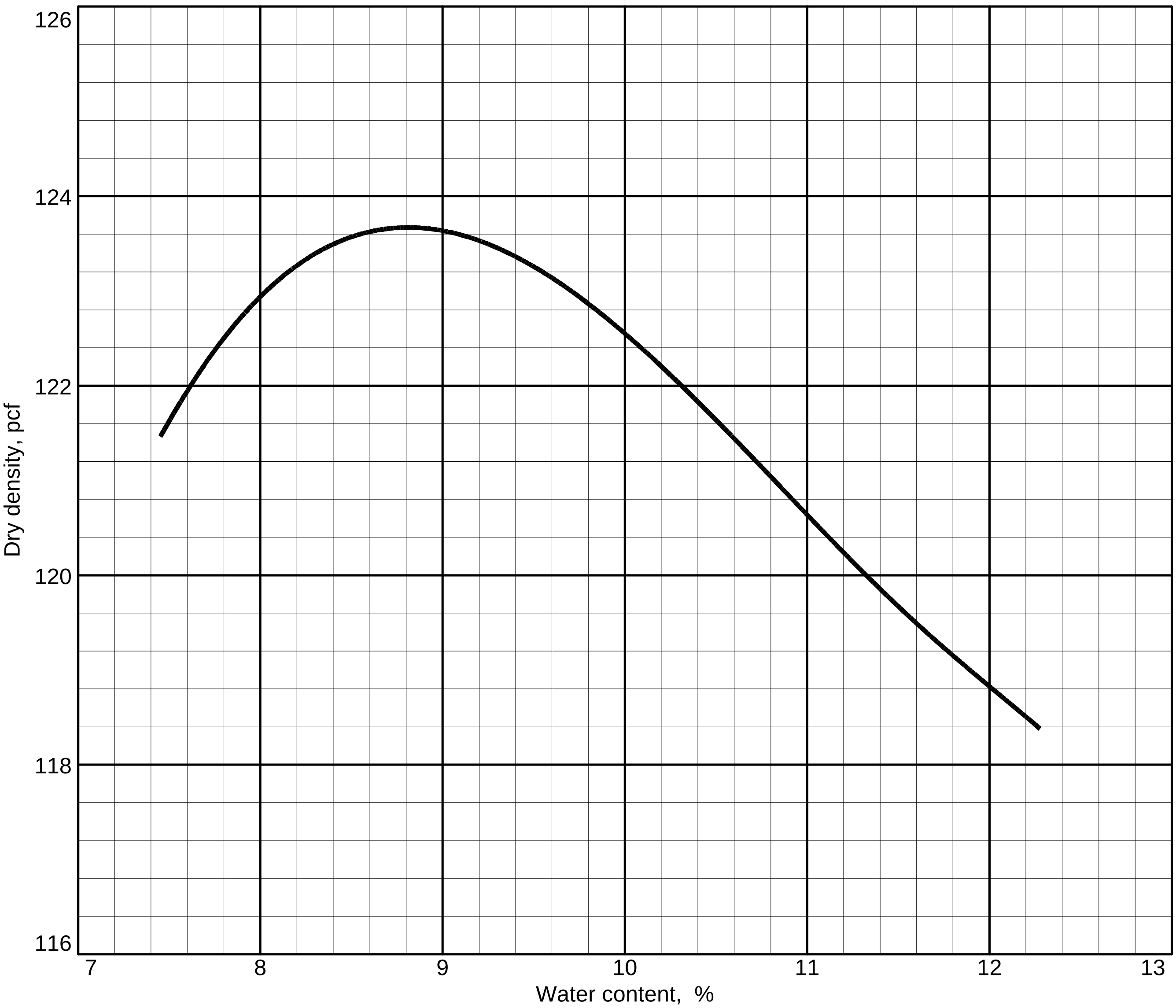
Sample of CLAYSTONE/SANDSTONE **Natural Dry Unit Weight=** 113.8 **(pcf)**

Source TH-6 (00-074) **Sample No.** ______ **Elev./Depth** 9.0 feet **Natural Moisture Content=** 15 **%**

SWELL-CONSOLIDATION TEST RESULTS

Job No. 07-020 **FIG.** 18

MOISTURE-DENSITY RELATIONSHIP TEST



Test specification: ASTM D 698-00a Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
0.0-9.0 feet		A-4	8	2.65	39	9	14.0	36

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 123.7 pcf Optimum moisture = 8.8 %	SAND, Silty, Clayey, Gravelly (Crushed Sandstone)

Project No. 07-020 **Client:**

Project: BEAR CLAW III STEAMBOAT SPRINGS

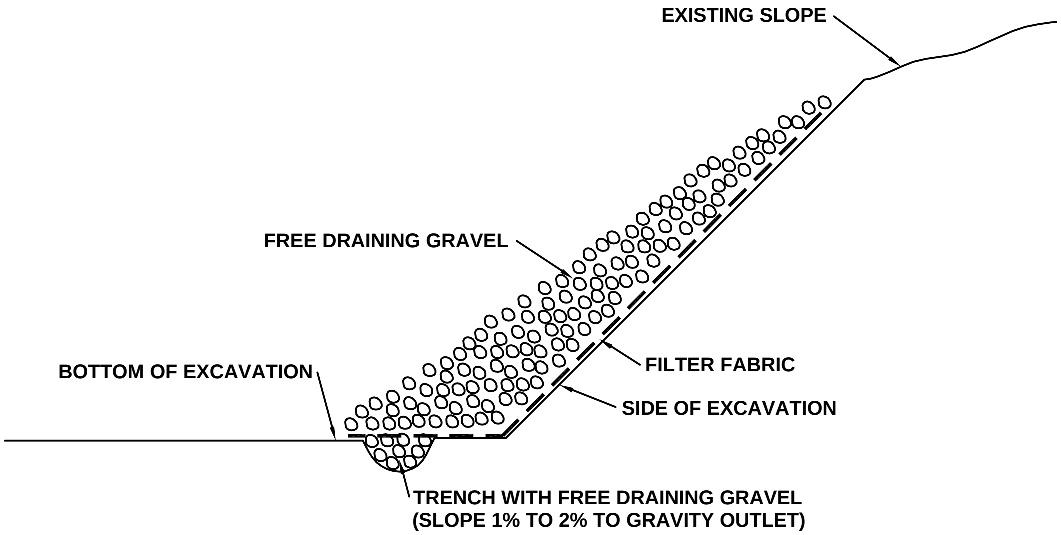
● **Source:** TH-14 (00-074) **Elev./Depth:** 0.0-9.0 feet

Remarks:

MOISTURE-DENSITY RELATIONSHIP TEST

KOECHLEIN CONSULTING ENGINEERS

FIG. 19



NOT TO SCALE.

NOTES:

1. HEIGHT OF FILTER FABRIC TO BE DETERMINED IN THE FIELD.

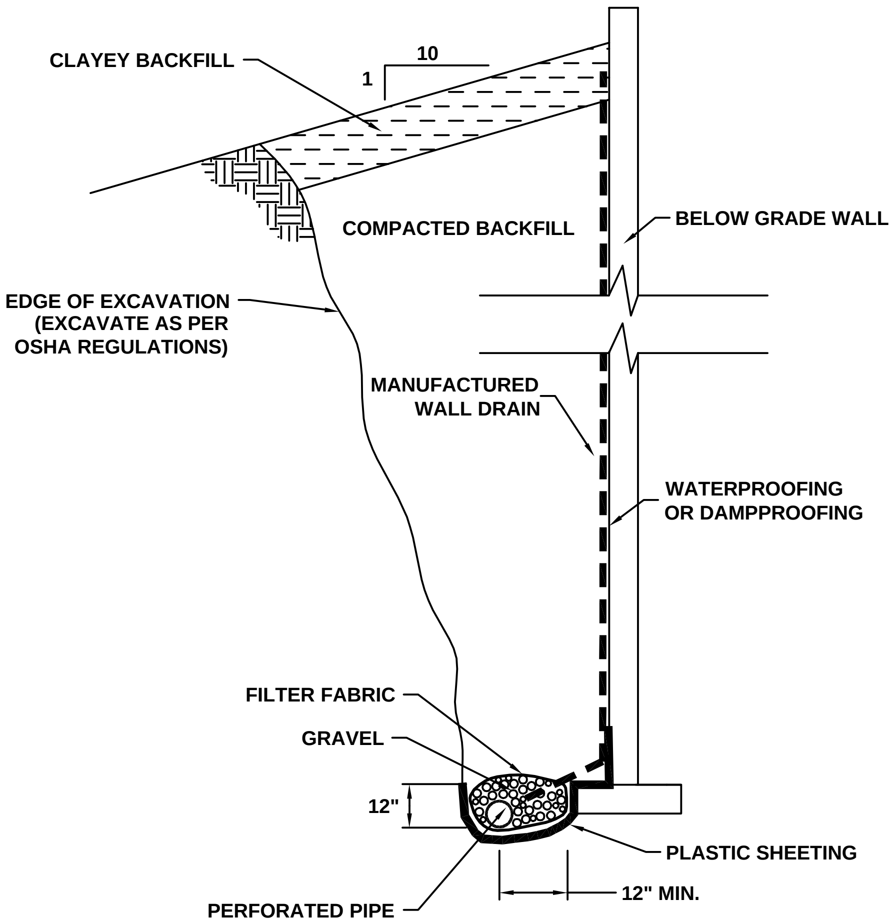
2. TOP OF FILTER FABRIC SHOULD BE A MINIMUM OF 1.0 FOOT ABOVE THE HIGHEST POINT WHERE WATER IS OBSERVED SEEPING INTO THE EXCAVATION.

3. AMOUNT, EXTENT AND CONFIGURATION OF THE FREE DRAINING GRAVEL SHOULD BE DETERMINED IN THE FIELD.

4. A REPRESENTATIVE FROM OUR OFFICE SHOULD OBSERVE THE EXCAVATION SLOPE PRIOR TO STABILIZATION. OUR REPRESENTATIVE CAN ASSIST IN DETERMINING THE EXTENT OF THE FILTER FABRIC AND FREE DRAINING GRAVEL.

5. THIS TECHNIQUE MAY BE USED IN ISOLATED AREAS OF SEEPAGE WHERE EXCESSIVE ERODING OF THE EXCAVATION SLOPES IS OCCURRING.

TYPICAL SLOPE STABILIZATION



NOTES:

1. DRAIN SHOULD BE AT LEAST 12 INCHES BELOW TOP OF FOOTING AT THE HIGHEST POINT AND SLOPE DOWNWARD TO A POSITIVE GRAVITY OUTLET OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPING.

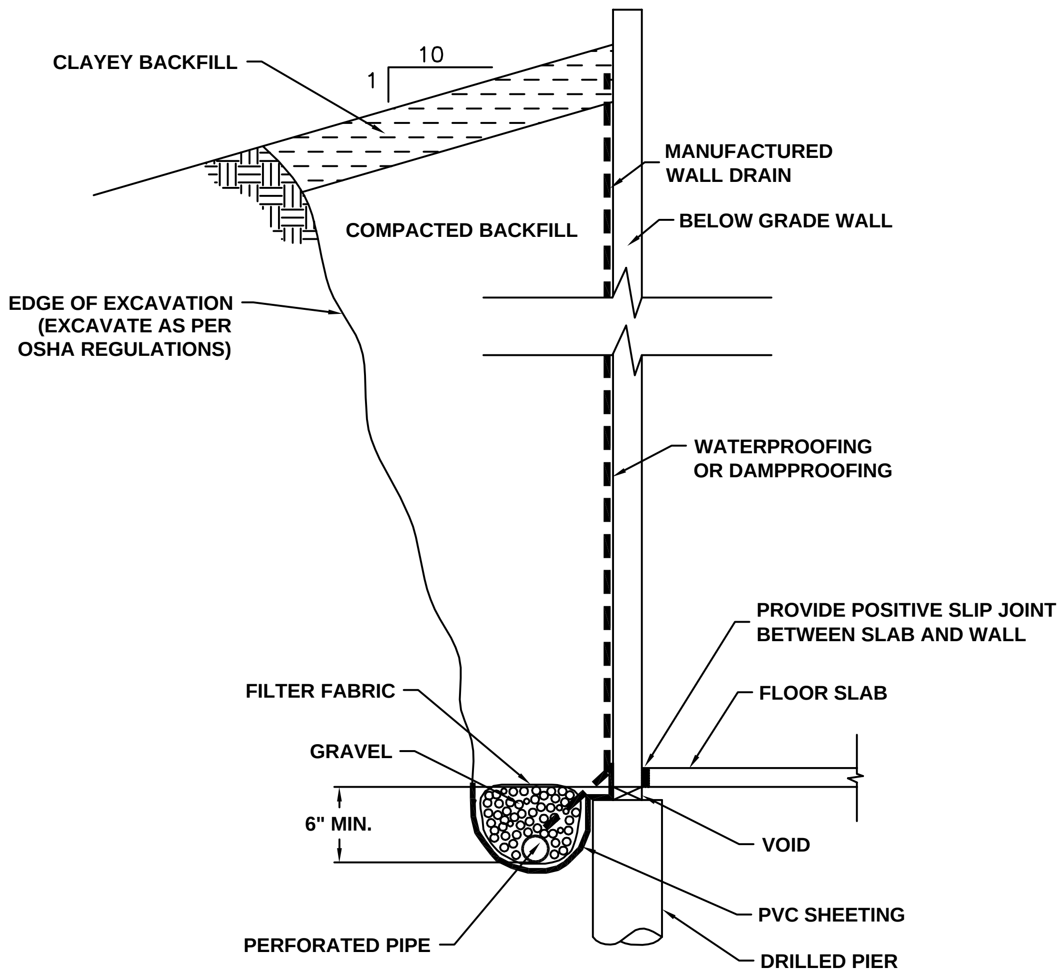
2. EXCAVATIONS ADJACENT TO FOOTINGS SHOULD BE CUT AT A 1 TO 1 (HORIZONTAL TO VERTICAL) OR FLATTER SLOPE FROM THE BOTTOM OF THE FOOTINGS. EXCAVATIONS ADJACENT TO FOOTINGS SHOULD NOT BE CUT VERTICALLY.

3. THE DRAIN SHOULD BE LAID ON A SLOPE RANGING BETWEEN 1/8 INCH AND 1/4 INCH DROP PER FOOT OF DRAIN.

4. GRAVEL SPECIFICATIONS: 1.5 INCH TO NO. 4 GRAVEL WITH LESS THAN 3% PASSING THE NO. 200 SIEVE.

5. THE BELOW GRADE CONCRETE FOUNDATION WALLS SHOULD BE PROTECTED FROM MOISTURE INFILTRATION BY APPLYING A SPRAYED ON MASTIC WATERPROOFING, DAMPPROOFING, OR AN EQUIVALENT PROTECTION METHOD.

TYPICAL SPREAD FOOTING WALL DRAIN DETAIL



NOTES:

1. DRAIN SHOULD BE AT LEAST 6 INCHES BELOW BOTTOM OF SLAB AT THE HIGHEST POINT AND SLOPE DOWNWARD TO A POSITIVE GRAVITY OUTLET OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPING.

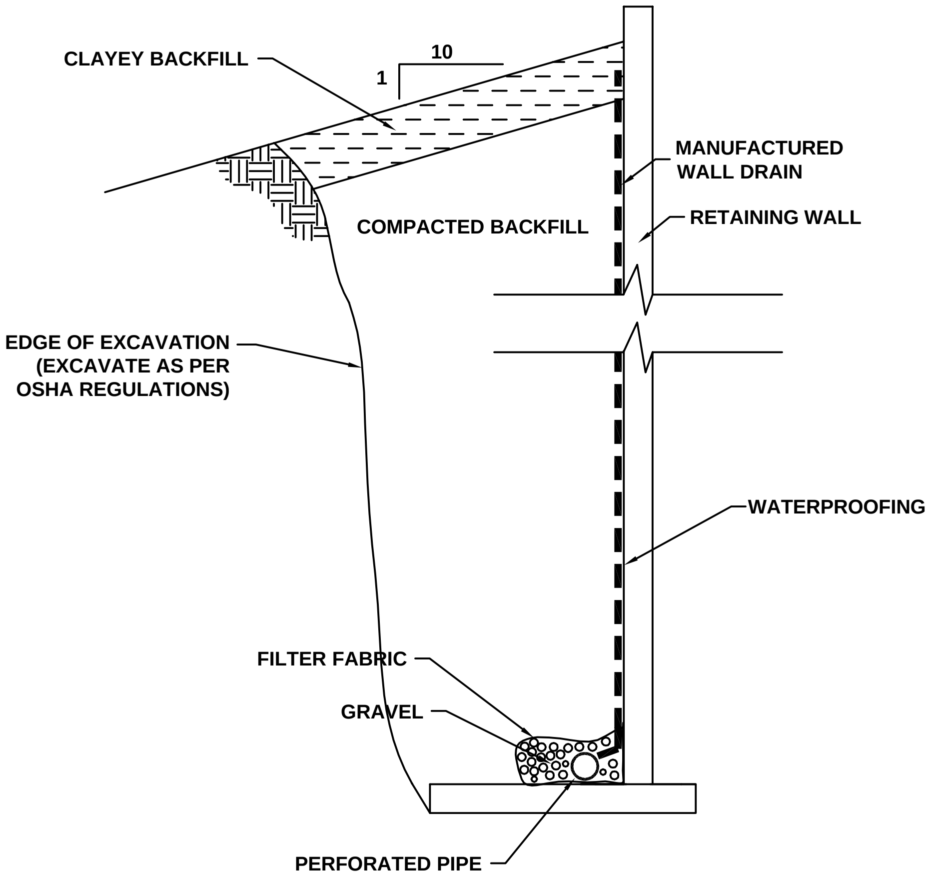
2. THE DRAIN SHOULD BE LAID ON A SLOPE RANGING BETWEEN 1/8 INCH AND 1/4 INCH DROP PER FOOT OF DRAIN.

3. GRAVEL SPECIFICATIONS: 1.5 INCH TO NO. 4 GRAVEL WITH LESS THAN 3% PASSING THE NO. 200 SIEVE.

4. THE BELOW GRADE CONCRETE FOUNDATION WALLS SHOULD BE PROTECTED FROM MOISTURE INFILTRATION BY APPLYING A SPRAYED ON MASTIC WATERPROOFING, DAMPPROOFING, OR AN EQUIVALENT PROTECTION METHOD.

5. PVC SHEETING SHOULD BE GLUED TO FOUNDATION WALL TO PREVENT MOISTURE PENETRATING THROUGH VOID.

TYPICAL DRILLED PIER WALL DRAIN DETAIL



NOTES:

1. DRAIN SHOULD BE SLOPED DOWNWARD TO A POSITIVE GRAVITY OUTLET OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPING.

2. THE DRAIN SHOULD BE LAID ON A SLOPE RANGING BETWEEN 1/8 INCH AND 1/4 INCH DROP PER FOOT OF DRAIN.

3. GRAVEL SPECIFICATIONS: WASHED 1.5 INCH TO NO. 4 GRAVEL WITH LESS THAN 3% PASSING THE NO. 200 SIEVE.

4. THE BELOW GRADE CONCRETE RETAINING WALLS SHOULD BE PROTECTED FROM MOISTURE INFILTRATION BY APPLYING A SPRAYED ON MASTIC WATERPROOFING OR AN EQUIVALENT PROTECTION METHOD.

TYPICAL RETAINING WALL DRAIN DETAIL

SUMMARY OF LABORATORY TEST RESULTS

TABLE 1

HOLE	SAMPLE DEPTH (ft)	NATURAL MOISTURE (%)	NATURAL DRY DENSITY	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTICITY INDEX (%)	PASSING NO. 200 SIEVE (%)	PERCENT SWELL AT 1,000 PSF (%)	pH	SOLUBLE SULFATES (%)	RESISTIVITY (OHM-CM)	DESCRIPTION
TH-1	4.0	12	119			52					CLAYSTONE
TH-1	9.0							8.6			CLAYSTONE
TH-1	13.5	12	122				+1.8				CLAYSTONE
TH-2	17.0	9	126			45					CLAYSTONE
TH-2	29.0								0		CLAYSTONE
TH-3	9.0	16	115				0.0				CLAYSTONE
TH-3	13.0	12	111			58					CLAYSTONE
TH-4	12.0	13	118				+1.4				CLAYSTONE
TH-4	29.0							8.5			CLAYSTONE
TH-5	14.0	14	116			69					CLAYSTONE
TH-6	9.0	15	114				0.0				CLAYSTONE
TH-6	34.0	14	118			58					CLAYSTONE
TH-9	9.0	10	125			23					SANDSTONE
TH-10	14.0	12				50					CLAYSTONE
TH-11	14.0							8.5			SANDSTONE
TH-11	24.0	10	110			44					CLAYSTONE
TH-12	4.0	15	117			35					CLAYSTONE
TH-12	9.0	15	117			44					CLAYSTONE
TH-13	0-4.0	20		44	38	60					SILT, Sandy
TH-14	0-9.0	10		39	9	36		8.3	0	4008	SANDSTONE

Notes:

1. Borings were drilled in May 2000 for Job No. 00-074.

APPENDIX A

PRESSUREMETER TEST RESULTS

REPORT

PRESSUREMETER TESTING BEARCLAW III STEAMBOAT SPRINGS, COLORADO

Prepared for

Koechlein Consulting Engineers
Lakewood, Colorado

May 12, 2000

URS Greiner Woodward Clyde

URS Greiner Woodward Clyde
4582 South Ulster Street, Suite 1000
Denver, CO 80237

Project No. 6800044521.00

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This report presents the results of 10 pressuremeter tests conducted in 5 drill holes (TH1 to TH5) located at the proposed Bearclaw III site in Steamboat Springs, Colorado. The pressuremeter shown in Figure 1 is a drill hole deformation device used to measure the stress-strain characteristics of soil or rock in situ. The pressuremeter tests were conducted under ambient stress, density, structural and moisture content conditions to provide a realistic measure of the properties of the rock under in situ conditions. Characterization of engineering properties from the pressuremeter test results include the in situ lateral stress, stress history, elastic deformation modulus, shear strength, cohesion, and angle of internal friction.

The objective of the pressuremeter testing program was to characterize the in situ engineering properties of the interbedded clayey sandstone and claystone bedrock encountered in drill holes TH1 to TH5, located within the footprint of the proposed Bearclaw III condominium building for use in design of the foundation. The proposed construction includes retaining walls and drilled pier foundations. Two tests were conducted in each drill hole, with an upper test located near the top of bedrock (depths of 11.5 to 16.5 feet) and a lower test in the bedrock (depths of about 30 to 33 feet). Tests in the upper portions of the drill holes were conducted to provide information about bedrock in areas to be excavated and to contain retaining walls. Tests at lower portions of the drill holes corresponded to the expected locations of the bottom of drilled piers.

The pressuremeter test results and interpretations of the engineering properties are presented and discussed in this report.

The drill holes were drilled by Ager Drilling Inc. using a CME 45 track mounted drill rig. Drill holes were advanced to test depths using 4-inch solid flight augers. Pressuremeter tests were conducted in 4-foot long drill holes prepared using a 3-inch solid flight auger. Pressuremeter test depths, shown on Table 1, represent the center of the approximately 15-inch long probe.

Pressuremeter tests were performed using the WCC Menard GAm pressuremeter system. As shown by Figure 1, the pressuremeter apparatus consists of a probe, volume measurement and pressure-control instrument, and nitrogen gas-supply tank. The 70-mm (N size) or 3-inch diameter cylindrical probes contain an expandable rubber membrane (measuring cell) and two contiguous, independently expandable guard cells. The measurement system and the measurement cell were filled with water and the volume change induced in the measuring cell was measured using the sight tube volumeter. Expansion of the measuring cell was controlled by applying gas pressure from the gas-supply tank to the fluid column through a pressure regulator system. The maximum pressure capacity of the pressuremeter system is 100 t/ft^2. The pressure-control system contains a fluid pressure regulator and a differential pressure regulator. This differential pressure regulator maintains a constant differential pressure of 1 t/ft^2 between the fluid pressure in the measuring cell and the gas pressure in the guard cell at all times during the test. Pressure gages measured the pressure in both the guard cell and the measuring cell.

Immediately after the pressuremeter drill hole was prepared, the probe was lowered to the designated testing depth. The test was performed by expanding the probe in equal pressure increments and measuring the change in volume of the measuring cell with time for each increment. For a given pressure, volume readings were taken at 15, 30, and 60 seconds for the standard short-term test in an attempt to model undrained conditions.

An unload-reload cycle was performed in each test, typically at the initiation of yield and plastic deformation. The pressure was increased in equal increments until the pressure or volume change capacity of the system was reached.

The pressure and volume change measurements obtained during the pressuremeter tests were corrected for inertia of the probe, expansion of the measurement system, and the hydraulic pressure of the column of fluid between the instrument and the measuring cell. The measuring system expansion was calibrated by testing at various probe pressures in a rigid steel casing, and the appropriate volume change due to system expansion was deducted from the volume measurements. The inertia of the measuring system was calibrated in open air by determining the pressure necessary to expand the measuring cell membrane to specific volumes without any external resistance. The inertia-correction pressure corresponding to each volume reading was subtracted from each test pressure, and the hydrostatic pressure of the column of fluid between the instrument and the probe was added to the gage pressure.

Pressuremeter test data were reduced on a personal computer using the WCC "PMT" computer program. The test curves were computer generated, and the computer aided in calculation of engineering properties interpreted from the test curves.

3.1 PRESSUREMETER TEST CURVES

The results of the pressuremeter tests are presented as a plot of probe volume change versus pressure. A typical pressuremeter test plot is shown in Figure 2. The pressure-volume change curve is typically made up of three components: (1) the initial reloading portion - as the probe expands through the drill hole, meets the drill hole walls and restresses the soil back to its in situ condition; (2) the linear pseudo-elastic portion; and (3) the plastic portion where the soil exhibits substantial nonlinear deformation. An unload-reload cycle is also typically included in the pressuremeter test. The pressure at the initiation of plastic deformation is termed the creep pressure (P_f); whereas, the asymptotical axis of the plastic deformation curve is interpreted as the limit pressure (P_L). The pressure at the inception of the pseudo-elastic response is generally interpreted as the in situ horizontal stress (P_o). The slope of the linear segment of the curve is used to calculate the initial modulus (E); whereas, the reload slope of an unload-reload test cycle is used to calculate the reload modulus (E+). Pressuremeter test curves are presented in Appendix A.

The pressuremeter test curves exhibit initial recompression and have a satisfactory hole size which allows the linear elastic and plastic portions of the curves to be well developed up to the pressure or volume change limits of the system. Two of the tests (TH2 at 30.5 feet and TH3 at 32.0 feet) were conducted in what was interpreted to be interbedded hard and soft rock. The probe developed a leak before the yield pressure was reached in these tests. The yield pressure and reload modulus were estimated for each of these tests, and are based on the initial moduli measured before the probe leak developed. The volume capacity of the probe was reached shortly after the creep pressure was reached.

Geotechnical parameters interpreted from the pressuremeter test curves are summarized in Table 1. A glossary of property symbols used in the table is given in Appendix B. These property interpretations are discussed below.

3.2 IN SITU STRESS STATE

The in situ horizontal total stress σ_{ho} was determined from the pressuremeter results as the stress (P_o) corresponding to the initiation of linear elastic response following the procedure discussed by Davidson (1979). Unavoidable drill hole wall disturbance generally makes the interpretation of P_o less reliable than other parameters. However, these problems do not appear to have a significant impact on the pressuremeter test interpretations for this project. Interpreted in situ horizontal stress parameter values are presented in Table 1.

The coefficient of earth pressure at rest (K_o) can be calculated based on effective stresses:

$$K_o = \sigma'_{ho} / \sigma'_{vo}$$

where:

σ'_{ho} = σ_{ho} - u, in situ horizontal effective stress

σ'_{vo} = σ_{vo} - u, vertical effective overburden stress

u = ambient pore pressure

σ_{ho} = P_o = in situ horizontal total stress

The vertical overburden stress was calculated using total unit weight for the bedrock of 150 pcf for the bedrock, and the ambient pore pressures were assumed to be hydrostatic below the groundwater table measured at the time of drilling. Calculated values of K_o ranged from 0.7 to 4.2.

3.3 STRESS HISTORY

Stress history is characterized by the overconsolidation ratio (OCR), which is the ratio of the maximum past vertical effective stress to the present vertical effective stress. A horizontal overconsolidation ratio can be inferred from pressuremeter test results using creep pressure:

$$OCR = \sigma'_{hmax} / \sigma'_{ho}$$

where: σ'_{hmax} = σ_{hmax} - u, maximum past effective horizontal stress

σ_{hmax} = P_c or P_f = pressuremeter creep pressure

The creep pressure corresponds to the initiation of yield after the linear pseudo-elastic portion of the test curve.

Based on our experience, this procedure has been found to be reliable for overconsolidated clays, medium dense to dense sands, residual soils and weathered rock. Comparisons have been made between pressuremeter derived OCRs and those determined from known construction or stress history induced levels of overconsolidation and consolidation test results.

Horizontal overconsolidation ratios determined from pressuremeter test results are presented in Table 1. The OCR measurements ranged from 6.7 to 23.7 in the bedrock.

3.4 ELASTIC DEFORMATION MODULUS

Several elastic deformation moduli can be calculated from linear portions of the pressuremeter test curves, including an initial modulus E, a re-load modulus E+, and an unload modulus E-. The initial modulus is determined from the slope of the "pseudo elastic" portion of the curve using:

$$E = 2\,(1+v)\,V_o \quad \Delta P / \Delta V$$

where: V_o = $V_i + \Delta V_o$

v = Poissons ratio;

ΔV_o = volume change corresponding to P_o;

$\frac{\Delta P}{\Delta V}$ = linear slope of test curve; and

V_i = initial volume of probe

The reload and unload moduli are calculated using the preceding equation with the slope of the reload and unload portions of cycled pressuremeter curves. Pressuremeter initial, reload, and unload modulus parameter values are presented in Table 1.

Pressuremeter experience in clay, glacial till, residual soil and weathered rock has shown the initial pressuremeter modulus is generally not equivalent to the initial tangent modulus measured in a good quality consolidated triaxial compression test run under similar strain rate and drainage conditions as the pressuremeter test. Hole disturbance, even in the most carefully prepared drill hole, and yielding during drill hole unloading influence the pressuremeter-measured initial modulus. The pressuremeter initial modulus typically falls somewhere between the secant modulus at 50 percent of the peak stress and secant modulus at failure from a good quality triaxial test, depending on the level of drill hole disturbance. However, excellent correlation between pressuremeter reload modulus, good quality triaxial initial tangent modulus, and low strain elastic modulus back-calculated from foundation performance has been observed in many types of soil and soft rock (Davidson and Bodine, 1986; Denby et al., 1981). Unloading and reloading at the initiation of yield erases the effects of hole disturbance and a low strain response of the undisturbed soil is measured. The reload moduli ranged from 770 to 9500 tsf.

3.5 UNDRAINED SHEAR STRENGTH

Since the pressuremeter test is a total stress test, undrained shear strength can be determined from the plastic failure portion of the pressuremeter curve. Most of the tests reached yield. Ultimate failure was reached in two of the tests conducted, as the tests were carried up to the volume capacity of the probe.

To calculate undrained shear strength, S_u, the Gibson and Anderson (1961) procedure was used:

$$P_L - P_o = S_u \ (1+\ln (E/2\ S_u (1+v)))$$

If the factor in parenthesis is defined β as then:

$$S_u = P_L - P_o / \beta$$

These equations were solved by estimating β in the second equation and then iterating to find the solution to the first equation. Based on empirical data, the reload modulus should be utilized for E in the equation.

Interpretations of undrained shear strength S_u are presented in Table 1. The undrained shear strength S_u values determined from the pressuremeter tests ranged from 2.4 to 18.2 tsf.

3.6 ANGLE OF INTERNAL FRICTION AND COHESION

The drained shear strength can be characterized from the failure portion of pressuremeter tests. Although the test induces pore pressure of unknown magnitude, it has been assumed that the stress increment duration was long enough for excess pore pressure to dissipate in the rock because the volume change had stabilized for each pressure increment. Furthermore, at the ultimate failure condition, we have assumed that the rock deforms without further volume change or change in stress and the rock has reached the critical void ratio. This critical state condition is assumed in the calculation of drained shear strength.

Methods are available to estimate both the cohesion intercept and friction angle under drained conditions. The drained angle of internal friction φ' was estimated for the portion of the test curve where unlimited plastic yielding is occurring at the ultimate stress (near P_L) using the

Hughes et. al (1977) procedure. It has been assumed that at this large strain, cohesive bonds have been broken ($c' = 0$) and only the frictional component of the shear strength is being mobilized (ϕ'). Using critical state soil mechanics concepts, the drained friction angle was calculated using:

$$\sin\phi' = S(K+1)/(S(K-1)+2)$$

where: S = $\tan\theta$;

θ = slope of effective pressure - radial strain plot at ultimate failure;

K = $\tan^2(45° + \phi'_{cv}/2)$; and

ϕ'_{cv} = critical state friction angle.

The peak cohesion intercept c' can also be calculated from the initial yield portion of the test curve (near P_f) using the same ϕ', and an extension of the above procedure developed by Bachus et. al (1982):

$$c' = \sigma_r \tfrac{1}{2}[\tan(45° - \phi'/2) - \tan(45° + \phi'/2)((1-S)/KS+1)]$$

where: σ_r = average radial stress along the yield portion of the curve.

ϕ' = slope of effective pressure - radial strain plot during initial yield.

K,S = as defined previously

At this point of initial yield in the pressuremeter expansion curve, it is assumed that the rock has mobilized its peak shear strength (c', ϕ').

Interpretations of drained friction angle ϕ' and peak cohesion c' from two of the pressuremeter tests conducted (TH1 at 29.8 feet, and TH4 at 12.0 feet) are presented in Table 1. In the other tests the ultimate failure portion of the curves was not fully developed so a friction angle could not be estimated. The interpreted friction angles ranged from 28° to 32° and the cohesion ranged from 4.7 to 5.7 tsf.

The pressuremeter testing program conducted for this project and presented in this report had the intent of providing engineering properties of the rock encountered at the site for consideration along with similar types of data from standard penetration test results and laboratory testing. The results of these tests are for the specific materials and locations tested and are not to be construed to be representative of the entire geologic unit present at the site. Variations in engineering properties and differences in conditions are often encountered within each geologic unit. The data presented in this report were collected to help develop subsurface characterizations for this project.

URS Greiner Woodward Clyde Consultants warrants that our services are performed within the limits prescribed by our clients, with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended in our proposals, contracts, or reports.

The pressuremeter testing program was conducted and the data was reduced and interpreted by Mr. Dale Baures. Mr. Jim Scott reviewed this report.

Report Prepared and Reviewed By:

Dale Baures, P.E, P.G..
Project Manager

James C. Scott, P.E.
Senior Consultant

Bachus, R.C., Clough, G.W., Sitar, N.C., Shafir-Rad, N., Crosby, N. and Kaboli, P. 1982. "Behavior of Weekly Cemented Soil Slopes Under Static and Seismic Loading", Vol. II, John A. Blume Earthquake Engineering Center, Stanford University, California.

Brooker, E.W., and Ireland, H.O. 1965. "Earth Pressures at Rest Related to Stress History", Canadian Geotechnical Journal, 2, No. 1.

Davidson, R.R. 1979. "Interpretation of the Pressuremeter Test," Woodward-Clyde Consultants Technical Memorandum No. 104-78, December.

Davidson, R.R. and Bodine, D.G. 1986. "Analysis and Verification of Louisiana Pile Foundation Design Based on Pressuremeter Results", The Pressuremeter and Its Marine Applications: Second International Symposium, ASTM STP 950.

Denby, G.M., Aboim Costa, C., Clough, G.W. and Davidson, R.R. 1981. "Laboratory and Pressuremeter Tests on a Stiff Clay", Tenth International Conference on Soil Mechanics and Foundation Engineering, Stockholm, Sweden, July.

Gibson, R.E. and Anderson, W.F. 1961. "In-Situ Measurement of Soil Properties with the Pressuremeter", Civil Engineering and Public Works Review, London, May, 56, No. 658, pp. 615-618.

Hughes, J.M.O., Wroth, C.O. and Windle, D. 1977. "Pressuremeter Tests in Sands," Geotechnique, 27, No. 4, December, pp. 445-477.

TABLE 1

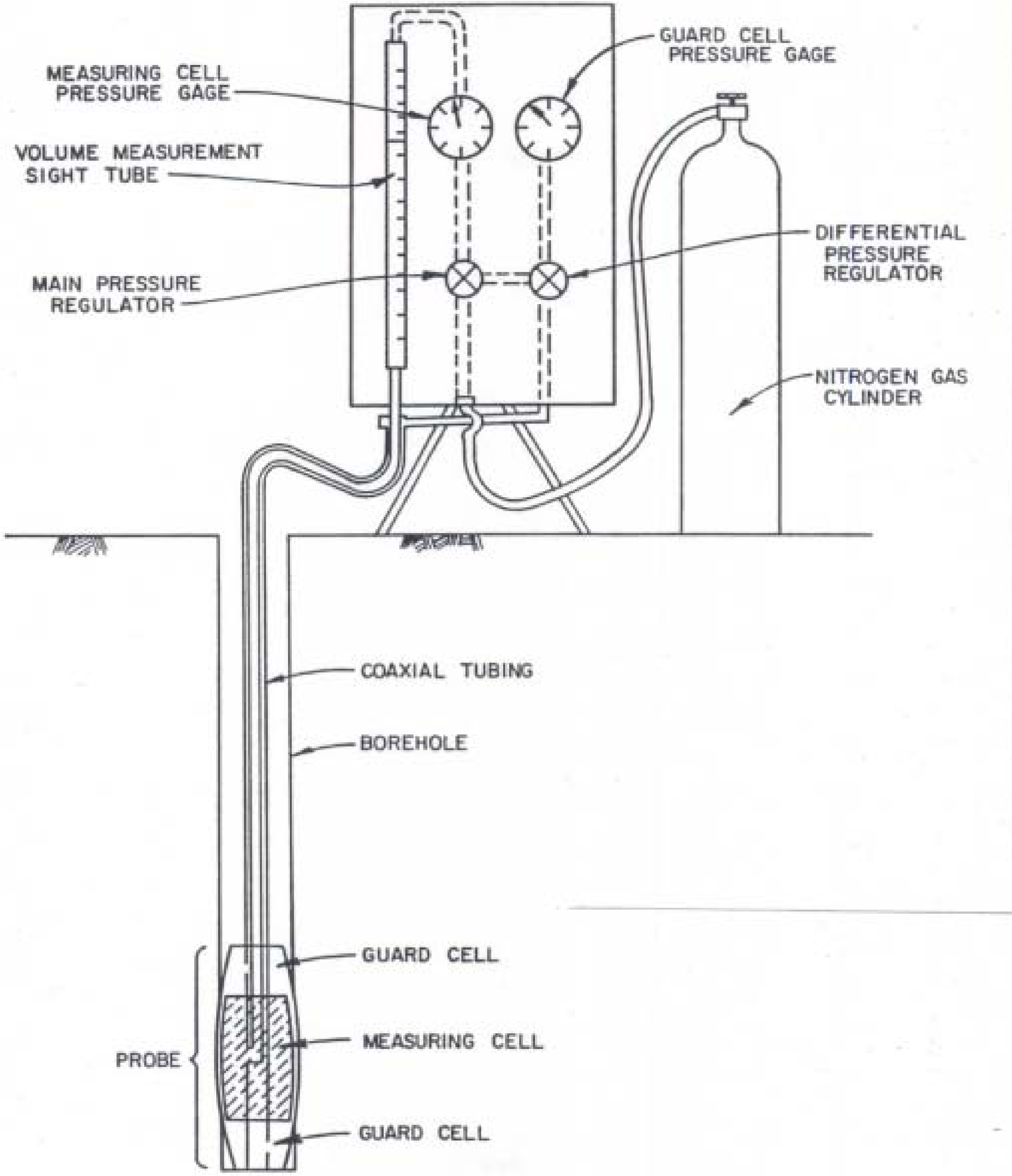
SUMMARY OF PRESSUREMETER TEST RESULTS

PROJECT Bearclaw III **LOCATION** Steamboat Springs, CO **NO.** 6800044521.00 **DATE** 5/12/00

BORING	DEPTH ft	P_O t/ft²	σ'_{ho} t/ft²	σ'_v t/ft²	K_o	P_f t/ft²	OCR	E t/ft²	E+ t/ft²	E- t/ft²	S_u t/ft²	c t/ft²	ϕ °	P_L t/ft²	COMMENTS Blow Count -N	
															Above Test	Below Test
TH1	11.5	3.0	3.0	.86	3.5	38.0	12.7	1800	4800	5700	17.1	-	-	100 *	50/5"	50/5"
TH1	29.8	3.7	3.4	1.9	1.8	34.8	10.2	1300	4100	7400	13.2	5.7	32	80 *	50/1"	50/3"
TH2	16.0	2.8	2.8	1.2	2.3	44.0	15.7	1400	6300	7200	18.2	-	-	110 *	50/6"	50/2"
TH2	30.5	2.3	2.0	2.0	1.0	30 *	14.9	3200	9500 *	-	9.8	-	-	70 *	50/3"	
TH3	11.8	0.8	0.8	0.9	0.9	18.5	23.7	700	2700	4000	6.4	-	-	40 *	50/12"	50/3"
TH3	32.0	2.3	1.9	2.0	1.0	30 *	15.6	2400	7300 *	-	10.2	-	-	70 *	50/0"	50/0"
TH4	12.0	3.8	3.8	0.9	4.2	25.4	6.7	1300	4500	9800	9.0	4.7	28	60 *	50/7"	50/6"
TH4	32.5	2.4	2.0	2.1	1.0	17.1	8.3	700	2700	3600	6.2	-	-	40 *	50/5"	50/6"
TH5	16.5	0.9	0.9	1.2	0.7	6.6	7.8	260	770	1500	2.4	-	-	15 *	50/5"	50/1"
TH5	33.0	2.8	2.1	1.8	1.2	23.3	11.0	1900	3300	4600	9.8	-	-	60 *	50/0"	

* ESTIMATED

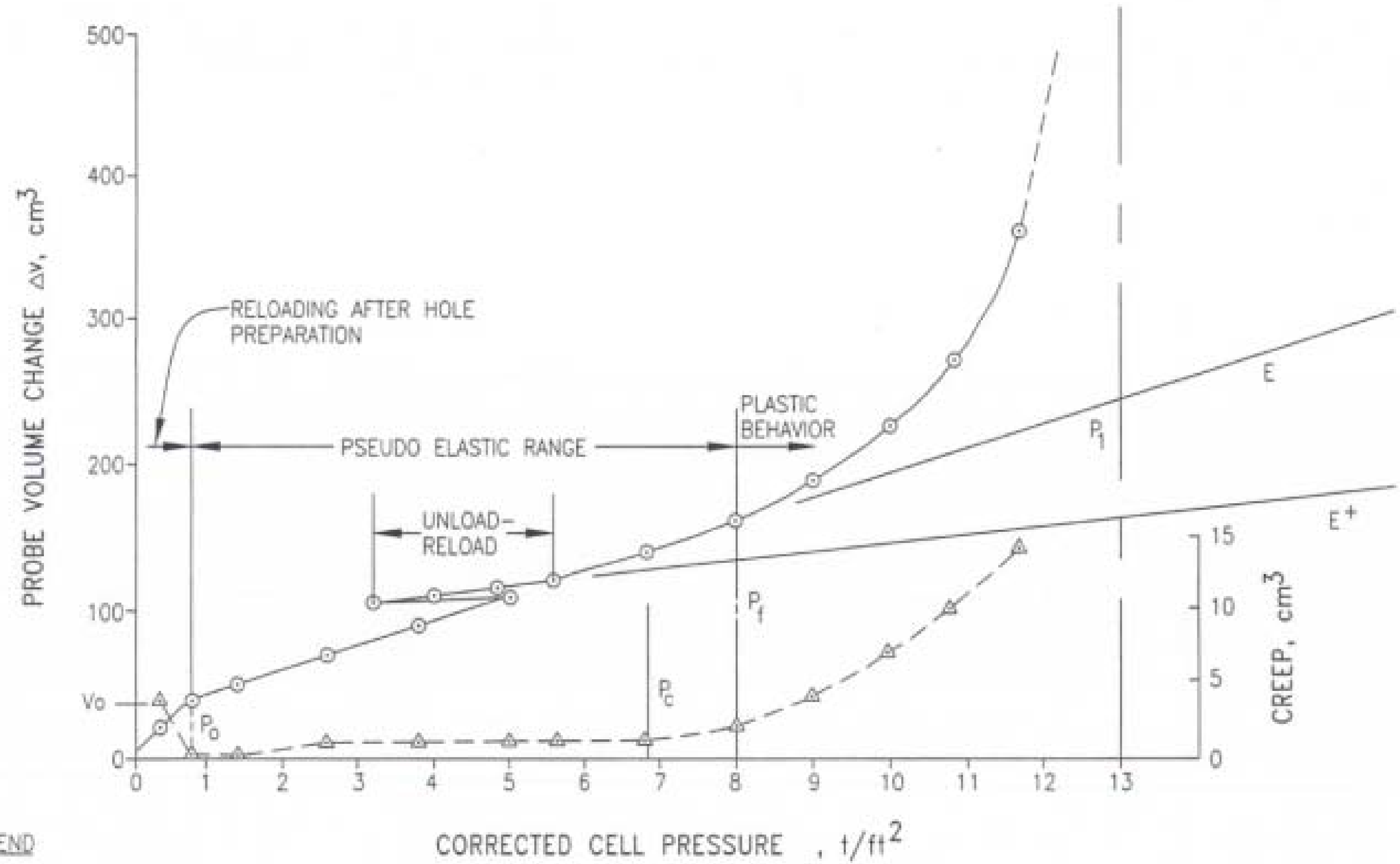
URS Greiner Woodward Clyde



Job No.: 680004452100	MENARD GAM PRESSUREMETER TEST APPARATUS SCHEMATIC
Prepared by: DMB	
Date: 5-10-00	

FIG. 1

Woodward-Clyde International



LEGEND

⊙ PROBE VOLUME CHANGE VS. PRESSURE

△ CREEP VS. PRESSURE

P_o IN SITU HORIZONTAL STRESS

P_f YIELD PRESSURE

P_c CREEP PRESSURE

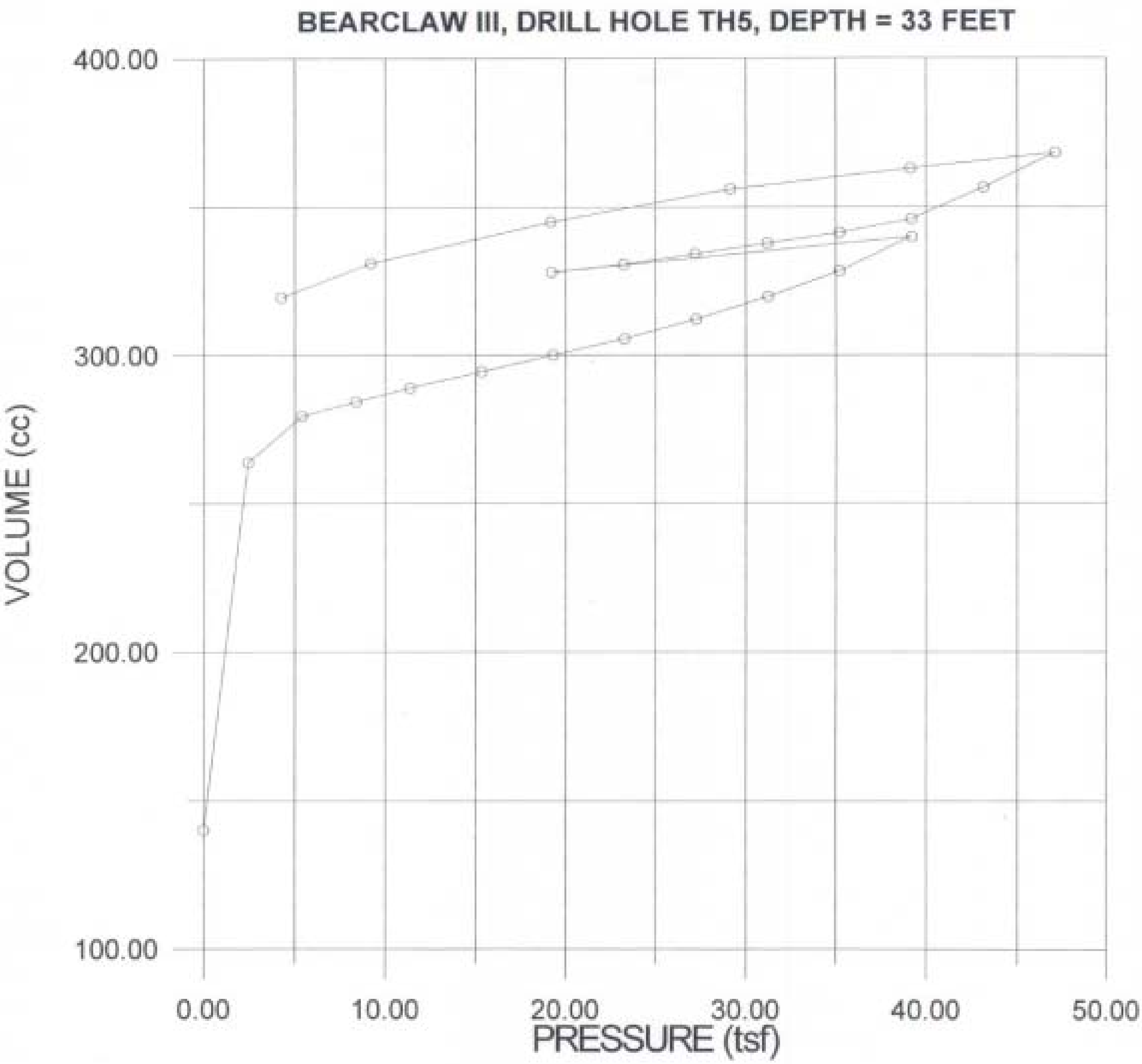
P_1 LIMIT PRESSURE

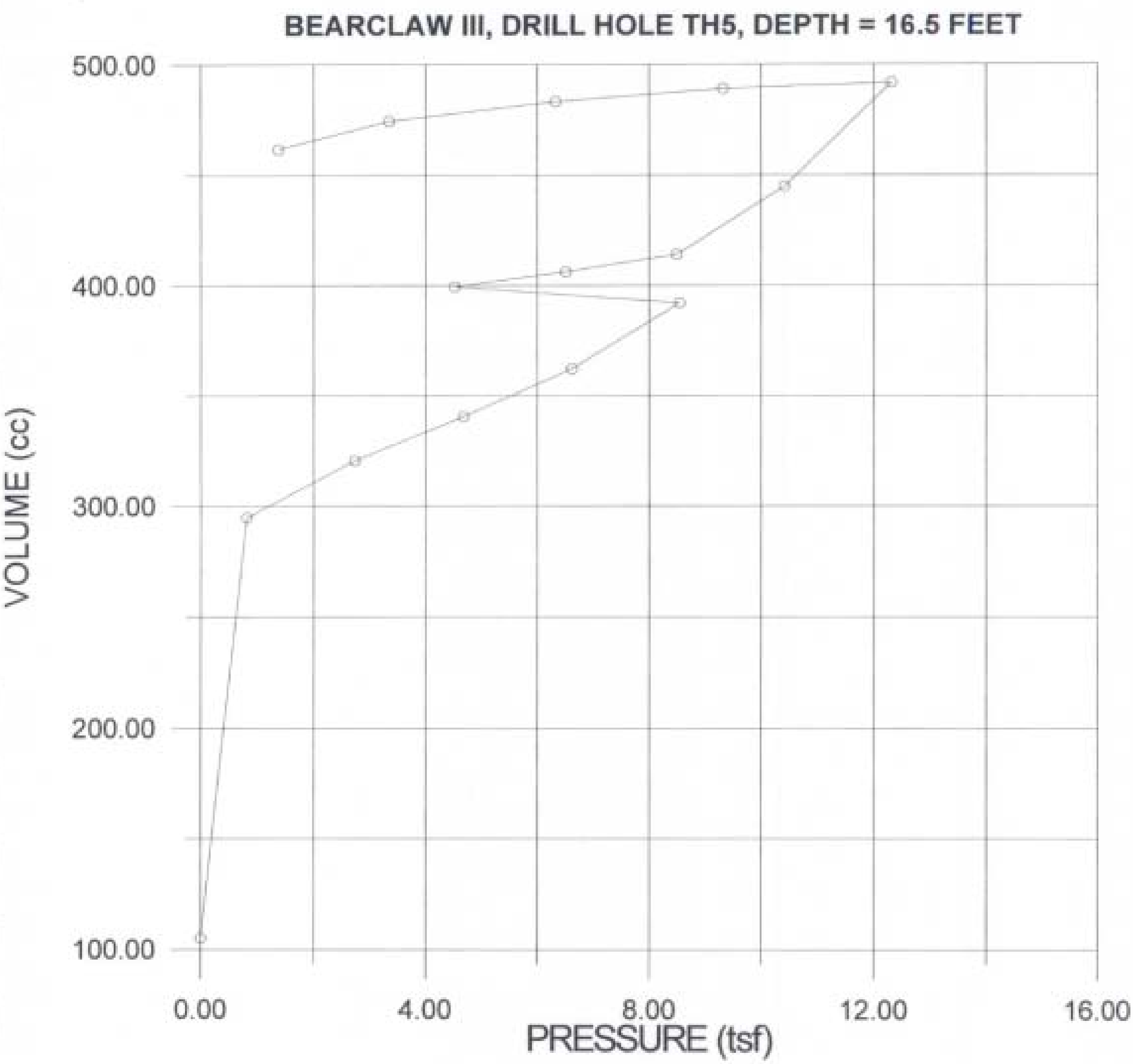
E INITIAL ELASTIC SOIL MODULUS

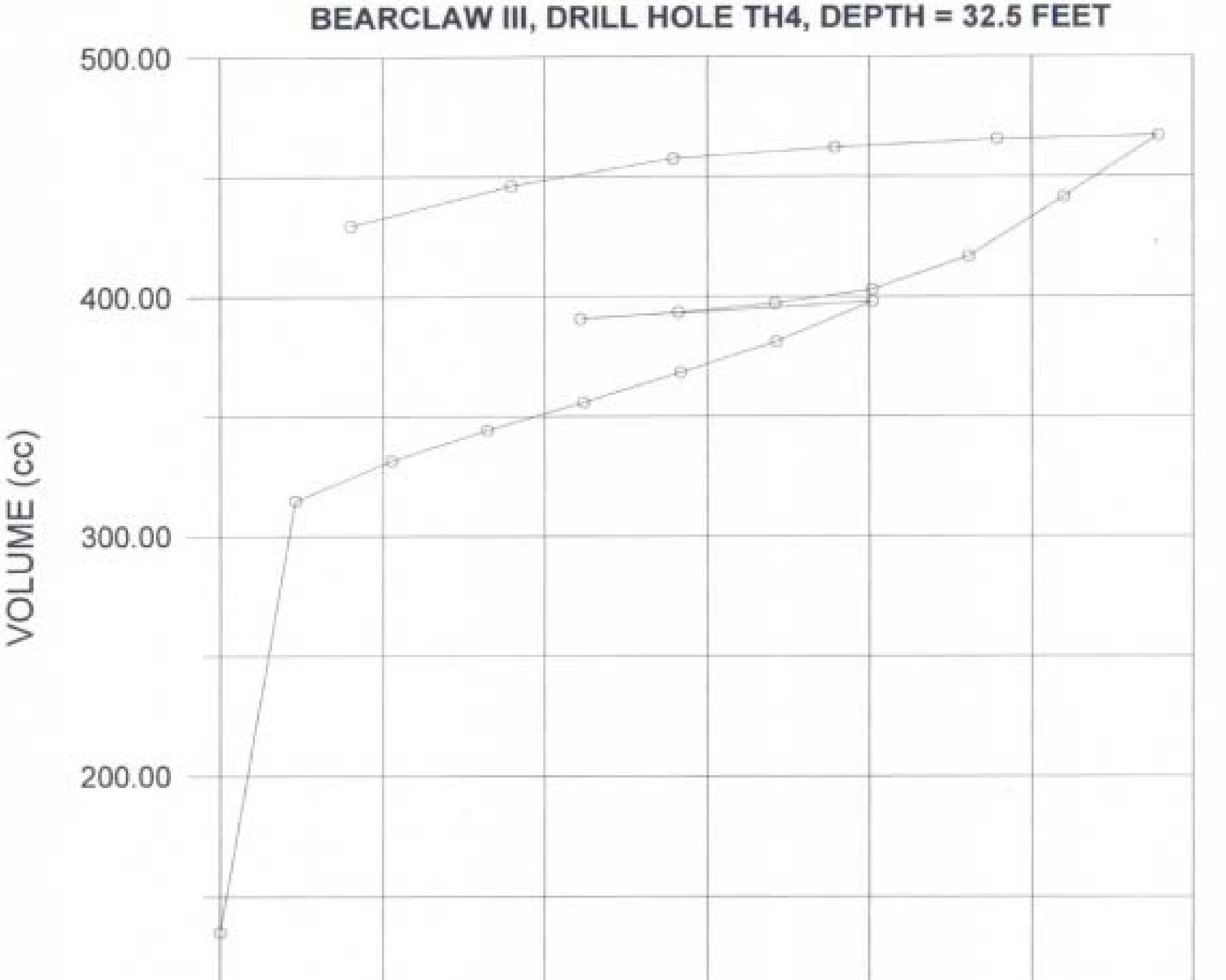
E^+ RELOAD MODULUS

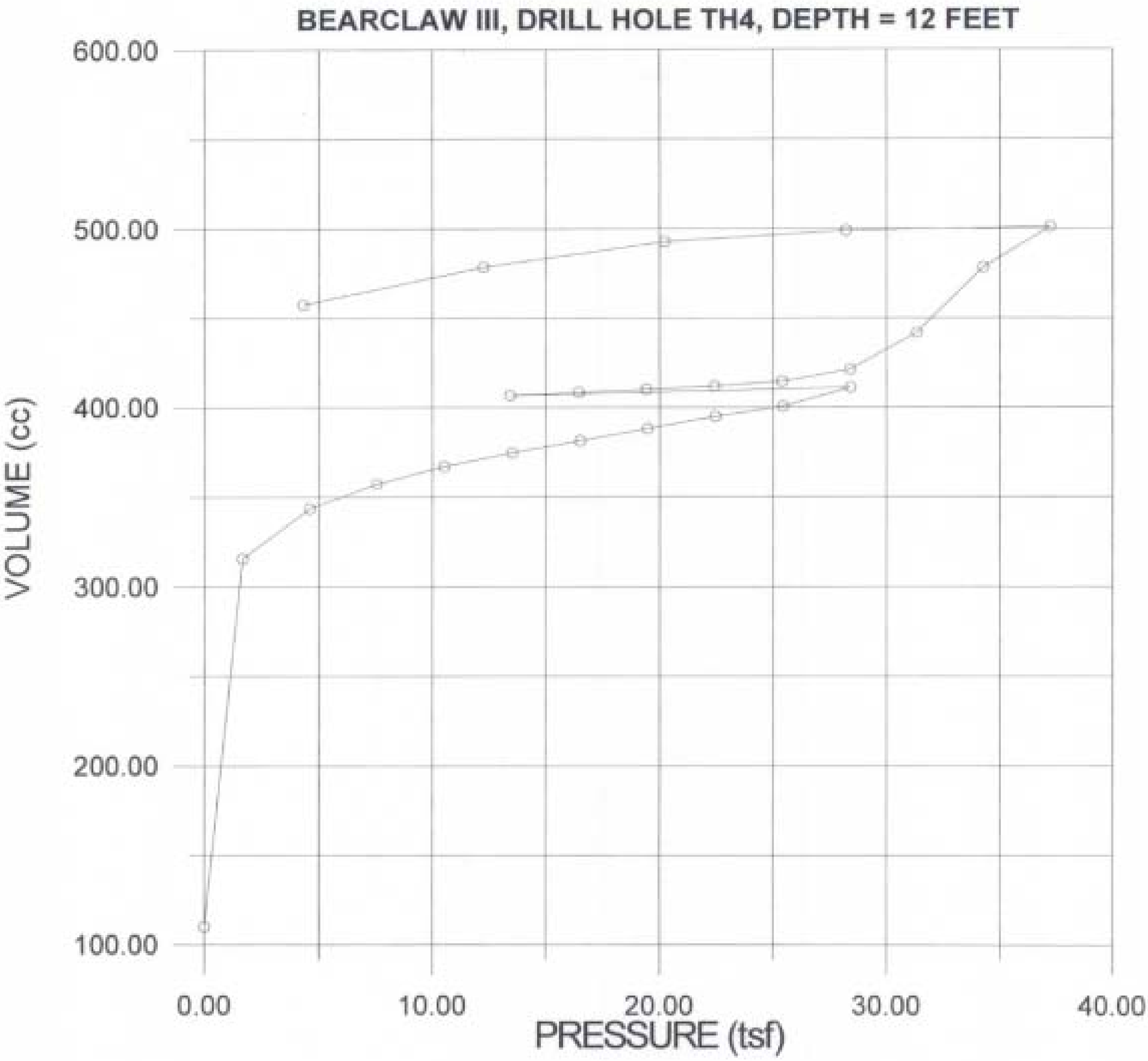
Job No. : 68000445Z100	TYPICAL PRESSUREMETER TEST PLOT
Prepared by DMB	
Date : 5-10-00	

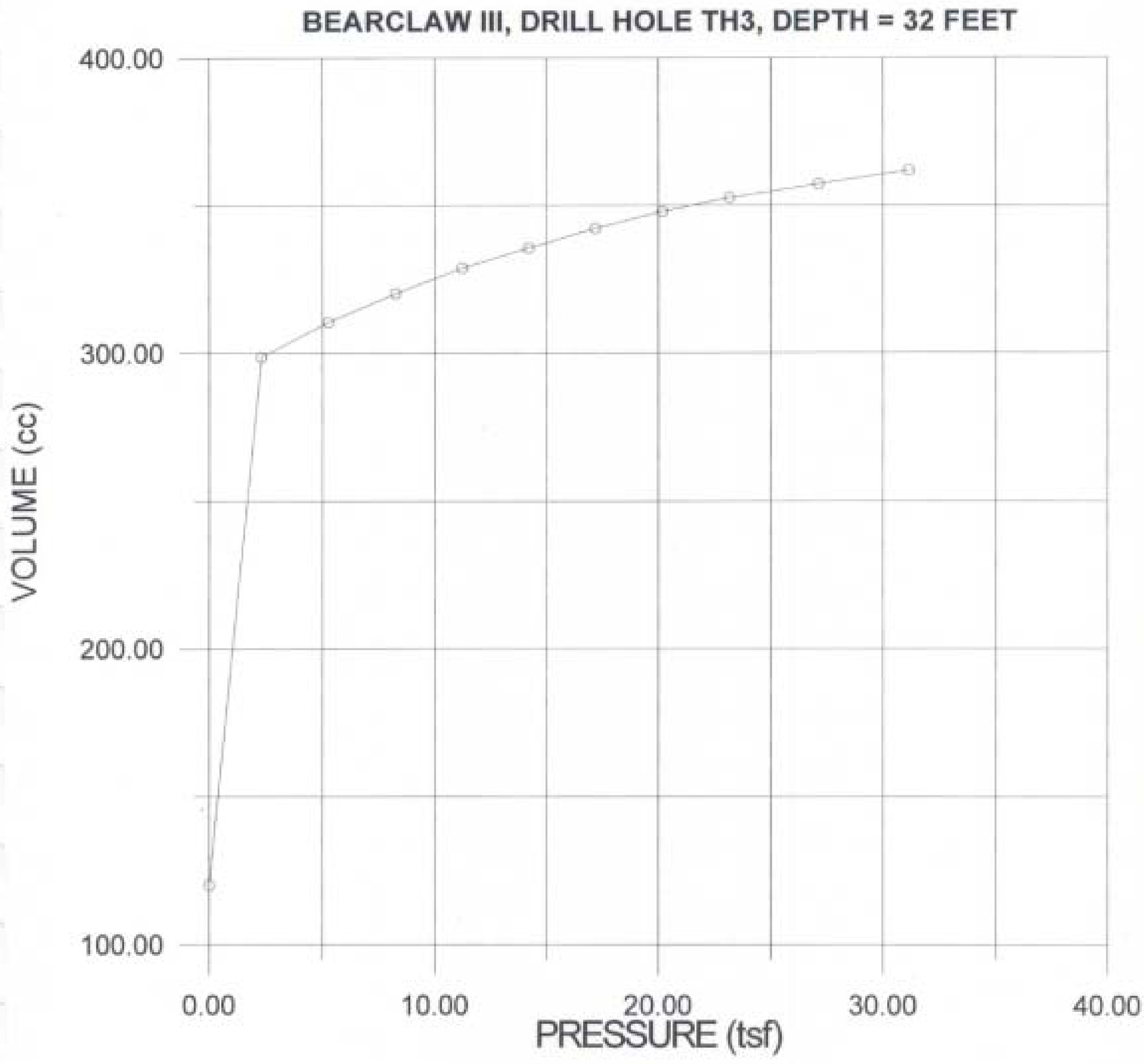
FIG. 2


BEARCLAW III, DRILL HOLE TH5, DEPTH = 33 FEET
400.00
300.00
200.00
100.00
VOLUME (cc)
0.00
10.00
20.00
30.00
40.00
50.00
PRESSURE (tsf)


BEARCLAW III, DRILL HOLE TH5, DEPTH = 16.5 FEET
VOLUME (cc)
500.00
400.00
300.00
200.00
100.00
0.00
4.00
8.00
12.00
16.00
PRESSURE (tsf)


BEARCLAW III, DRILL HOLE TH4, DEPTH = 32.5 FEET
500.00
400.00
300.00
200.00
100.00
VOLUME (cc)
0.00
10.00
20.00
30.00
PRESSURE (tsf)


BEARCLAW III, DRILL HOLE TH4, DEPTH = 12 FEET
VOLUME (cc)
600.00
500.00
400.00
300.00
200.00
100.00
0.00
10.00
20.00
30.00
40.00
PRESSURE (tsf)


BEARCLAW III, DRILL HOLE TH3, DEPTH = 32 FEET
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300.00
200.00
100.00
VOLUME (cc)
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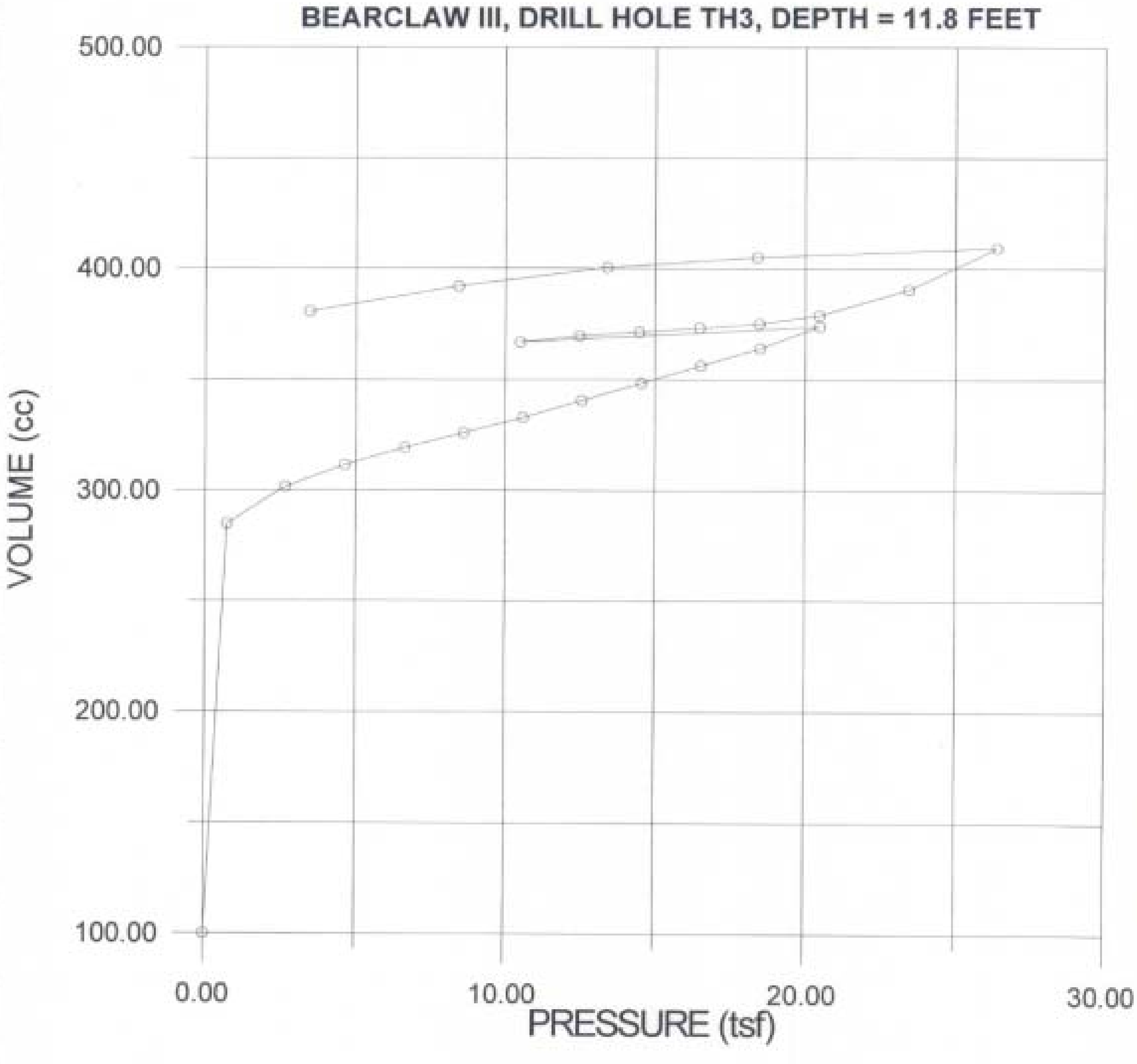
10.00
20.00
30.00
40.00
PRESSURE (tsf)


BEARCLAW III, DRILL HOLE TH3, DEPTH = 11.8 FEET
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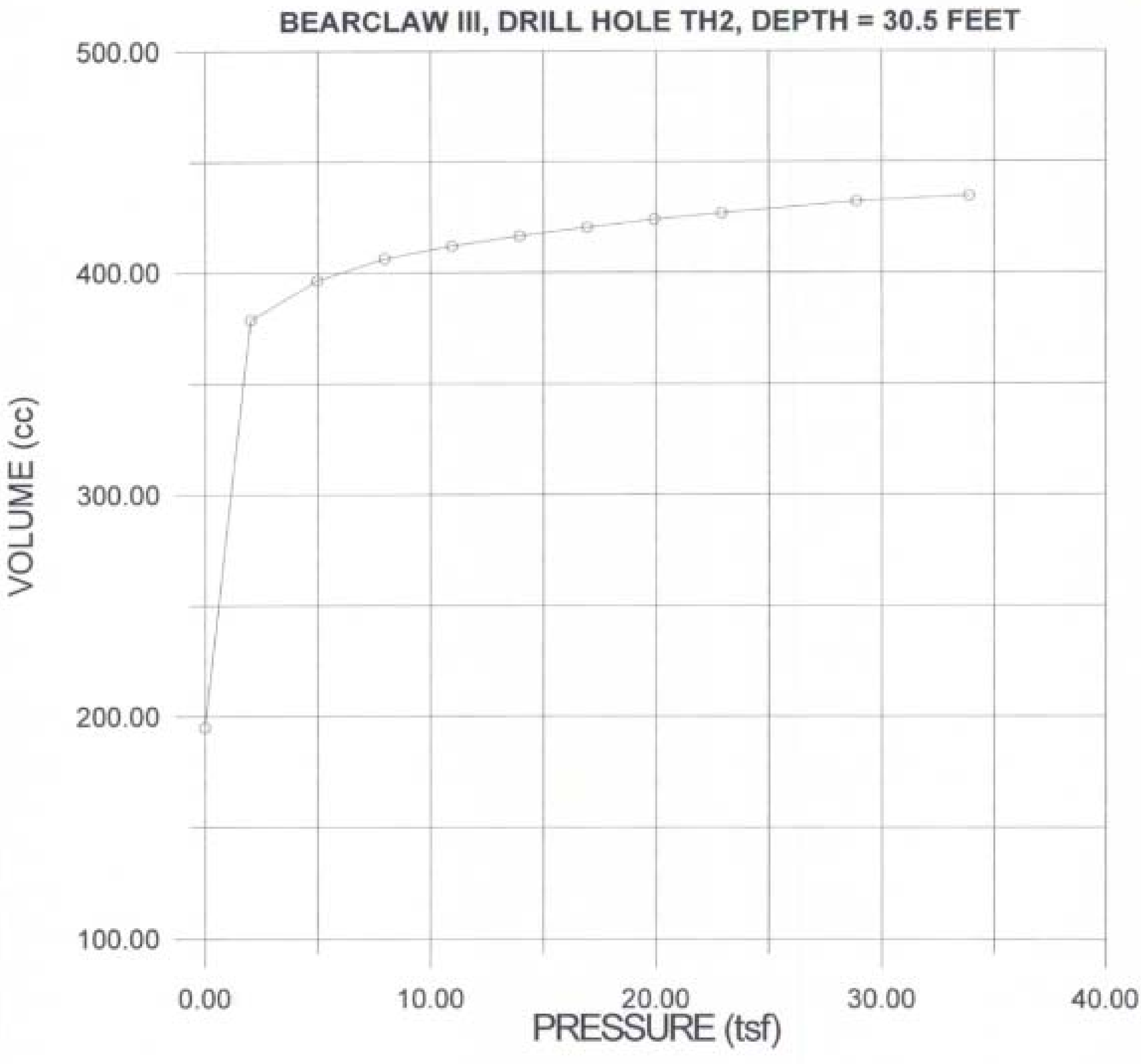
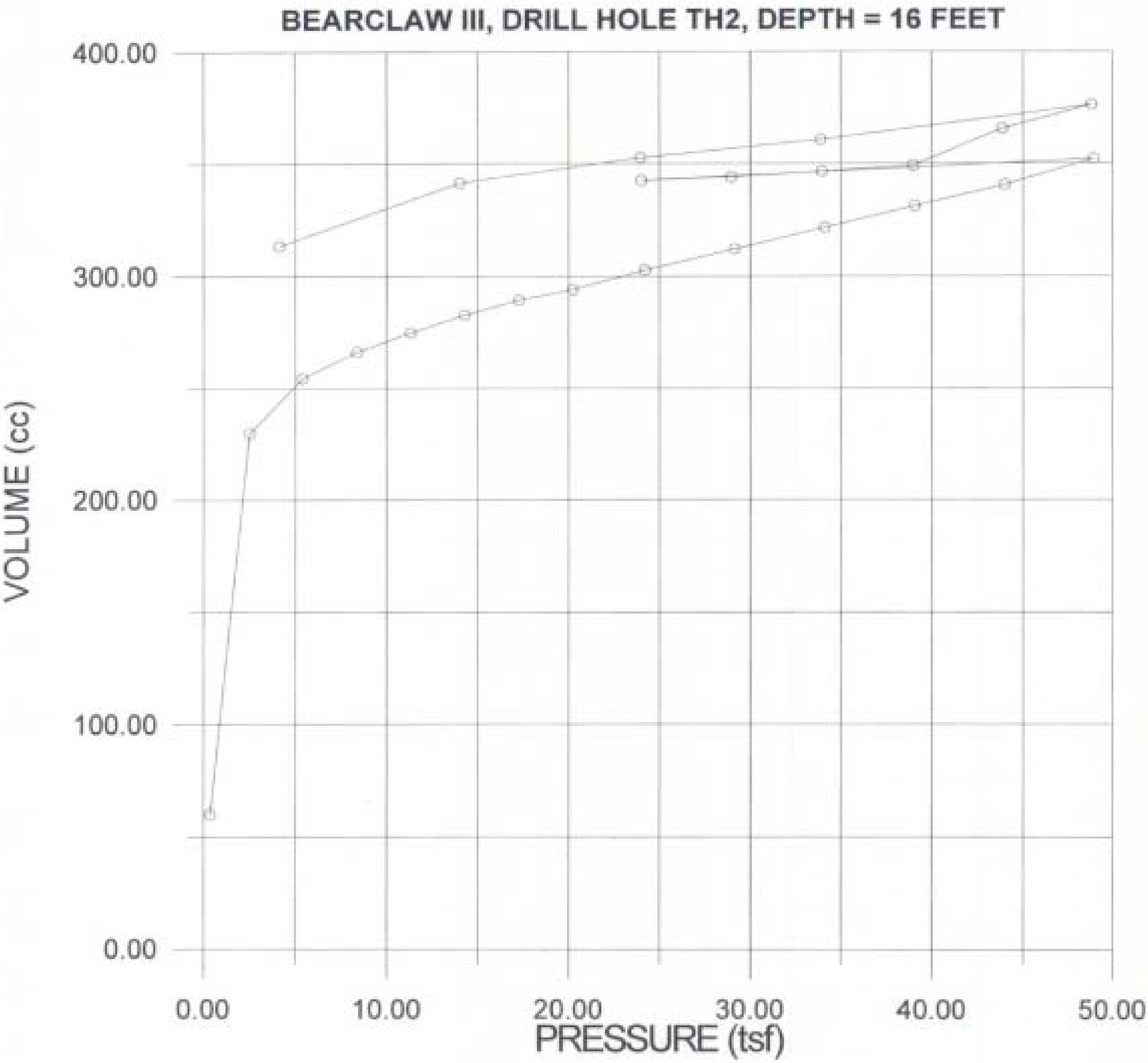
400.00
300.00
200.00
100.00
VOLUME (cc)
0.00
10.00
20.00
30.00
PRESSURE (tsf)


BEARCLAW III, DRILL HOLE TH2, DEPTH = 30.5 FEET
500.00
400.00
300.00
200.00
100.00
VOLUME (cc)
0.00
10.00
20.00
30.00
40.00
PRESSURE (tsf)


BEARCLAW III, DRILL HOLE TH2, DEPTH = 16 FEET
VOLUME (cc)
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200.00
100.00
0.00
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30.00
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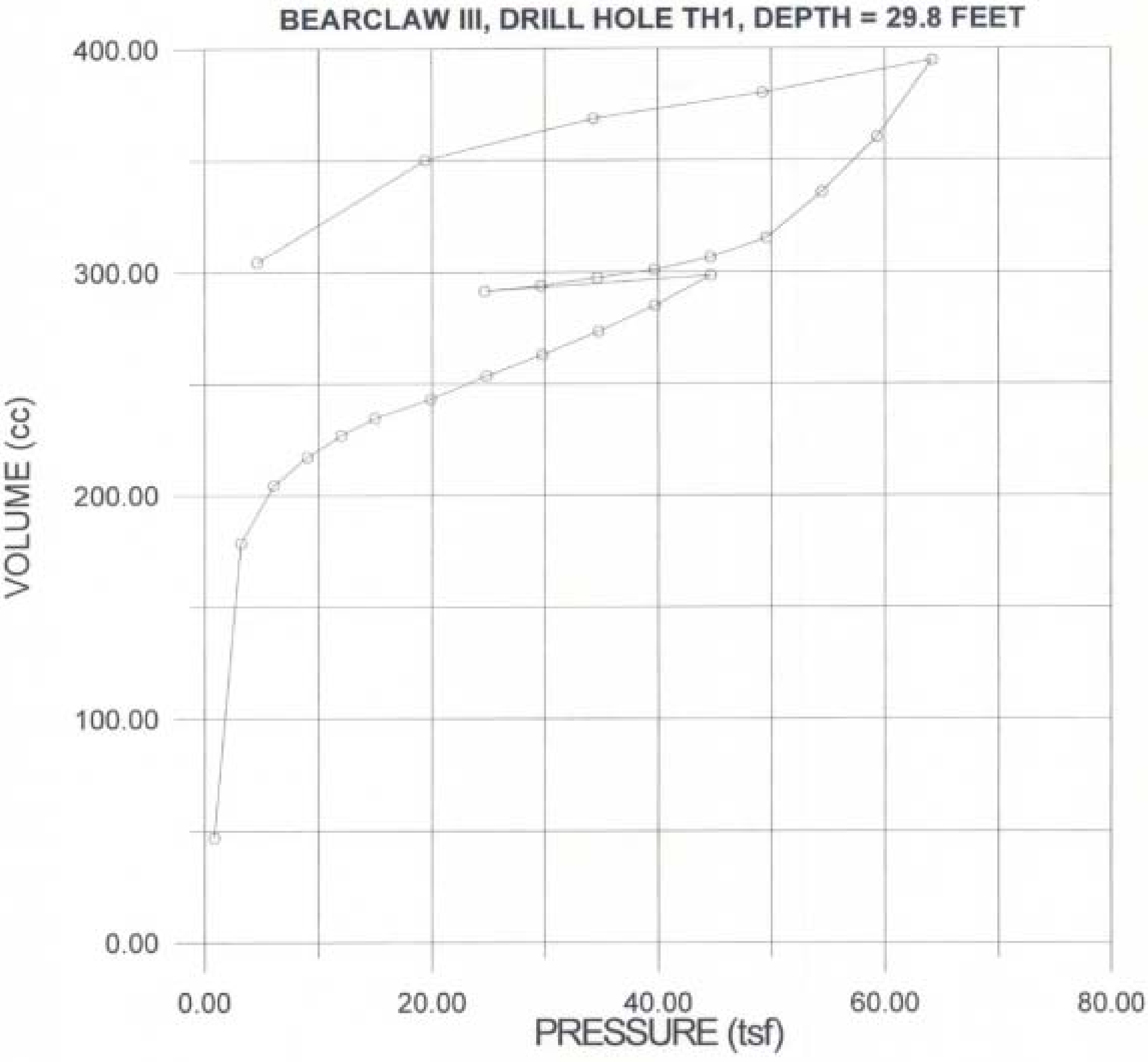
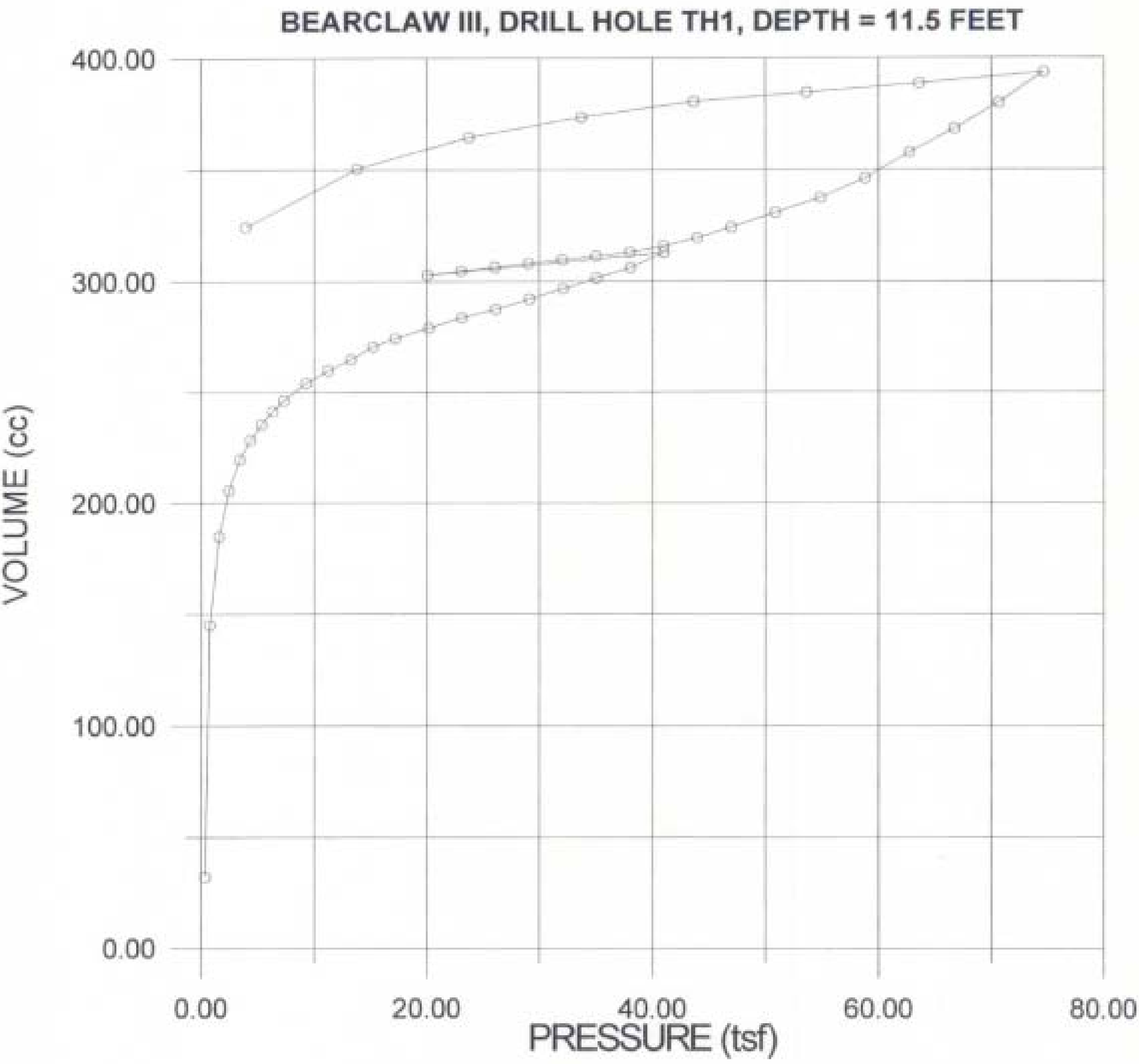
50.00
PRESSURE (tsf)



BEARCLAW III, DRILL HOLE TH1, DEPTH = 29.8 FEET
400.00
300.00
200.00
100.00
0.00
VOLUME (cc)
0.00
20.00
40.00
60.00
80.00
PRESSURE (tsf)


BEARCLAW III, DRILL HOLE TH1, DEPTH = 11.5 FEET
VOLUME (cc)
400.00
300.00
200.00
100.00
0.00
0.00
20.00
40.00
60.00
80.00
PRESSURE (tsf)

APPENDIX B Glossary of Pressuremeter Derived Geotechnical Parameters

σ_{ho}	=	In situ horizontal total stress (P_o from test curve)
u	=	Hydrostatic pore pressure
σ'_{ho}	=	In situ horizontal effective stress (σ_{ho} - u)
σ'_v	=	In situ vertical overburden stress (calculated from unit weights and u)
K_o	=	Coefficient of earth pressure at rest $K_o = \sigma'_{ho}/\sigma'_v$
P_c	=	Creep Pressure
P_f	=	Yield Pressure
OCR	=	Overconsolidation ratio
E	=	Initial modulus
E+	=	Reload modulus
E-	=	Unload modulus
E/P_L^*	=	Design ratio, $P_L^* = P_L - P_o$
S_u	=	Undrained shear strength determined by Gibson and Anderson (1961) procedure
c	=	Cohesion determined by Bachus et. al (1983) procedure
ϕ	=	Angle of internal friction determined by Hughes et. al (1979) procedure
P_L	=	Limit pressure

APPENDIX B

RECOMMENDATIONS FOR PAVEMENT CONSTRUCTION

FLEXIBLE PAVEMENT CONSTRUCTION RECOMMENDATIONS

Experience has shown that construction methods can have a significant effect on the life and serviceability of a pavement system. We recommend the proposed pavement be constructed in the following manner:

1. Where the subgrade soils do not satisfy the compaction requirements, they should be scarified, moisture treated, and recompacted. Soils should be compacted as specified in the COMPACTED FILL section of this report.

2. Utility trenches and all subsequently placed fill should be properly compacted and tested prior to paving. Fill should be compacted as specified in the COMPACTED FILL section of this report.

3. After final subgrade elevation has been reached and the subgrade compacted, the area should be proof-rolled with a heavy pneumatic tired vehicle (i.e., a 10-wheel dump truck). Subgrade that is pumping or deforming excessively should be removed and replaced.

4. If areas of soft or wet subgrade are encountered, the material should be overexcavated and replaced. Suitable on-site soils or structural fill may be used. Where extensively soft, yielding subgrade is encountered, we recommend the excavation be inspected by a representative of our office.

5. Aggregate base course should be laid in loose lifts not exceeding 6.0 inches, moisture treated to within 2.0 percent of the optimum moisture content, and compacted as specified in the COMPACTED FILL section of this report.

6. The aggregate base course may consist of processed recycled asphalt. The recycled asphalt base course should meet the gradation requirements of CDOT Class 5 or Class 6 base course. The recycled asphalt base should be laid in loose lifts not exceeding 6.0 inches, moisture treated and compacted as specified in the COMPACTED FILL section of this report.

7. Asphaltic concrete should be plant-mixed material and compacted to 92 percent of the maximum Theoretical Density.

8. The subgrade preparation and the placement and compaction of all pavement layers should be observed and tested. Compaction criteria should be met prior to the placement of the next paving lift.

RIGID PAVEMENT CONSTRUCTION RECOMMENDATIONS

Rigid pavement sections are not as sensitive to subgrade support characteristics as flexible pavement. Due to the strength of the concrete, wheel loads from traffic are distributed over a large area and the resulting subgrade stresses are relatively low. The critical factors affecting the performance of a rigid pavement are the strength and quality of the concrete, and the uniformity of the subgrade. We recommend subgrade preparation and construction of the rigid pavement section be completed in accordance with the following recommendations.

1. Where the subgrade soils do not satisfy the compaction requirements, they should be scarified, moisture treated, and compacted. Soils should be compacted as specified in the COMPACTED FILL section of this report.

2. Utility trenches and all subsequently placed fill should be properly compacted and tested prior to paving. Fill should be compacted as specified in the COMPACTED FILL section of this report.

3. The resulting subgrade should be checked for uniformity and all soft or yielding materials should be replaced prior to paving. This should be done by proof-rolling with a heavy pneumatic tired vehicle (i.e., a 10-wheel dump truck). Concrete should not be placed on soft, spongy, frozen, or otherwise unsuitable subgrade.

4. Subgrade should be kept moist prior to paving.

5. Curing procedures should protect the concrete against moisture loss, rapid temperature change, freezing, and mechanical injury for at least 3 days after placement. Traffic should not be allowed on the pavement for at least one week.

6. A white, liquid membrane curing compound, applied at the rate of 1 gallon per 150 square feet, should be used.

7. Construction joints, including longitudinal joints and transverse joints, should be formed during construction or should be sawed shortly after the concrete has begun to set, but prior to uncontrolled cracking. All joints should be sealed.

8. Construction control and inspection should be carried out during the subgrade preparation and paving procedures. Concrete should be carefully monitored for quality control.