GEOTECHNICAL INVESTIGATION

HELIX INDUSTRIAL BUILDINGS C AND D
NORTHWEST OF AVIATOR WAY AND DOUBLE HELIX COURT
DOUGLAS COUNTY, COLORADO

Prepared for:

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SCOPE

This report presents the results of our Geotechnical Investigation for the Helix Industrial Buildings C and D planned Northwest of Aviator Way and Double Helix Court in Douglas County, Colorado (Fig. 1). The purpose of our investigation was to evaluate the subsurface conditions to provide geotechnical design and construction recommendations for the project. The scope was described in our Service Agreement No. DN 23-0069R dated February 24, 2023. Evaluation of the property for the possible presence of potentially hazardous materials (Environmental Site Assessment) was not included in our scope.

This report was prepared from data developed during previous investigations nearby, site reconnaissance, field and laboratory testing, engineering analysis, and our experience. It includes our opinions and recommendations for design criteria and construction details for foundations, floor systems, slabs-on-grade, and drainage precautions. The recommendations presented in the report are based on the construction as currently planned. Other types of construction may require revision of this report and the recommended design criteria. A summary of our conclusions and recommendations follows. Detailed design criteria are presented within the report.

SUMMARY OF CONCLUSIONS

1. Strata encountered in our exploratory borings consisted of about 2 to 12 feet of existing sandy clay to clayey sand fill in ten borings underlain by sandstone and claystone bedrock. Sandy clay was encountered at the surface in one of the borings, and was underlain by sandstone bedrock. Sandstone and claystone bedrock was encountered at the surface in ten of the borings. Testing indicates the clay, existing fill, and claystone bedrock is expansive. The existing fill is unsuitable to support new improvements.

2. Groundwater was encountered during drilling in eight of the twenty-one borings at depths from 13 to 31 feet below existing grades. When the test holes TH-1 through TH-13 were checked after drilling on April 19, 2023, water was measured at depths of about 12 to 29 feet be-low grade in all thirteen building borings; no water was found in S-1 through S-8 to 20 feet. Groundwater levels may fluctuate seasonally and rise after construction in response to precipitation, landscape irrigation, and changes in land-use.

3. The presence of expansive and compressible soils and existing (undocumented) fill constitute a geologic hazard. There is risk that slabs-on-grade and foundations

may experience heave or settlement, and be damaged. We believe the recommendations presented in this report will help to control risk of damage; they will not eliminate that risk. Slabs-on-grade and, in some instances, foundations may be damaged by soil movements. Shallow groundwater may also be considered a geologic hazard.

4. We estimate up to about 5 ½ inches of ground heave is possible with normal post-construction wetting. Existing fill may also cause settlement. To reduce potential movements and provide more uniform support conditions for footing foundations, we recommend performing sub-excavation to a depth of at least 10 feet below foundations in Building D, and 12 feet below foundations in Building C. Existing soils are considered suitable for re-use as new fill from a geotechnical standpoint provided debris, vegetation/organics, and deleterious materials are removed. There should be relatively low risk of differential movements after sub-excavation, estimated to be about 1 inch or less. Design criteria for footings are presented in the report. Drilled piers bottomed in bedrock should be used if less movements are desirable.

5. Slab-on-grade floors should have about 1 inch of potential movement after sub-excavation unless excessive wetting occurs. Slabs should be isolated from foundations, framing and finishes to avoid transmitting movements. Structurally supported floors should be used if movements and damage are not tolerable. If structural floors are used above crawl spaces, we recommend foundation drains around the crawl space perimeters.

6. The expansive clayey surficial soils present risk of damaging heave to pavements and exterior flatwork. We recommend sub-excavating at least 3 feet below pavements and exterior flatwork to reduce movements and provide more uniform support subgrade. Deeper sub-excavation may be considered for better performance.

7. Surface drainage should be designed, constructed, and maintained to provide rapid removal of runoff away from the building and off pavements and flatwork. Water should not be allowed to pond adjacent to the buildings or on pavements or flatwork.

8. The design and construction criteria for foundations and floor system alternatives in this report were compiled with the expectation that all other recommendations presented related to surface drainage, landscaping irrigation, backfill compaction, etc. will be incorporated into the project and that the owner will maintain the structures, use prudent irrigation practices and maintain surface drainage. It is critical that all recommendations in this report are followed.

SITE CONDITIONS

The site consists of an approximate 6-acre parcel located northwest of Aviator Way and Double Helix Court in Douglas County, Colorado (Fig. 1 and Photo 1). This site is bordered by vacant parcels to the west and east, and industrial buildings to the north and south. A regional airport is located in the northwest vicinity of the site, and Colorado E470 is located in the southern vicinity. A brief review of historical Google Earth aerial photographs dating back to 1937 indicates the parcel was previously undeveloped and vacant. Several drainage paths were present in the southern half of the site. Between 1999 and 2002, Aviator Way is constructed directly south of the site. A drainage culvert was constructed as part of this roadway and partially encroached the southern site boundary. This culvert was removed between 2017 and 2018. Rough grading of the site took place between this timeframe as well. A temporary gravel parking lot in the northwest corner and an access road in the western half was constructed between 2017 and 2018. These were both removed prior to September 2019. Another gravel parking lot was constructed between 2019 and 2020 in the western half of the site, with an additional gravel parking area installed between June and September of that year in the southeast corner. The site has remained generally unchanged since 2019. The ground surface slopes generally from the west to east with topographic relief of about 20 feet.

Photo 1 – Google Earth© Aerial Site Photo, June 10, 2021

PROPOSED CONSTRUCTION

Plans prepared by MOA Architecture dated March 1, 2023 indicate the project will consist of construction of an approximate 93,600 square-foot warehouse building (Building C) in the southern half of the site part of the site, and an approximate 31,200 square-foot warehouse building (Building D) in the northern half of the site. Loading docks and ramps are planned along the western edges of each building. Paved access drives and parking will surround the buildings. We anticipate the building will be a one-story structure with mezzanines with no below-grade areas unless crawl spaces are used.

INVESTIGATION

We investigated subsurface conditions on March 15, March 16, and March 20, 2023, by drilling and sampling twenty-one exploratory borings at the approximate locations on Fig. 1. Thirteen deep borings were drilled for Buildings C and D to depths of 35 to 45 feet, and eight shallower borings located in the pavement area were advanced to depths of 5 to 20 feet. The borings were drilled using 4-inch diameter, continuous-flight solid-stem auger and a truck-mounted CME-45 drill rig. Prior to drilling, we contacted the Utility Notification Center of Colorado and local sewer and water districts to identify locations of buried utilities. The approximate boring elevations and locations were determined using a Leica GS18 GPS unit referencing the NAD83 and NAVD88 systems.

Samples were obtained at approximate 2 to 5 feet intervals using a 2.5-inch diameter (O.D.) modified California barrel sampler driven by blows from an automatic 140-pound hammer falling 30 inches. Bulk samples of auger cuttings from the upper 5 feet were also obtained from the shallow borings. Our field representative was present to observe drilling operations, log the strata encountered, and obtain samples for laboratory testing. Upon completion of drilling, we inserted hand-slotted PVC pipe in the test holes to facilitate delayed water level measurements. Graphical summary logs of the exploratory borings, including results of field penetration resistance tests and a portion of laboratory test results, are presented on Appendix A.

Samples were returned to our laboratory where they were examined, classified, and assigned testing. Laboratory tests included moisture content, dry density, percent silt and clay-sized particles (percent passing No. 200 sieve), Atterberg limits, swell-consolidation, unconfined compression, and water-soluble sulfate concentration. Swell-consolidation tests were performed by wetting samples under approximate overburden pressure (the pressure exerted by overlying soils). Laboratory test results are presented in Appendix B and summarized in Table B-I.

PREVIOUS INVESTIGATION

We performed a Geologic and Preliminary Geotechnical Investigation that included the subject site and presented result in a report dated, June 14, 2017 (Project No. DN48,887-115-R1). Our preliminary investigation consisted of nine exploratory borings drilled to depths of 25 to 35 feet. Strata encountered in our borings generally consisted of about 4 to 11 feet of sandy clay and/or clean to clayey sand underlain by claystone and sandstone bedrock, and conditions were highly variable. About 5 feet of fill was encountered in one of the borings. The clay and claystone were variably expansive. We recommended to remove and replace existing fill.

Our representatives observed placement and compaction of the Double Helix Court sub-excavation, wet utility trench backfill, street subgrade, and curb and gutter subgrade and performed tests with results presented under Project No. DN48,887.000-345. Pertinent data from the previous investigations were considered in preparation of this report.

SUBSURFACE CONDITIONS

Strata encountered in our exploratory borings consisted of about 2 to 12 feet of existing sandy clay to clayey sand fill in ten borings underlain by sandstone and claystone bedrock. Sandy clay was encountered at the surface in one of the borings, and was underlain by sandstone bedrock. Sandstone and claystone bedrock was encountered at the surface in ten of the borings. Some of the pertinent engineering characteristics of the soil and bedrock are described in the following paragraphs.

Existing Fill

Overburden soils consisted of sandy clay and clayey sand fill, and extended to depths of 2 to 12 feet in ten borings. The fill was stiff to very stiff, and medium dense based on results of field penetration resistance tests. Five of the sandy clay fill samples swelled 0.1 to 5.8 percent when wetted under overburden pressures, and one exhibited no swell. Two samples developed load-back swelling pressures of approximately 1,200 psf, and a soil suction values of 3.36. Three of the sandy clay fill samples contained 58 to 81 percent silt and clay-sized particles and exhibited moderate to high plasticity. One of the clayey sand fill samples exhibited no swell, and two samples contained 19 to 45 percent silt and clay-sized particles.

Natural Clay

Overburden soils consisted of sandy clay to a depth of about 12 feet in one of the building borings. The clay was stiff to very stiff based on results of field penetration resistance tests. Six clay samples swelled from 0.2 to 6.1 percent, one sample exhibited no swell, and one sample compressed 0.1 percent when wetted under overburden pressures. Four samples developed load-back swelling pressures of approximately 1,400 to 3,200 psf, and soil suction values of 3.46 to 4.58 pF. One sample exhibited an unconfined compressive strength of 13,580 psf, and four samples contained 54 to 79 percent silt and clay sized particles. Three samples exhibited moderate plasticity. Two sand samples contained 25 to 36 percent silt and clay sized particles.

Bedrock

Bedrock was encountered at grade or at depths of 7 to 17 feet below grade in all borings. The bedrock consisted of claystone and sandstone and is considered medium hard to very hard. Four claystone samples did not swell, and sixteen samples swelled 0.1 to 8.3 percent when wetted. Six claystone samples developed load-back swelling pressures of 2,900 to 11,800 psf, and nine claystone samples developed soil suction values of 3.10 to 4.85 pF. Four claystone samples had 50 to 91 percent fines and exhibited moderate plasticity. Four sandstone samples swelled 0.1 to 0.9 percent when wetted. Five sandstone samples did not swell, and seventeen samples compressed 0.1 to 1.9 percent when wetted. Three sandstone samples developed load-back swelling pressures of 1,300 to 6,700 psf, and Fourteen sandstone samples developed soil suction values of 2.34 to 4.65 pF. Three sandstone samples exhibited unconfined compressive strengths from 8,170 to 16, 310 psf. Two sandstone samples had 0 to 1 percent gravel. Eight sandstone samples had 6 to 46 percent fines and exhibited low to moderate plasticity. One interbedded claystone/sandstone sample swelled 3.3 percent when wetted. The interbedded claystone/sandstone sample contained 42 percent fines and exhibited moderate plasticity.

Groundwater

Groundwater was encountered during drilling in eight of the twenty-one borings at depths from 13 to 31 feet below existing grades. When the test holes TH-1 through TH-13 were checked after drilling on April 19, 2023, water was measured at depths of about 12 to 29 feet below grade in all thirteen building borings; water was not present in S-1 through S-8 to 20 feet. Groundwater levels may fluctuate seasonally and rise after construction in response to precipitation, landscape irrigation, and changes in land-use.

Seismicity

According to the USGS, Colorado's Front Range and eastern plains are considered low seismic hazard zones. The earthquake hazard exhibits higher risk in western Colorado compared to other parts of the state. The Denver Metropolitan area has experienced earthquakes within the past 100 years, shown to be related to deep drilling, liquid injection, and oil/gas extraction. Naturally occurring earthquakes along faults due to tectonic shifts are rare in this area.

The soil and bedrock at this site are not expected to respond unusually to seismic activity. The International Building Code (Section 16.13.2.2) defers the estimation of Seismic Site Classification to ASCE7-22, a structural engineering publication. Updates from the previous versions of ASCE7 include (1) incorporation of additional Site Classifications BC, CD, and DE, (2) removal of tabulated blow-count and shear-strength correlations to shear wave velocity, and (3) requires the engineer to reduce shear wave velocity values by a factor of 1.3 when empirically estimated or not directly measured. The table below summarizes ASCE7-22 Site Classification Criteria.

ASCE7-22 SITE CLASSIFICATION CRITERIA

Seismic Site Class	$\bar{v}_s$, Calculated Using Measured or Estimated Shear Wave Velocity Profile (ft/s)
A. Hard Rock	>5,000
B. Medium Hard Rock	>3,000 to 5,000
BC. Soft Rock	>2,100 to 3,000
C. Very Dense Sand or Hard Clay	>1,450 to 2,100
CD. Dense Sand or Very Stiff Clay	>1,000 to 1,450
D. Medium Dense Sand or Stiff Clay	>700 to 1,000
DE. Loose Sand or Medium Stiff Clay	>500 to 700
E. Very Loose Sand or Soft Clay	≥500
F. Soils requiring Site Response Analysis	See Section 20.2.1

Based on the results of our investigation, the reduced, empirically estimated average shear wave velocity values for the upper 100 feet range between 832 and 1,887 feet per second, with an average of 1,400 feet per second. We judge a Seismic Site Classification of CD is appropriate. The subsurface conditions indicate low susceptibility to liquefaction from a materials and groundwater perspective.

GEOLOGIC HAZARDS

Colorado is a challenging location to practice geotechnical engineering. The climate is relatively dry, and the near-surface soils are typically dry and comparatively stiff. These soils and related sedimentary bedrock formations tend to react to changes in moisture content. Some soils swell as they increase in moisture and are referred to as expansive soils. Other soils can compress significantly upon wetting and are identified as compressible soils. The soils that exhibit compressible behavior are more likely west of the Continental Divide; however, both types of soils occur throughout the state. In older parts of urban Denver, it is common to encounter fill (sometimes with debris), which is considered compressible.

Covering the ground with buildings, streets, driveways, parking lots, etc., coupled with landscape irrigation and changing drainage patterns leads to an increase in subsurface moisture conditions. As a result, some soil movement is inevitable. It is critical that all recommendations in this report are followed to increase the chances that the foundations and slabs-on-grade will perform satisfactorily. Owners and/or property managers must assume responsibility for maintaining structures and use appropriate practices regarding drainage and landscaping.

Existing (undocumented) fill and expansive soil and bedrock are present at this site which constitute a geologic hazard. Existing fill may be poorly compacted and compressible upon wetting or additional loading. There is risk that ground heave or settlement will damage slabs-on-grade and foundations. The risks can be mitigated, but not eliminated, by careful design, construction, and maintenance procedures. We believe the recommendations in this report will help reduce risk of foundation and/or slab damage; they will not eliminate that risk. Slabs-on-grade and, in some instances, foundations may be affected. Maintenance will be required to reduce risk.

Estimated Potential Heave

We calculated total potential heave at the ground surface for each deep boring. We estimate total potential ground heave may range from about ½ to 5 ½ inches considering a 20-foot depth of wetting. The majority of the heave is attributed to the expansive surficial clay layer. It is not certain this heave will occur. Settlement may occur in the undocumented fill, or excessive wetting and softening of sub-excavation fill occurs. We believe potential differential movements can be reduced to about 1-inch or less provided sub-excavation is performed successfully, as discussed later in our report.

Existing Fill

The site was rough graded during roadway construction and development of the surrounding parcels. We believe that portions of the clayey surficial material may be man-placed fill. Documentation of fill placement and compaction were not provided. We judge the fill to be unsuitable to support proposed construction, both due to expansive and compressible soil movement considerations. All fill should be removed and recompacted as moisture-conditioned, compacted fill as discussed in **Fill and Backfill**. The fill is suitable for re-use as new fill from a geotechnical standpoint.

SITE DEVELOPMENT

The primary geotechnical concerns at this site are expansive soils and bedrock and existing (undocumented) fill. There is risk of movement for foundations, floor slabs, and other surface improvements. Sub-excavation can be performed to reduce potential movements and provide more uniform support characteristics. The following discussions present our opinions and recommendations for site development.

Sub-Excavation

Expansive soil and bedrock and existing fill are present at depths likely to influence the performance of shallow foundations and slab-on-grade floors. Potential ground heave could be as much as 5 ½ inches with typical post-construction wetting. Without mitigation, drilled pier foundations and structurally supported floors should be used. If shallow foundations are desired for the buildings, we recommend sub-excavating to a depth of 10 feet below the lowest foundation element in Building D, and 12 feet below the lowest foundation element in Building C to reduce potential heave and provide more uniform support conditions. Existing fill was encountered in our borings located within Building C. We believe sub-excavation to a depth of 12 feet below lowest foundation element will mitigate the existing fill however, more fill may be present than our borings imply. If existing fill is present in the sub-excavation bottom, sub-excavation should be deepened to natural soils and should extend to the same depth across the entire building footprint. We estimate potential movements on the order of 1-inch or less after sub-excavation is performed and results in low swelling fill, provided excessive wetting does not occur. Differential movements should also be substantially reduced, as the fill is expected to act as a buffer or cushion, and distribute heave more evenly, should it occur from the claystone underlying the fill. We anticipate this sub-excavation depth will terminate above groundwater. If the subgrade becomes soft and wet, it can be stabilized by crowding 1 to 3-inch crushed rock into the subgrade until firm.

The existing soils are suitable for re-use as new fill provided they are free of debris, organics/vegetation, and other deleterious materials, and are thoroughly moisture conditioned and compacted. Sub-excavation should extend at least 5 feet outside the lateral extent of foundations. A conceptual sub-excavation profile is shown on Fig. 2.

In order for the sub-excavation procedure to be performed properly, close control of fill placement to specifications is required. Sub-excavation fill should be placed in loose lifts no thicker than 8 inches, moisture conditioned and compacted to at least 95 percent of standard

Proctor maximum dry density (ASTM D 698). Clay fill should be moisture conditioned to between 1 and 4 percent above optimum moisture content and sand fill should be moistened to within 2 percent of optimum, and compacted to at least 95 percent of standard proctor maximum dry density. Our field representative should observe and test compaction of fill during placement.

Sub-excavation has been used in the Denver area with satisfactory performance for the large majority of the sites where this ground modification method has been completed. The extent and depth of sub-excavation should be surveyed and an “as-built” plan of the sub-excavated areas should be prepared. We have seen isolated instances where settlement of sub-excavation fill has led to damage to buildings supported on shallow foundations. In most cases, the settlement was caused by wetting associated with poor surface drainage and/or poorly compacted fill placed at the horizontal limits of the sub-excavation. Special precautions should be taken for compaction of fill at corners, access ramps and edges of the sub-excavation due to equipment access constraints. The contractor should have the appropriate equipment to reach and compact these areas.

The excavation contractor should be chosen based on experience with sub-excavation and processing high moisture content clay fills and have the necessary mixing and compaction equipment. The contractor should provide a construction disc to break down fill materials. The operation will be relatively slow. Soil and bedrock clods should be broken down to about 3 inches or less. The excavation slopes should meet OSHA, state, and local safety standards.

We recommend at least 3 feet of sub-excavation, moisture-conditioning and re-compaction below pavements, sidewalks and surface improvements. Deeper sub-excavation to 5 feet can be considered for better performance.

Stabilization

Soft/loose, wet soils may be encountered at the bottom of excavations. Soft/loose excavation bottoms can likely be stabilized by crowding crushed rock into the soils until firm. Acceptable rock materials include, but are not limited to, No. 2 and No. 57 rock. Crushed rock on a layer of geosynthetic grid or woven fabric can also be used, which should reduce the amount of aggregate needed to stabilize the subgrade. Typically, a biaxially woven fabric or geogrid topped with 8 to 12 inches of 1 to 3-inch crushed rock will provide a stable working surface.

Underdrain

With long-term development and subsequent irrigation, groundwater could develop and rise. We advocate that this water should be controlled using an under-drain. The use of an underdrain system below or adjacent to sanitary sewer mains and services (a.k.a. area drain) is a common method to help control groundwater and provide a gravity outlet for foundation drains. If used, the underdrain should consist of 0.75 to 1.5-inch clean, free draining gravel surrounding a perforated PVC pipe (Fig. 7). We believe use of perforated pipe below sanitary sewer mains is the most effective approach. The line should consist of perforated or slotted, rigid PVC pipe placed at a grade of at least 0.5 percent. A positive cutoff (concrete) should be constructed around the sewer pipe and underdrain pipe immediately downstream of the point where the underdrain pipe leaves the sewer trench (Fig. 8). Solid pipe should be used down gradient of this cutoff wall. The underdrains should be designed to discharge to a gravity outfall constructed with a permanent concrete headwall and trash rack. The underdrain should be installed with clean-outs. To reduce the risk of cross-connecting sewer and underdrain services, we recommend using a 4-inch diameter pipe for sewer services and 3-inch diameter pipe for the underdrain services.

Excavation

We believe the soil and bedrock penetrated in our exploratory borings can generally be excavated with conventional, heavy-duty excavation equipment. Medium hard to very hard bedrock was encountered in our borings, and is expected in the sub-excavation cut. Very hard bedrock may require use of more robust excavation techniques, such as heavy ripping with bulldozer equipment.

We recommend the owner and the contractor become familiar with applicable local, state and federal safety regulations, including the current OSHA Excavation and Trench Safety Standards. We anticipate the clay and bedrock will classify as Type B soils, which require maximum slope inclinations of 1:1 (horizontal:vertical) for temporary excavations in dry conditions. Flatter slopes will be required below groundwater or if seepage is present. The contractor's "competent person" is required to review excavation conditions and refer to OSHA Standards when worker exposure is anticipated. Stockpiles of soils and equipment should not be placed within a horizontal distance equal to one-half the excavation depth, from the edge of the excavation. A professional engineer should design excavations deeper than 20 feet, if any.

Fill and Backfill

The on-site soil is generally suitable for reuse as new fill provided debris, organics/vegetation and other deleterious materials are substantially removed. Soil and bedrock particles larger than 3 inches in diameter should not be used for fill unless broken down. Bedrock clods may require additional effort to break down to 3-inch minus material. If imported fill is necessary, it should ideally consist of soil having a maximum particle size of 3 inches, between 25 and 50 percent passing a No. 200 sieve, a liquid limit less than 35 and a plasticity index less than 15. Potential fill materials should be submitted to our office for approval prior to importing to the site.

Prior to fill placement, debris, organics/vegetation and deleterious materials should be substantially removed from areas to receive fill. The surface to be filled should be scarified to a depth of at least 8 inches, moisture conditioned and compacted to the criteria below. Subsequent fill should be placed in thin (8 inches or less) loose lifts, moisture conditioned to within 2 percent of optimum moisture content for sand and between 1 and 4 percent above optimum moisture content for clay, and compacted to a minimum of 95 percent of standard Proctor maximum dry density (ASTM D 698).

Our experience indicates fill and backfill can settle, even if properly compacted to the criteria provided above. Factors that influence the amount of settlement are depth of fill, soil type, degree of compaction, and time. The length of time for the compression to occur can be a few weeks to several years. The degree of compression of the recommended fill under its own weight will likely range from low for granular soils (½ percent or less), to moderate for clay mixtures (1 percent). Any improvements placed over backfill should be designed to accommodate movement.

Utilities

Water, storm sewer and sanitary sewer lines are often constructed beneath slabs and pavements. Compaction of utility trench backfill can have a significant effect on the life and serviceability of floor slabs, pavements and exterior flatwork. Our experience indicates use of self-propelled compactors results in more reliable performance compared to fill compacted by an attachment on a backhoe or trackhoe. The upper portion of the trenches should be widened to allow the use of a self-propelled compactor. During construction, careful attention should be paid to compaction at curb lines and around manholes and water valves.

Special attention should be paid to backfill placed adjacent to manholes as we have observed conditions where settlement in excess of 1 percent has occurred after completion of construction. Flowable fill may be considered at critical utility crossings where it would be difficult to achieve adequate compaction. Utility trench backfill should be moisture-conditioned and compacted per jurisdictional requirements. The placement and compaction of utility trench backfill should be observed and tested by a representative of our firm during construction.

Temporary construction dewatering systems may be needed to properly install deep utilities below groundwater. We believe that dewatering for excavations which penetrate less than 3 to 5 feet below groundwater may be accomplished using conventional sump and pump methods with trenches. Deeper excavations may require more elaborate dewatering (such as well points).

FOUNDATIONS

Our investigation indicates expansive soil and bedrock and ex fill is present at depths likely to influence the performance of shallow foundations. Provided sub-excavation is performed as recommended previously in our report, we believe footing foundations can be used for the structure with relatively low risk of differential movements. Drilled piers bottomed in bedrock and structurally supported floors should be used in lieu of sub-excavation if less movement is desired. Design and construction criteria for both footings and drilled piers are presented below. The criteria presented below were developed from analysis of field and laboratory data and our experience.

Footings (After Sub-Excavation)

1. Footings should be constructed on at least 10 or 12 feet of new, moisture conditioned and compacted fill and sited at least 5 feet inside the sub-excavation limits. Where soils are disturbed during excavation or in the forming process, or if any loose/soft soils are exposed in excavations, the soils should be removed and re-compacted or stabilized as recommended in **Fill and Backfill**, prior to placing concrete.

2. Footings should be designed for a maximum allowable soil pressure of 3,000 psf and a minimum deadload pressure of 1,000 psf. Lateral earth pressures can be calculated based on equivalent fluid density using at least 45 pcf for the active case. Walls which can deflect less than 1 percent of height can be designed using 60 pcf. For the at-rest case, where essentially no lateral movement is allowed, we recommend using at least 65 pcf. Footing translation can be resisted using an equivalent fluid density of 300 pcf for the passive case, providing backfill is similar to the site soils, is well compacted and remains in-place. The coefficient of friction for sliding may be taken as 0.35. These values have not been factored. The structural engineer should apply appropriate factors of safety in design.

3. If interrupted footings are necessary to maintain deadload, a 4-inch (minimum) continuous void should be constructed below grade beams or foundation walls, between pads. Void should also be used if interior drains are planned in crawl space areas (if any) to allow water to pass from the outside of the wall to the interior drain (Fig. 4).

4. Footings should have a minimum width of 18 inches. Foundations for isolated columns should have minimum dimensions of 24 inches by 24 inches. Larger sizes may be required depending upon the loads and structural system used.

5. Foundation walls and grade beams should be well-reinforced. We recommend reinforcement sufficient to span an unsupported distance of at least 10 feet, where applicable. Reinforcement should be designed by the structural engineer considering the effects of lateral loads on the wall performance.

6. Exterior footings must be protected from frost action. Normally, 3 feet of frost cover is assumed in the area.

7. The completed foundation excavations should be observed by a representative of our firm to confirm subsurface conditions are as anticipated. Our representative should observe and test moisture and compaction of the fill and backfill.

8. Excessive wetting of foundation soils during and after construction can cause heave or softening and consolidation of foundation soils and result in footing movements. Proper surface drainage around the building is critical to control wetting.

Drilled Piers Bottomed in Bedrock

1. Piers should be designed for a maximum allowable end pressure of 30,000 psf and allowable skin friction of 3,000 psf for the portion of pier in comparatively unweathered bedrock. Skin friction should be neglected in overburden soils, weathered claystone, and where temporary casing is used (if any).

2. We recommend designing the piers for a minimum deadload pressure of 10,000 psf based on the pier cross sectional area. If the minimum deadload pressure cannot be achieved, the minimum length should be increased to compensate for the deficiency, using the allowable skin friction value discussed above.

3. Piers should penetrate at least 5 feet into the comparatively unweathered bedrock with a minimum length of 27 feet.

4. There should be a 6-inch (or thicker) continuous void beneath all grade beams and foundation walls, between piers, to concentrate the deadload of the structures onto the piers.

5. Formation of "mushrooms" or enlargements at the tops of piers should be avoided during pier drilling and subsequent construction operations.

6. Shear rings should be installed in the penetration zone of all piers. Shear rings should have a height of at least 2 inches and extend 3 to 4 inches beyond the pier shaft to increase load transfer through skin friction. Rings should be spaced about 2 feet on-center.

7. Pier drilling should produce shafts with relatively undisturbed bedrock exposed. Excessive remolding and caking of bedrock on pier walls should be removed. The bedrock surface should be rough or roughened. Pier drilling contractors should be required to have properly sized augers. Use of side cutters or teeth to increase the effective diameter should not be allowed.

8. Piers should be reinforced their full length and the reinforcement should extend an adequate distance into grade beams, foundation walls, and pier caps. Reinforcement should be designed by the structural engineer considering lateral earth pressures on walls.

9. Piers should have a center-to-center spacing of at least three pier diameters when designing for vertical loading conditions, or they should be designed as a group. Piers aligned in the direction of lateral forces should have a center-to-center spacing of at least six pier diameters. Reduction factors for closely spaced piers are provided in the following section.

10. The minimum diameter will depend on the length-to-diameter ratio (L/D). We recommend the piers be designed with a maximum L/D ratio not to exceed 30.

11. Groundwater was encountered at 12 to 31 feet in our borings. Piers may require casing. Cased portions of the piers should not be relied upon for side shear resistance.

12. Piers should be carefully cleaned prior to placement of concrete. Concrete should be on site and placed in the pier holes immediately after the holes are drilled, cleaned and inspected to avoid collecting water and possible contamination of open pier holes. Concrete should not be placed by free fall if there is more than about 3 inches of water at the bottom of the hole.

13. Concrete should have sufficient slump to fill the pier holes and not hang on the reinforcement. We recommend a slump of 6 inches ± 1 inch.

14. Some pier-drilling contractors use casing with an I.D. equal to the specified pier diameter. This practice results in a pier diameter less than specified. The design specification of piers should consider the alternatives. If full-size casing is desired (I.D. of casing equal to specified pier diameter) it should be clearly specified. If the design considers the reduction in diameter, then the specification should include a tolerance for smaller diameter for cased piers.

15. Some movement of drilled pier foundations should be anticipated to mobilize the skin friction. We estimate the movement will be on the order of ¼ to ½ inch. Differential movement between adjacent piers may equal total movement.

16. Installation of drilled piers should be observed by a representative of our firm to identify the proper bearing strata, confirm subsurface conditions are as anticipated from our borings, and observe the contractor's installation procedures.

Laterally Loaded Piers

Lateral load analysis of piers can be performed with the software analysis package LPILE by Ensoft, Inc. We believe this method of analysis is appropriate for piers with a pier length to diameter ratio of seven or greater. Suggested criteria for LPILE analysis are presented in the following table.

SOIL INPUT DATA FOR "LPILE"

Soil Type	Clay	Claystone Bedrock
Soil Model Type	Stiff Clay w/o Free Water	Stiff Clay w/o Free Water
Effective Unit Weight (pci)	0.074	0.076
Cohesive Strength, c (psi)	10	55 (LPile maximum)
Soil Strain, ε_{50} (in/in)	0.005	0.004

The ε_{50} represents the strain corresponding to 50 percent of the maximum principal stress difference.

Closely Spaced Pier Reduction Factors

For axial loading, no reduction is needed for a minimum spacing of three diameters (center to center). At one diameter (piers touching), the skin friction reduction factor for both piers would be 0.5. End pressure values would not be reduced provided the bases of the piers are at similar elevations. Interpolation can be used between one and three diameters.

For lateral loading, no reduction is needed for piers in-line with the direction of lateral loads with a minimum spacing of six diameters (center-to-center) based upon the larger pier. If a closer spacing is required, the modulus of subgrade reaction for initial and trailing piers should be reduced. At a spacing of three diameters, the effective modulus of subgrade reaction of the first pier can be estimated by multiplying the given modulus by 0.6; for trailing piers in a line at three-diameter spacing, the factor is 0.4. Linear interpolation can be used for spacing between three and six diameters.

Reductions to the modulus of subgrade reaction can be accomplished in LPILE by inputting the appropriate modification factors for p-y curves. Reducing the modulus of subgrade reaction in trailing piers will result in greater computed deflections on these piers. In practice, a grade beam can force deflections of all piers to be equal. Load-deflection graphs can be generated for each pier by using the appropriate p-multiplier values. The sum of the piers lateral load resistance at selected deflections can be used to develop a total lateral load versus deflection graph for the system of piers.

For lateral loads perpendicular to the line of piers, a minimum spacing of three diameters can be used with no capacity reduction. At one diameter (piers touching) the piers should be analyzed as one unit. Interpolation can be used for intermediate conditions.

FLOOR SYSTEMS

Slabs-On-Grade

With the recommended sub-excavation, we estimate potential movements of about 1-inch or less are probable for slab-on-grade floors built on new fill placed and compacted as discussed in **Sub-Excavation**. More heave or settlement may occur if excessive wetting occurs. Conventional slab-on-grade floors can be used provided risk of heave and distress is acceptable to the owner. There will likely be distress to the slabs and sensitive finishes. We recommend structurally supported floors if movements cannot be tolerated.

Where conventional slabs-on-grade are used and the owner accepts the risks, we recommend the following design and construction criteria. These recommendations will not prevent movement. Rather, they tend to reduce damage if movement occurs.

1. Slabs should be separated from exterior walls and interior bearing members with a slip joint that allows free vertical movement of the slabs. This detail can reduce cracking when movement occurs.

2. Slabs should be placed directly on properly moisture conditioned, well-compacted fill. The 2021 International Building Code (IBC) requires a vapor retarder between the base course or subgrade soils and the concrete slab-on-grade floor, including PTS. The merits of installation of a vapor retarder below floor slabs depend on the sensitivity of floor coverings and building use to moisture. A properly installed vapor retarder (6 mil minimum) is more beneficial below concrete slab-on-grade floors where floor coverings, painted floor surfaces or products stored on the floor will be sensitive to moisture. The vapor retarder is most effective when concrete is placed directly on top of it, rather than placing a sand or gravel leveling course between the vapor retarder and the floor slab. The placement of concrete on the vapor retarder may increase the risk of shrinkage cracking and curling. Use of concrete with reduced shrinkage characteristics including minimized water content, maximized coarse aggregate content, and reasonably low slump will reduce the risk of shrinkage cracking and curling. Considerations and recommendations for the installation of vapor retarders below concrete slabs are outlined in Section 5.2.3.2 of the 2015 report of American Concrete Institute (ACI) Committee 302, "Guide for Concrete Floor and Slab Construction (ACI 302.1R-15)".

3. Use of slab-supported partition walls should be minimized. If slab-bearing partitions are used, they should be designed and constructed with a minimum 2-inch space to allow for slab movement. Differential slab movements may cause cracking of partition walls. If the void is provided at the top of partitions, the connection between the slab-supported partition and foundation-supported walls should be

detailed to allow differential movement. Doorways, wall partitions perpendicular to the exterior wall or walls supported by foundations should be detailed to allow for vertical movement. Interior perimeter framing and finishing should not extend onto slabs-on-grade, or if necessary, should be detailed to allow for movement.

4. Underslab plumbing should be eliminated where feasible. Where such plumbing is unavoidable, it should be pressure tested for leaks prior to slab construction and provided with flexible couplings. Pressurized water supply lines should be brought above the floors as quickly as possible.

5. Plumbing and utilities that pass through the slabs should be isolated from the slabs and constructed with flexible couplings. Utilities, as well as electrical and mechanical equipment should be constructed with sufficient flexibility to allow for movement.

6. HVAC or other mechanical systems supported by the slabs (if any) should be provided with flexible connections capable of withstanding at least 2 inches of movement.

7. Exterior flatwork and sidewalks should be separated from the structures. These slabs should be detailed to function as independent units. Movement of these slabs should not be transmitted to the foundations.

8. The American Concrete Institute (ACI) recommends frequent control joints be provided in slabs to reduce problems associated with shrinkage cracking and curling. To reduce curling, the concrete mix should have a high aggregate content and a low slump. If desired, a shrinkage compensating admixture could be added to the concrete to reduce the risk of shrinkage cracking. We can perform a mix design or assist the design team in selecting a pre-existing mix.

Structurally Supported Floors

To our knowledge, there are no soil treatments combined with slab-on-grade floors that will result in the same reduction in risk of floor movement (relative to the risk inherent for a floor slab placed directly on the natural soils), as would be provided by a structural floor. If floor movement cannot be tolerated, then a structurally supported floor should be used.

A structural floor is supported by the foundation system. Design and construction issues associated with structural floors include ventilation and lateral loads. Where structurally supported floors are installed over a crawl space, the required air space depends on the materials used to construct the floor and the potential expansion of the underlying soils. Building codes require a clear space of 18 inches between exposed earth and untreated wood floor components. For non-organic floor systems, we recommend a minimum clear space of 8 inches. This minimum clear space should be maintained between any point on the underside of the floor system (including beams and floor drain traps) and the soils.

A slab-on-void system may also be considered. Void form should be chosen to break down quickly after the slab is placed. A sand or gravel leveling base below the void form should not be used. We recommend against the use of wax or plastic-coated boxes unless provisions are made to allow water vapor to penetrate the boxes, resulting in softening. The voids should not transmit heave to the floor system.

Where structurally supported floors are used, utility connections including water, gas, air duct, and exhaust stack connections to floor supported appliances should be capable of absorbing some deflection of the floor. Plumbing that passes through the floor should ideally be hung from the underside of the structural floor and not lain on the bottom of the excavation. It is prudent to maintain the minimum clear space below all plumbing lines; this configuration may not be achievable for some parts of the installation.

Control of humidity in crawl spaces is important for indoor air quality and performance of wood floor systems. We believe the best current practices to control humidity involve the use of a vapor retarder or vapor barrier (6 mil) placed on the soils below accessible subfloor areas. The vapor retarder/barrier should be sealed at joints and attached to concrete foundation elements.

Exterior Flatwork

We recommend exterior flatwork and sidewalks around the building be isolated to reduce the risk of transferring slab movement to the structure. One alternative would be to construct the inner edges of the flatwork on haunches or steel angles bolted to the foundation walls and detailing the connections such that movement will cause less distress to the building, rather than tying the slabs directly into the building foundations. Construction on haunches or steel angles and reinforcing the sidewalks and other exterior flatwork will reduce the potential for differential settlement and better allow them to span across foundation wall backfill. Frequent control joints should be provided to reduce problems associated with shrinkage. Panels that are approximately square perform better than rectangular areas.

RETAINING WALLS

Mechanically-stabilized earth retaining walls are planned along the western side of the site and will be up to about 5 feet in height. Concrete retaining walls are assumed for the loading dock ramp walls. Retaining walls should be designed to resist lateral earth pressures. The lateral earth pressure will depend on the height of the wall, type of backfill, slope of backfill surface, surcharge loads and allowable horizontal movement at the top of the wall. Where multiple walls are closely spaced, the lower wall(s) design should consider surcharge from upper walls. Internal and global stability of the walls should be considered. We expect retaining walls may be constructed using reinforced concrete or mechanically stabilized by earth (MSE).

For a very rigid wall where negligible or very little deflection will occur, an “at-rest” lateral earth pressure should be used in design. For walls that are free to rotate slightly, an “active” earth pressure resistance can be used. A “passive” earth pressure resistance can be used to resist sliding and overturning. Passive resistance requires movement to generate resistance.

We have tabulated equivalent fluid density values for on-site soil used as backfill in lateral earth pressure restraint design below. These values assume that backfill will be moisture-conditioned and compacted as described previously. The values do not include allowances for surcharge loads such as adjacent foundations, sloping backfill, vehicle traffic, or hydrostatic pressure.

TABLE I: LATERAL EQUIVALENT FLUID DENSITIES

Load Condition	Clay
Active Equivalent Fluid Density (pcf)	50
At-Rest Equivalent Fluid Density (pcf)	70
Passive Equivalent Fluid Density (pcf)*	250*

*Assumes backfill will not be removed

We encountered shallow expansive clay and claystone in our exploratory borings. Sub-excavation, as discussed previously in our report, can be considered below retaining walls to reduce potential movements. We have seen instances where owners/contractors elect to perform less or no sub-excavation and assume the risk of movement. The owner should determine the acceptable tolerance of movement. MSE walls can likely tolerate more movement than concrete walls.

MSE Walls

MSE retaining walls assume that some movement of the wall will occur to mobilize the shear strength of the soil. We assume retained soil and backfill above the reinforced zone will be on-site soils or similar soils. The onsite soil should not be used in the reinforced zone. We recommend the reinforced zone of the MSE Walls be constructed with imported sand and gravel meeting CDOT Class 5 or 6 Aggregate Road Base Specification (or better). Angular gravel meeting AASHTO No. 57 or 67 Specification may be used for the reinforced soil (if desired) and is recommended for the leveling pad and drainage material.

Most MSE block retaining wall design programs require input of soil parameters for foundation soil, leveling pad, reinforced soil and retained soil. We recommend the parameters presented below be used for the design of the wall.

MSE SOIL INPUT PARAMETERS

Material Use	Material Description & Classification	Cohesion (psf)	Internal Friction Angle (degrees)	Unit Weight (pcf)
Foundation Soil	Clay, Sandy (CL)	50	25	120
Leveling Pad	Gravel (imported) AASHTO #57 or 67 Coarse Concrete Aggregate	0	38	105
Reinforced Soil (import recommended)	Sand, Gravelly, Silty, CDOT Class 6 Road Base (or better)	0	34	135
Retained Soil	Clay, Sandy (CL)	0	25	120

Free draining granular backfill should be used adjacent to the wall. We recommend a free-draining sand and gravel material with less than 3 percent fines (passing No. 200 sieve) be used as backfill for a zone within at least 1 foot behind the wall. Imported backfill should be tested and approved by our firm prior to importing. The upper 2 feet of wall backfill should be derived from the on-site clay. Fill should be placed and compacted to the criteria provided in Fill and Backfill. Special precautions should be taken to avoid over-stressing the wall during compaction. We recommend small, hand-operated compactors be used.

We recommend a drain pipe be installed beneath the free-draining backfill zone. The drain should consist of a 4-inch perforated PVC pipe encased in at least 1 foot of free draining gravel. The drain should slope to a positive gravity outlet. Any pipe installed beneath the wall should be solid and strong enough to resist the overburden pressure from the weight of the wall. Drain discharge in front of the wall should occur to well drained areas at least 5 feet beyond the toe of the wall.

Concrete Walls

Conventional cast-in-place concrete retaining walls are expected to be used for the loading dock ramp walls. Independent retaining walls not attached to buildings may be constructed on footings, provided the walls can tolerate some heave. Frequent control joints are recommended in concrete walls to control cracking. Footings for retaining walls can be designed for a maximum allowable soil pressure of 3,000 psf, have a minimum width of 20 inches, and be provided at least 3 feet of frost cover. Buildings should not be constructed within the retaining wall backfill zone.

For lateral load resistance, footings can be designed with a coefficient of friction between the base of the footings and soils of 0.3. Lateral loads can be resolved by evaluating passive resistance using a passive equivalent fluid density as presented in Table I for on-site soils or backfill that is properly compacted and will not be removed. These are unfactored values; appropriate factors of safety should be applied during design.

We recommend a drain be installed beneath the free-draining backfill zone. A typical earth retaining wall drain detail is provided on Fig. 6. The drain should consist of a 4-inch perforated PVC pipe encased in at least 1-foot of free draining gravel. The drain should slope at least 0.5 percent to a positive gravity outlet or weep holes. Pipes installed beneath the wall should be solid and strong enough to resist the overburden pressure from the weight of the wall. Drain discharge in front of the wall should occur at least 5 feet beyond the toe of the wall and into well-drained areas.

SUBSURFACE DRAINAGE

Water from surface irrigation of landscaping frequently flows through relatively permeable backfill placed adjacent to a building and collects on the surface of less permeable soils occurring at the bottom of foundation excavations. This process can cause wet or moist crawl space conditions after construction.

Foundation drains are typically not installed for buildings where no below-grade construction is planned. Installation of these drains can help control accumulation of moisture around footings, and help to control excessive wetting. Drains do not eliminate wetting. Installation of drains would be a benefit in areas where the ground surface next to the building will not be paved. If a structural floor and crawl space floor system is selected, a drain system should be considered around the perimeter (Figs. 3 and 4). The drain should consist of a 3 or 4-inch diameter, perforated or slotted pipe encased in free-draining gravel. The drain should lead to a positive gravity outlet, such as a subdrain located beneath the sewer, or to a sump where water can be removed by pumping. Sump pumps must be maintained by the owner.

PAVEMENTS

The project will include automobile parking accessible via Aviator Way and Double Helix Court. The performance of pavements is dependent upon the characteristics of the subgrade soil, traffic loading and frequency, climatic conditions, drainage and pavement materials.

As part of our investigation for this project, we drilled eight borings in the area of proposed automobile parking, access drives, truck lanes, and loading docks. A bulk sample comprised of the upper 5 feet from S-3, S-4, and S-5 contained 54 percent silt and clay sized particles with a liquid limit of 42 and a plasticity index of 26. The sample classifies as A-7-6, which is considered poor subgrade.

Pavement subgrade soils will generally consist of sandy clay or fill of similar composition and also classify as A-7-6 according to AASHTO criteria. Clayey soils are considered poor subgrade. Pavements can experience heave due to expansive clay or settlement due to compression of wetted clay or compression of utility trench backfill. Swell tests indicate the subgrade soils have high expansion potential. We recommend sub-excavating pavement areas to a depth of at least 3 feet to mitigate the presence of expansive existing fill and improve pavement performance. Deeper sub-excavation of 5 feet can be considered for better performance. Subgrade should be proof-rolled with a loaded, tandem-axle dump truck to disclose soft/loose areas. These areas should be reworked and compacted. Subgrade areas that pass proof-roll should be stable enough to pave.

We assume flexible hot mix asphalt (HMA) pavement is planned for the parking area. Rigid portland cement concrete (PCC) pavement should be considered for areas where the pavement will be subjected to frequent turning of heavy vehicles (such as loading dock areas). Alternatives that include each material are provided below. We followed the Douglas County Roadway Design and Technical Criteria Manual to calculate a subgrade resilient modulus of 7,610 psi, and considered their methodology to develop our pavement thickness calculations for both flexible and rigid pavements. Minimum pavement sections based on soil types and expected traffic are tabulated below. Flexible and rigid pavement materials, construction and maintenance guidelines are presented in Appendix C.

SUMMARY OF RECOMMENDED MINIMUM PAVEMENT ALTERNATIVES

Traffic Classification	Hot Mix Asphalt (HMA) + Aggregate Base (ABC)	Hot Mix Asphalt (HMA)	Portland Cement Concrete (PCC)
Automobile Parking Area	4" HMA* + 8” ABC	6” HMA	6” PCC
Access Drives and Truck/Fire Lanes	5” HMA + 10” ABC	8” HMA	6” PCC
Loading Docks	-	-	7” PCC

If use o<u>f the flexible pavement is elected, the owner must be willing to accept the risk of comparatively high maintenance costs</u>. To improve performance of flexible pavement sections in the access drives and truck parking areas we recommend the full-depth section consist of 8 inches of “G” mix asphalt. Our experience indicates problems with asphalt pavements can occur where heavy trucks drive into loading and unloading zones and turn at low speeds. In areas of concentrated loading and turning movements by heavy trucks, such as at entrances, loading and unloading areas, and trash collection areas, we recommend a 6-inch or thicker Portland cement concrete pad be constructed at any loading docks and dumpster locations, or other areas where trucks will stop or turn. The concrete pads should be of sufficient size to accommodate truck turning, trash pickup and delivery/loading areas. A section of 7 inches can be used if extra durability is desired.

The design of a pavement system is as much a function of paving materials as supporting characteristics of the subgrade. All soils that will support pavements should be scarified, moisture conditioned, and compacted prior to paving. The quality of each construction material is reflected by the strength coefficient used in the calculations. If the pavement system is constructed of inferior material, then the life and serviceability of the pavement will be substantially reduced. Materials and placement methods should conform to the requirements of Douglas County. All materials planned for construction should be tested to confirm their compliance with project specifications.

Control joints should separate concrete pavements into panels as recommended by ACI. No de-icing salts should be used on paving concrete for at least one year after placement. Routine maintenance, such as sealing and repair of cracks and overlays at 5 to 7-year intervals, are necessary to achieve long-term performance of an asphalt system. We recommend application of a rejuvenating sealant such as fog seal after the first year. Deferring maintenance usually results in accelerated deterioration of pavements leading to higher future maintenance costs.

A primary cause of early pavement deterioration is water infiltration into the pavement system. The addition of moisture usually results in softening of the subgrade and eventual failure of the pavement. We recommend drainage be designed for rapid removal of surface runoff. Curb and gutter should be backfilled and the backfill compacted to reduce ponding adjacent to the pavements. Final grading of the subgrade should be carefully controlled so that design cross-slope is maintained and low spots in the subgrade which could trap water are eliminated. Seals should be provided between curb and pavement and at all joints to reduce moisture infiltration. Landscaped areas and detention ponds in pavements should be avoided.

Recommended material properties and construction criteria for pavements are provided in Appendix C. These criteria were developed from analysis of the field and laboratory data and our experience. If the materials cannot meet these recommendations, then the pavement design should be re-evaluated based upon available materials.

CONCRETE

Concrete in contact with soil can be subject to sulfate attack. We measured water-soluble sulfate concentrations of 0.07 percent or less in three samples. As indicated in our tests and ACI 318-19, the sulfate exposure class is *Not Applicable or S0.*

SULFATE EXPOSURE CLASSES PER ACI 318-19

Exposure Classes		Water-Soluble Sulfate (SO_4) in Soil [A] (%)
Not Applicable	S0	< 0.10
Moderate	S1	0.10 to 0.20
Severe	S2	0.20 to 2.00
Very Severe	S3	> 2.00

A) Percent sulfate by mass in soil determined by ASTM C1580

For this level of sulfate concentration, ACI 318-19 *Code Requirements* indicates there are no special cement type requirements for sulfate resistance as indicated in the table below.

CONCRETE DESIGN REQUIREMENTS FOR SULFATE EXPOSURE PER ACI 318-19

Exposure Class		Maximum Water/ Cement Ratio	Minimum Compressive Strength (psi)	Cementitious Material Types A			Calcium Chloride Admixtures
				ASTM C150/ C150M	ASTM C595/ C595M	ASTM C1157/ C1157M	
S0		N/A	2,500	No Type Restrictions	No Type Restrictions	No Type Restrictions	No Restrictions
S1		0.50	4,000	II[B]	Type with (MS) Designation	MS	No Restrictions
S2		0.45	4,500	V [B]	Type with (HS) Designation	HS	Not Permitted
S3	Option 1	0.45	4,500	V + Pozzolan or Slag Cement [C]	Type with (HS) Designation plus Pozzolan or Slag Cement [C]	HS + Pozzolan or Slag Cement [C]	Not Permitted
S3	Option 2	0.40	5,000	V [D]	Type with (HS) Designation	HS	Not Permitted

A) Alternate combinations of cementitious materials shall be permitted when tested for sulfate resistance meeting the criteria in section 26.4.2.2(c).

B) Other available types of cement such as Type III or Type I are permitted in Exposure Classes S1 or S2 if the C3A contents are less than 8 or 5 percent, respectively.

C) The amount of the specific source of pozzolan or slag to be used shall not be less than the amount that has been determined by service record to improve sulfate resistance when used in concrete containing Type V cement. Alternatively, the amount of the specific source of the pozzolan or slab to be used shall not be less than the amount tested in accordance with ASTM C1012 and meeting the criteria in section 26.4.2.2(c) of ACI 318.

D) If Type V cement is used as the sole cementitious material, the optional sulfate resistance requirement of 0.040 percent maximum expansion in ASTM C150 shall be specified.

Superficial damage may occur to the exposed surfaces of highly permeable concrete, even though sulfate levels are relatively low. To control this risk and to resist freeze-thaw deterioration, the water-to-cementitious materials ratio should not exceed 0.50 for concrete in contact with soils that are likely to stay moist due to surface drainage or high-water tables. Concrete should have a total air content of 6 percent ± 1.5 percent. We advocate damp-proofing of all foundation walls and grade beams in contact with the subsoils.

SURFACE DRAINAGE

Performance of foundations, flatwork, and other surface improvements is influenced by the moisture conditions existing within the foundation or subgrade soils. The risk of wetting the foundation and floor subgrade soils can be reduced by carefully planned and maintained surface grades and drainage. Excessive wetting before, during and/or after construction may cause movement of foundations and slabs-on-grade. Surface drainage should be designed, constructed, and maintained to provide rapid removal of surface water runoff away from the proposed buildings and off pavements and flatwork. We recommend the following precautions be observed during construction and maintained at all times after construction is completed.

1. Wetting or drying of open foundation, utility and earthwork excavations should be avoided.

2. Positive drainage should be provided away from the building. Paved surfaces should be sloped to drain away from the buildings. A minimum slope of 5 percent is suggested. More slope is desirable. Final grading should be carefully controlled so that the designed cross slopes are maintained and low spots in the subgrade that could trap water are eliminated.

3. Concrete sidewalks and flatwork may “dam” surface runoff and disrupt proper flow. Use of “chase” drains or weep holes at low points in the curb should be considered to promote proper drainage.

4. Backfill around foundations should be moistened and compacted according to criteria presented in **Fill and Backfill**.

5. Landscaping should be carefully designed to minimize irrigation. We do not recommend providing landscape irrigation within 5 feet from foundations. Plants used close to buildings should be limited to those with low moisture requirements. Irrigation should be limited to the minimum amount sufficient to maintain vegetation. Application of more water will increase likelihood of slab and foundation movements and associated damage. Landscaped areas should be adequately sloped to direct flow away from the improvements. Use of area drains can assist draining areas that cannot be provided with adequate slope.

6. Impervious plastic membranes should not be used to cover the ground surface immediately surrounding foundations. These membranes tend to trap moisture and prevent normal evaporation from occurring. Geotextile fabrics can be used to control weed growth and allow evaporation.

7. Roof drains should be directed away from the structures and discharge beyond backfill zones or into appropriate storm sewer or detention area. Downspout extensions and splash blocks should be provided at all discharge points. Roof drains can also be connected to buried, solid pipe out-lets. Roof drains should not be directed below slab-on-grade floors. Roof drain outlets should be maintained.

CONSTRUCTION OBSERVATIONS

This project will involve activities that should be monitored during the construction phase by a geotechnical engineering firm. To provide continuity between design and construction we recommend that CTL|Thompson, Inc. provide these services. If others perform these services, they must accept responsibility to evaluate whether conditions exposed during construction are consistent with the findings in this report and judge whether the recommendations in this report remain appropriate.

GEOTECHNICAL RISK

The concept of risk is an important aspect with any geotechnical evaluation, primarily because the methods used to develop geotechnical recommendations do not comprise an exact science. We never have complete knowledge of subsurface conditions. Our analysis must be tempered with engineering judgment and experience. Therefore, the recommendations presented in any geotechnical evaluation should not be considered risk-free. Our recommendations represent our judgment of those measures that are necessary to increase the chances that the structure and improvements will perform satisfactorily. It is critical that all recommendations in this report are followed during construction. Owners must assume responsibility for maintaining the structures and use appropriate practices regarding drainage and landscaping. Improvements after construction should be completed in accordance with recommendations provided in this report and may require additional soil investigation and consultation.

LIMITATIONS

This report has been prepared for the exclusive use of Shea Properties and the design team for the purpose of providing geotechnical design and construction criteria for the warehouse buildings project. The information, conclusions and recommendations presented herein are based upon consideration of many factors including, but not limited to, the type of structures proposed, and the subsurface conditions encountered. The conclusions and recommendations contained in the report are not valid for use by others. The recommendations provided are appropriate for about three years. If the proposed project is not constructed within about three years, we should be contacted to determine if we should update this report.

Our borings were located to obtain a reasonably accurate picture of subsurface conditions below the proposed project. The borings are representative of conditions encountered only at the boring locations. Subsurface variations not indicated by our borings are likely. We believe this investigation was conducted in a manner consistent with that level of care and skill ordinarily used by geotechnical engineers practicing under similar conditions. No warranty, express or implied, is made. If we can be of further service in discussing the contents of this report, or in the analysis of the influence of the subsurface conditions on the design of the structure and improvements, or any other aspect of the proposed construction, please call or email.

CTL|THOMPSON, INC.

COLORADO LICENSED
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2/23/2024

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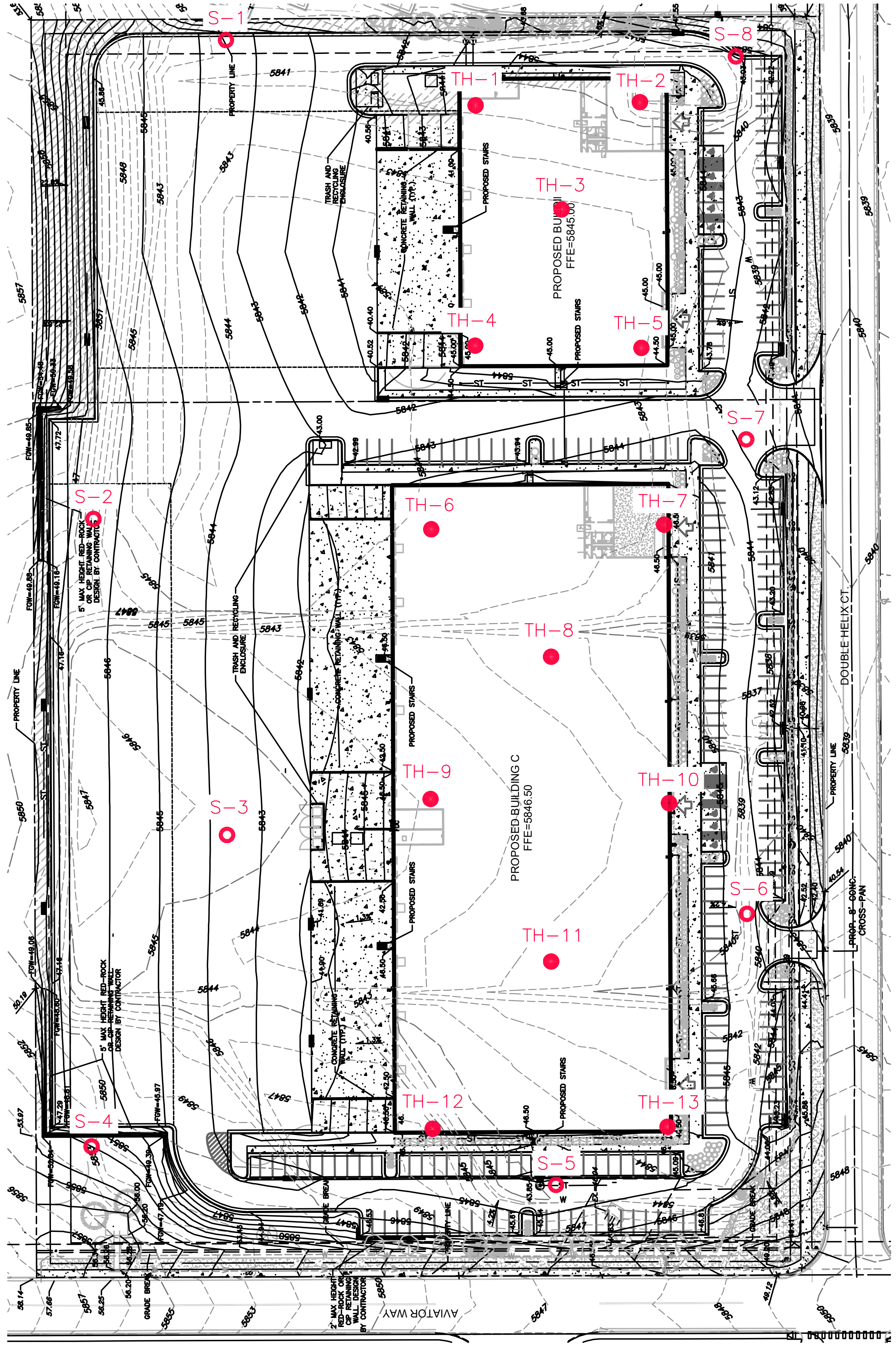
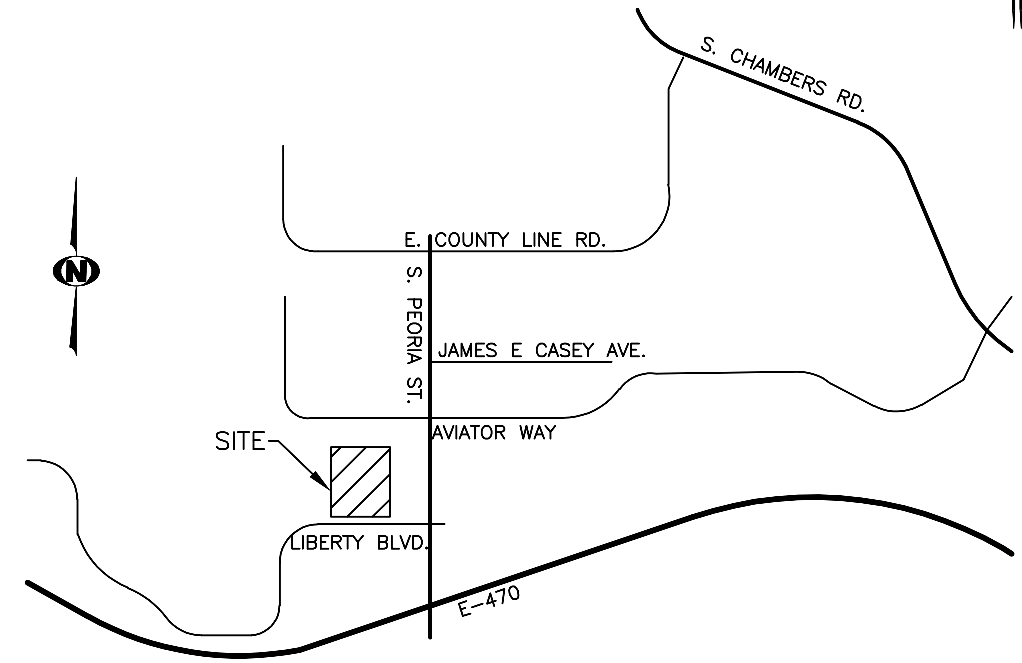




VICINITY MAP
NOT TO SCALE

LEGEND:

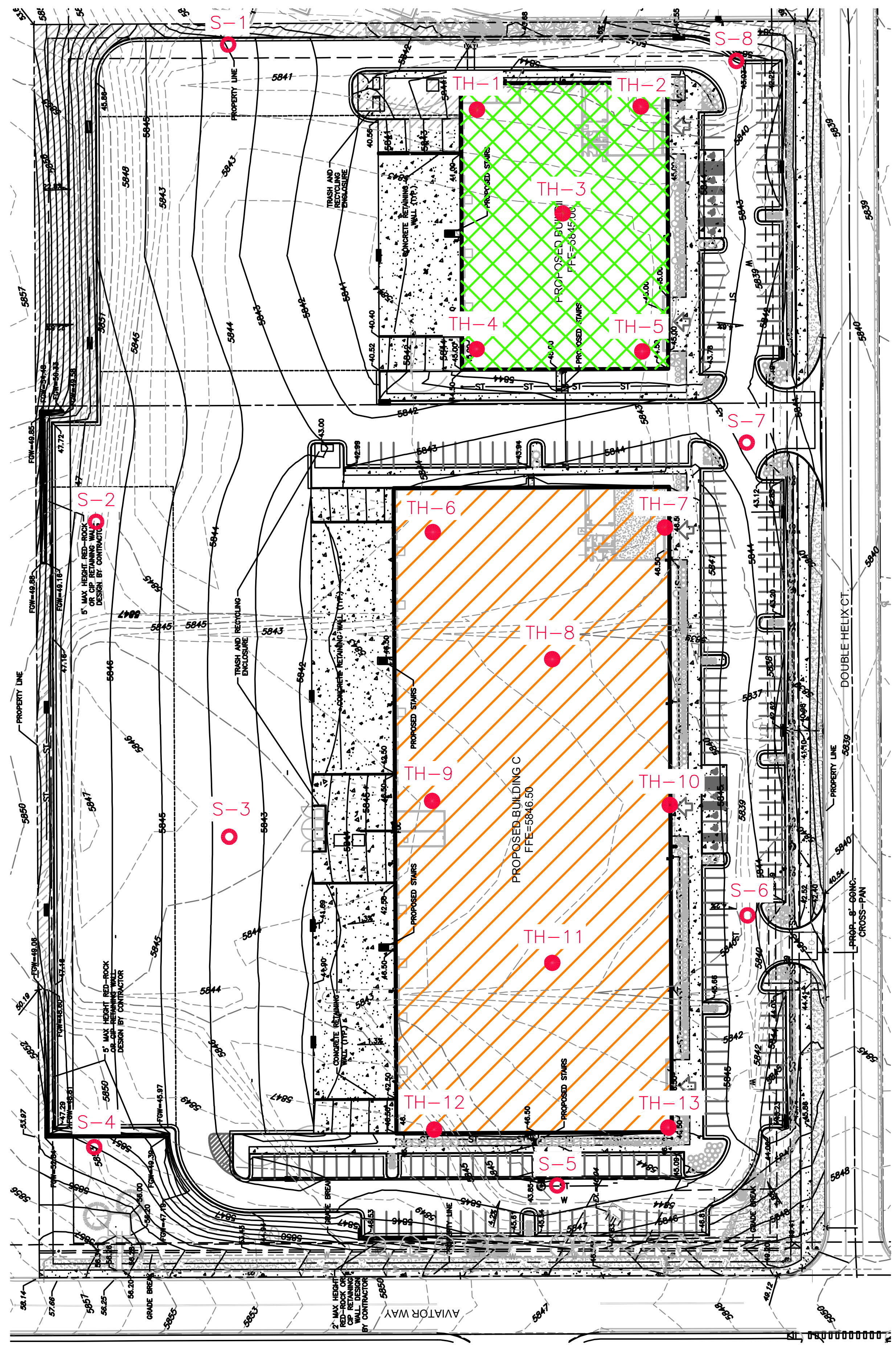
TH-1 ●	APPROXIMATE LOCATION OF EXPLORATORY BORING
S-1 ○	APPROXIMATE LOCATION OF PAVEMENT BORING

0 50 100
SCALE: 1" = 100'

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T Project No. DN51,883-125-R1

Locations of Exploratory Borings

Fig. 1



LEGEND:

TH-1 APPROXIMATE LOCATION OF EXPLORATORY BORING

S-1 APPROXIMATE LOCATION OF PAVEMENT BORING

SUB-EXCAVATE TO 10 FEET BELOW BOTTOM OF LOWEST FOUNDATION ELEMENT

SUB-EXCAVATE TO 12 FEET BELOW BOTTOM OF LOWEST FOUNDATION ELEMENT*

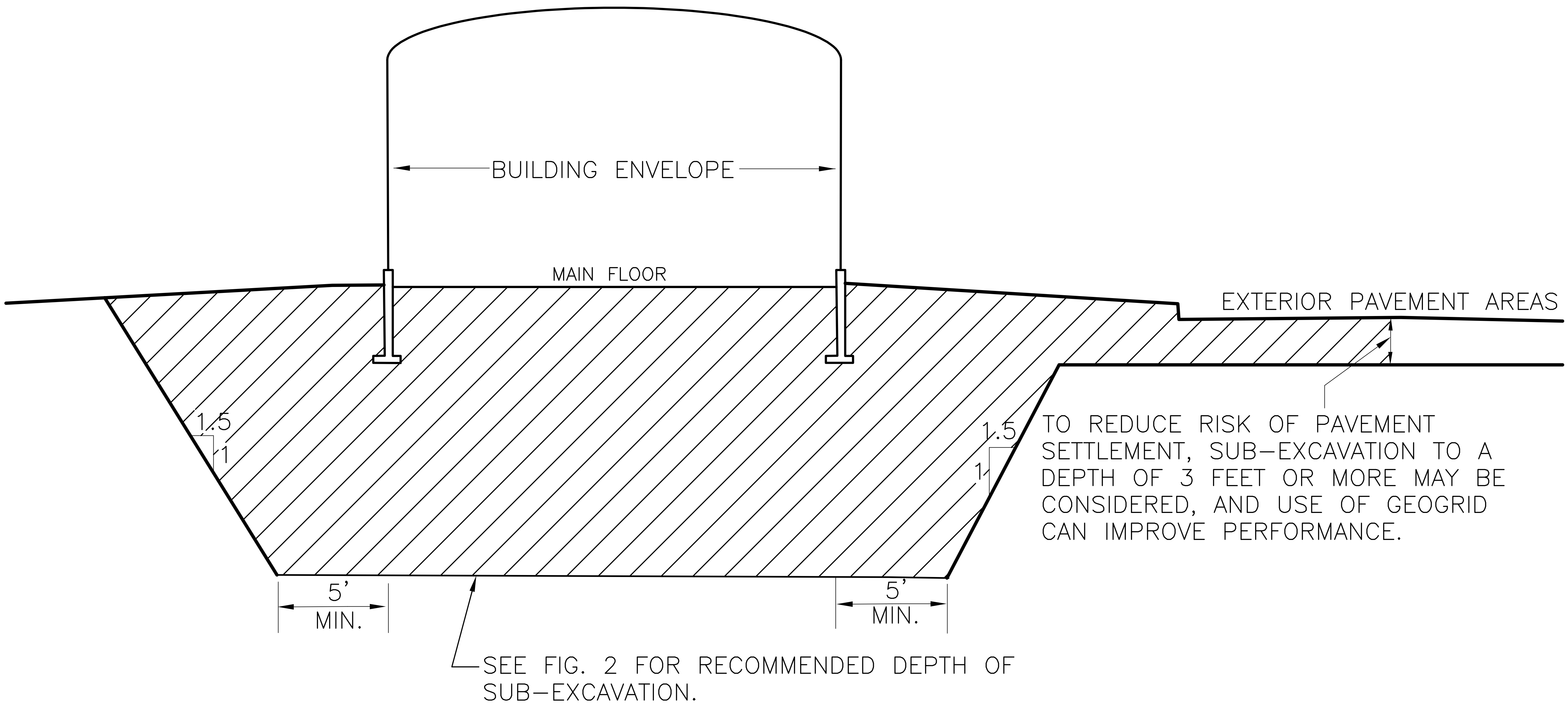
*EXISTING (UNDOCUMENTED) FILL WAS ENCOUNTERED IN OUR BORINGS FOR THIS BUILDING. MORE FILL MAY BE PRESENT THAN OUR BORINGS IMPLY, IF EXISTING FILL IS OBSERVED IN THE EXCAVATION BOTTOM, SUB-EXCAVATION SHOULD BE DEEPENED UNTIL NATURAL SOILS ARE ENCOUNTERED, AND SHOULD EXTENT ACROSS THE ENTIRE BUILDING FOOTPRINT.

0 50 100
SCALE: 1" = 100'

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T Project No. DN51,883-125-R1

Sub-Excavation Recommendation

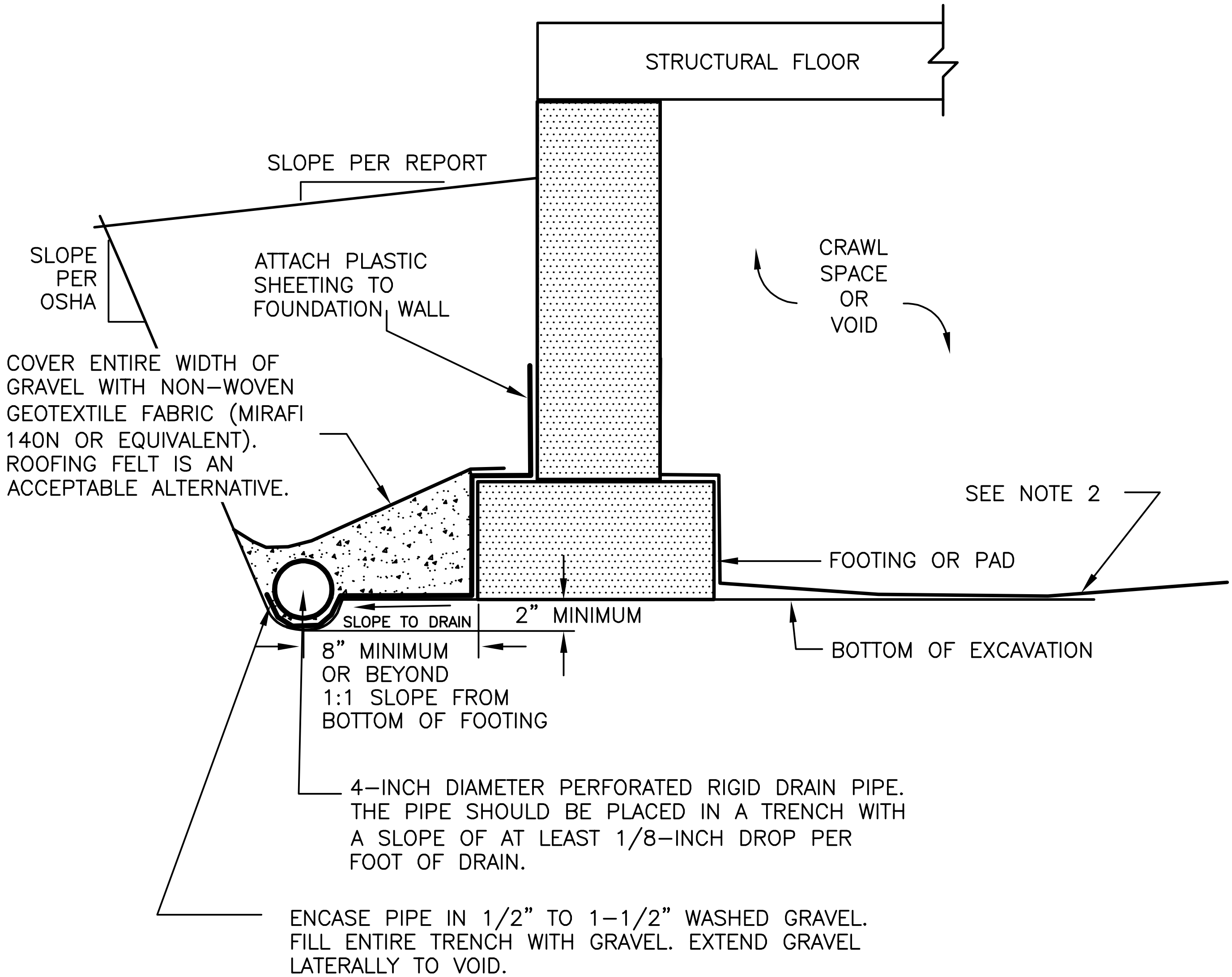
Fig. 2



SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T Project No. DN51,883-125-R1

Conceptual Sub-Excavation Profile

Fig. 3



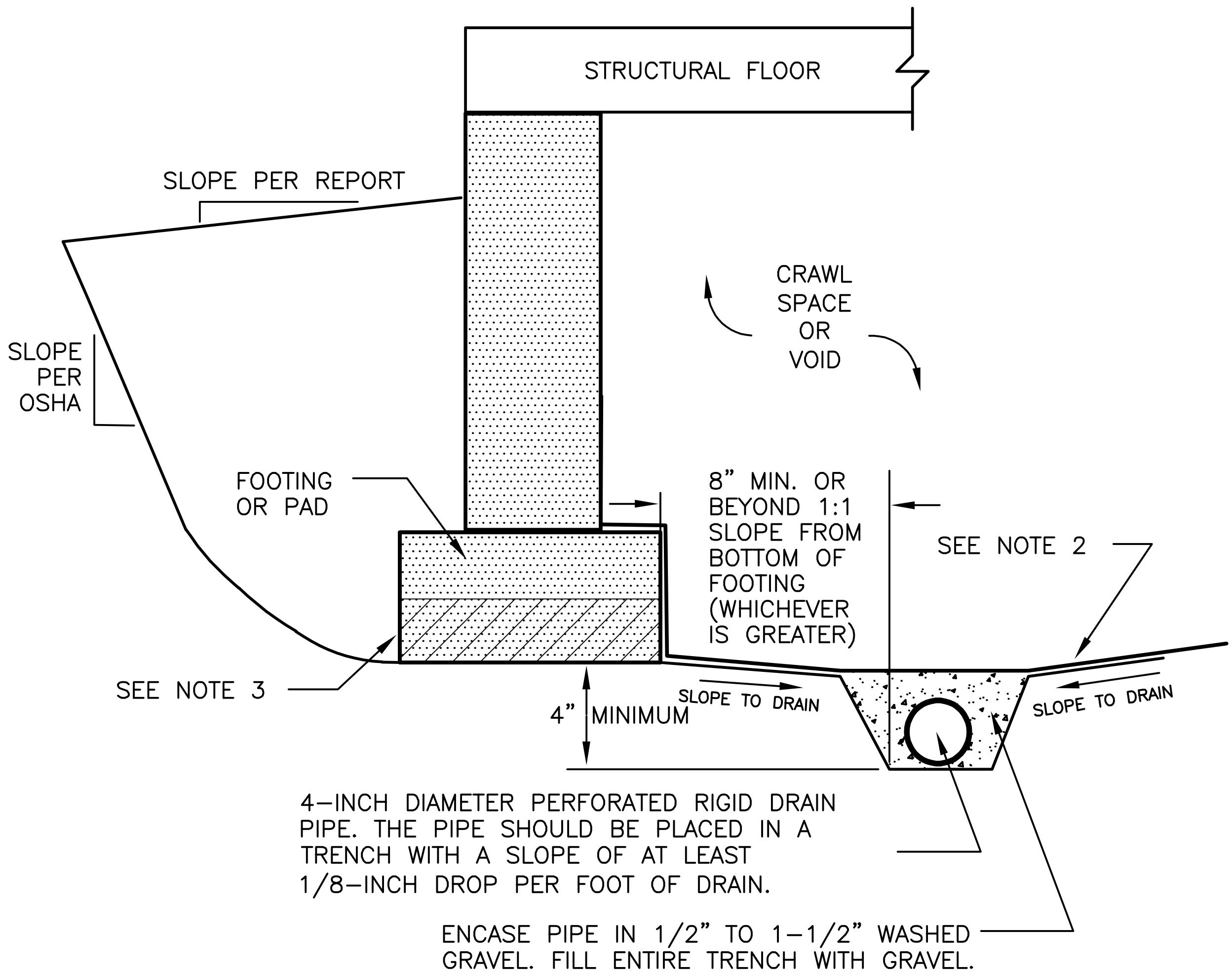
NOTES:

1) THE BOTTOM OF THE DRAIN SHOULD BE AT LEAST 4 INCHES BELOW BOTTOM 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) TO HELP CONTROL THE HUMIDITY IN THE CRAWL SPACE, A MINIMUM 6-MIL POLYETHYLENE VAPOR RETARDER SHOULD BE PLACED OVER THE CRAWL SPACE SOILS. THE RETARDER SHOULD BE ATTACHED TO CONCRETE FOUNDATION ELEMENTS AND EXTEND UP FOUNDATION WALLS AT LEAST 8 INCHES ABOVE TOP OF FOOTING. OVERLAP JOINTS 3 FEET AND SEAL.

Exterior Foundation Wall Drain

01-FTG-INT-STF-01



NOTES:

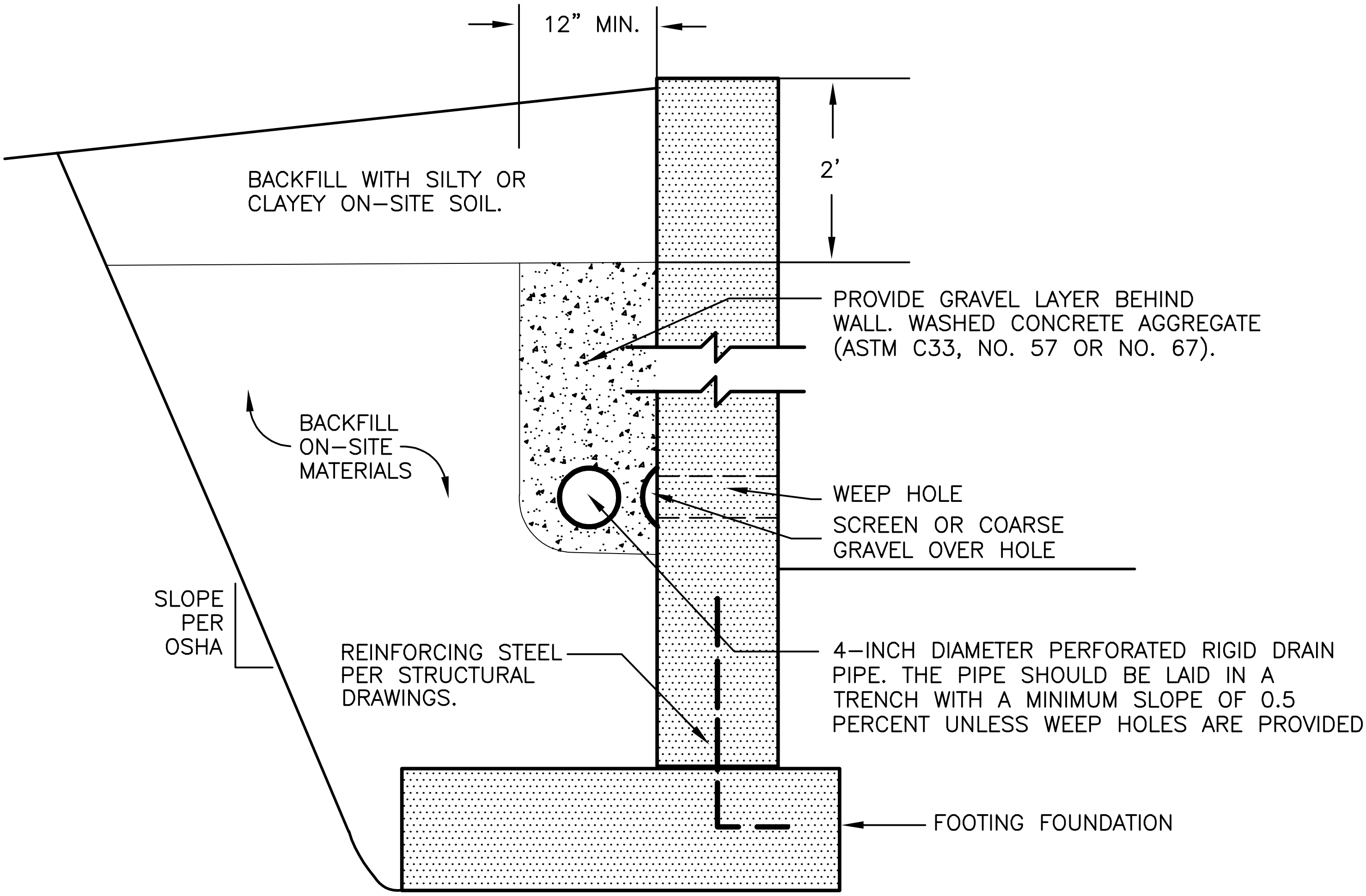
1) THE BOTTOM OF THE DRAIN SHOULD BE AT LEAST 4 INCHES BELOW BOTTOM OF FOOTING AT THE HIGHEST POINT AND SLOPE DOWNWARD TO A POSITIVE GRAVITY OUTLET OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPING.

2) TO HELP CONTROL THE HUMIDITY IN THE CRAWL SPACE, A MINIMUM 6-MIL, AND PREFERABLY 10-MIL POLYETHYLENE VAPOR RETARDER SHOULD BE PLACED OVER THE CRAWL SPACE SOILS. THE RETARDER SHOULD BE ATTACHED TO CONCRETE FOUNDATION ELEMENTS AND EXTEND UP FOUNDATION WALLS AT LEAST 8 INCHES ABOVE TOP OF FOOTING. OVERLAP JOINTS 3 FEET AND SEAL.

3) FOR FOOTINGS IN CRAWL SPACE AREAS, WE RECOMMEND PLACING A 4-INCH THICK, 8-INCH WIDE SECTION OF VOID FORM PERPENDICULAR TO FOOTING ABOUT EVERY 10 TO 15 FEET, TO ALLOW WATER IN WALL BACKFILL TO PASS BENEATH THE FOOTING INTO THE INTERIOR DRAIN. THIS CAN ALSO BE ACCOMPLISHED BY "TUNNELING" UNDER FOOTINGS AT THE TIME OF DRAIN INSTALLATION. THIS DETAIL SHOULD BE REVIEWED BY THE STRUCTURAL ENGINEER DURING FOUNDATION DESIGN AND INCORPORATED INTO THE FOUNDATION PLAN. ALTERNATIVELY, AN EXTERIOR FOUNDATION DRAIN CAN BE USED.

Interior Foundation Wall Drain

35-RETAINING-WALL_01



NOTE:
DRAIN PIPE TO GRAVITY OUTLET OR WEEP HOLES MAY BE USED.

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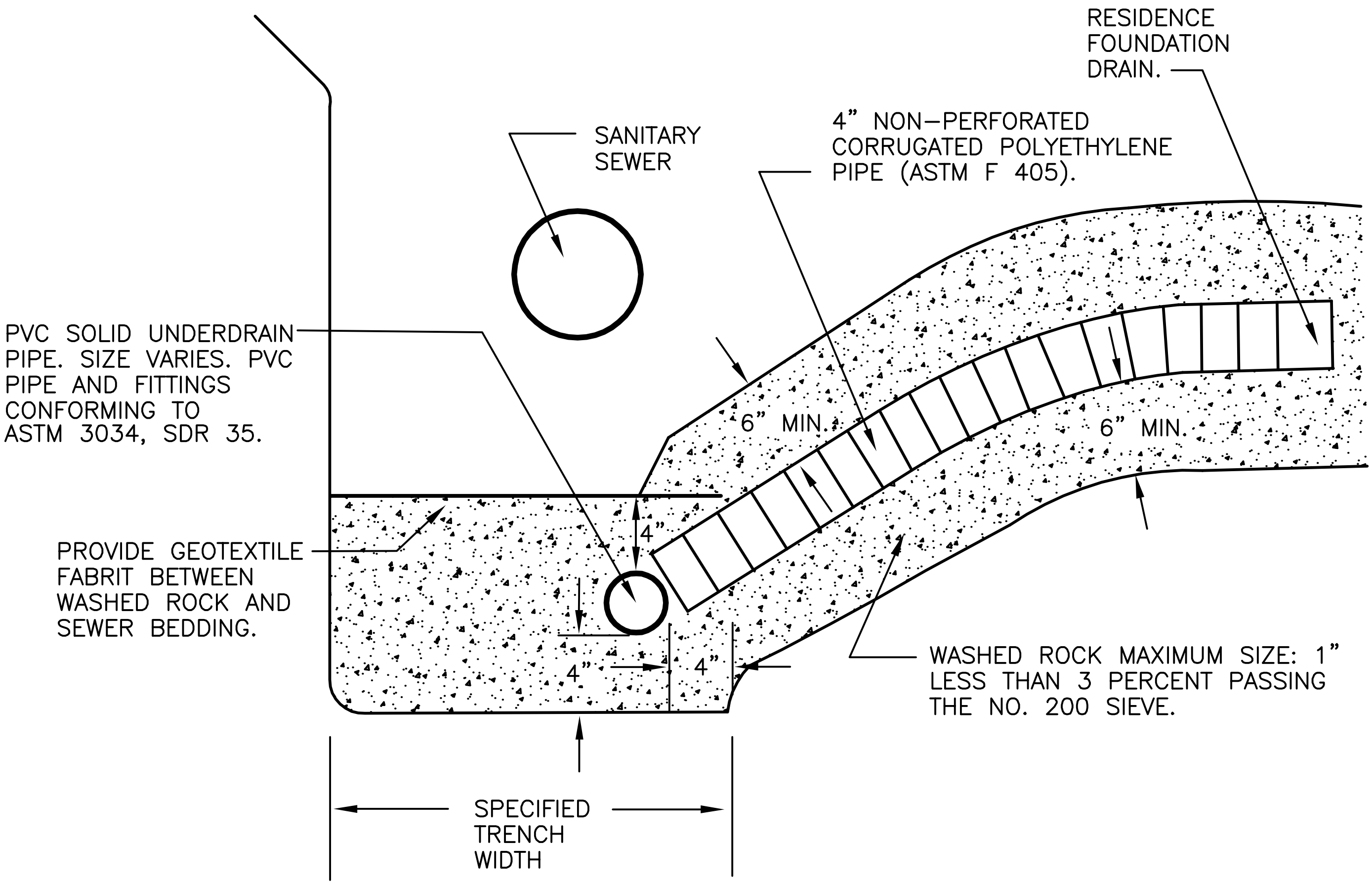
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T Project No. DN51,883-125-R1

Typical Earth Retaining Wall Drain

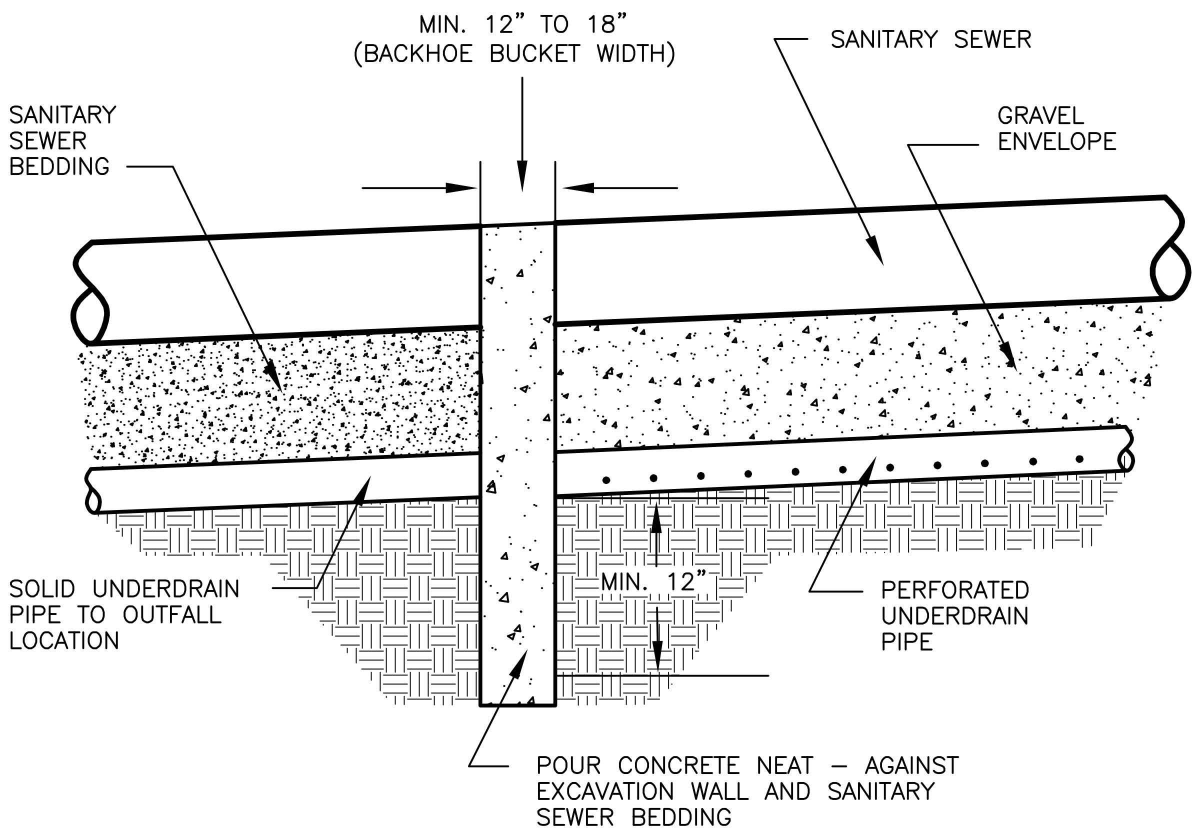
Fig. 6



NOTE: NOT TO SCALE.

Sewer Underdrain Detail

Fig. 7



NOTE:
THE CONCRETE CUTOFF WALL SHOULD EXTEND INTO THE UNDISTURBED SOILS OUTSIDE THE UNDERDRAIN AND SANITARY SEWER TRENCH A MINIMUM DISTANCE OF 12 INCHES.

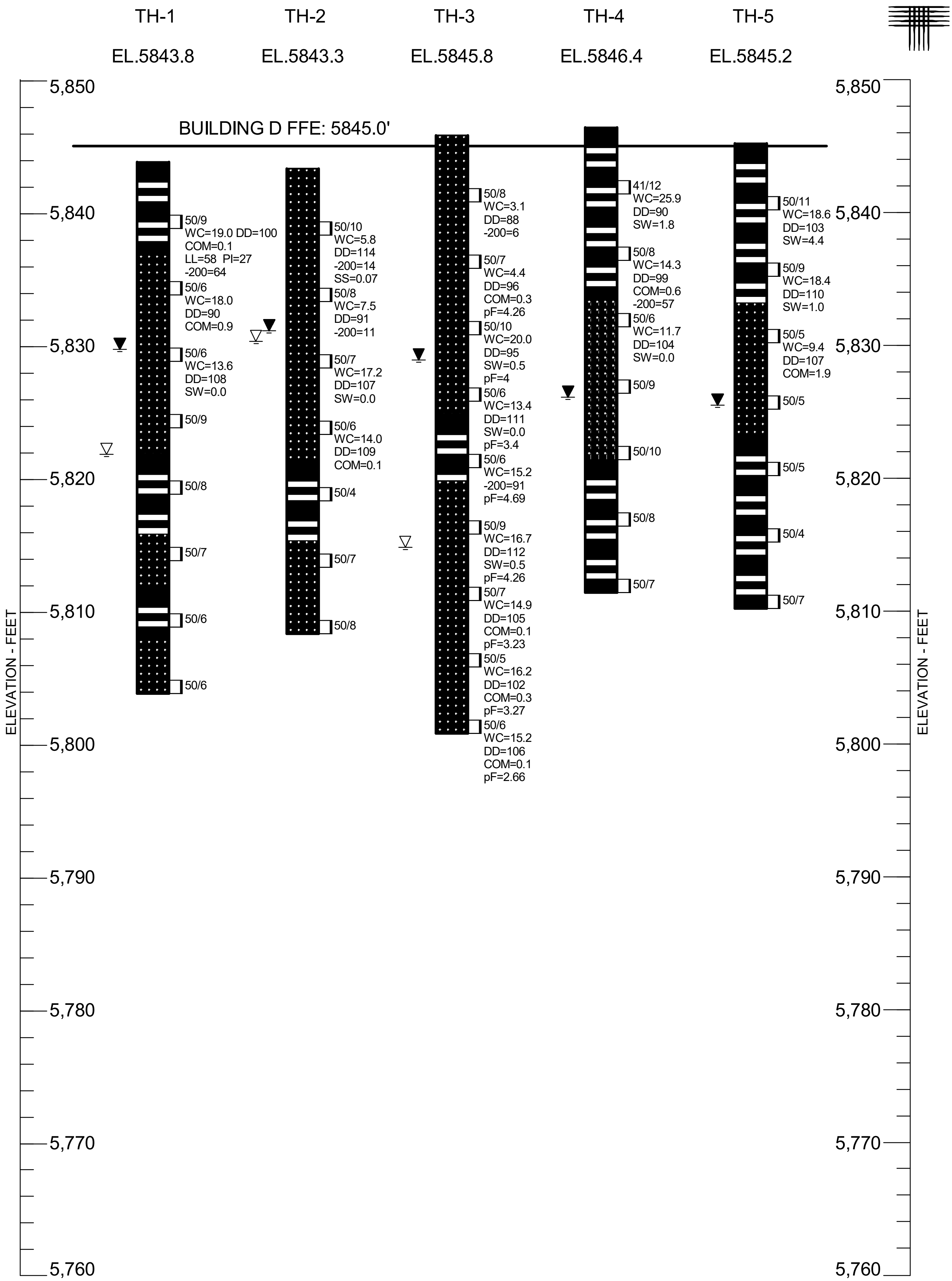
SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T Project No. DN51,883-125-R1

Underdrain Cutoff Wall Detail

Fig. 8

APPENDIX A

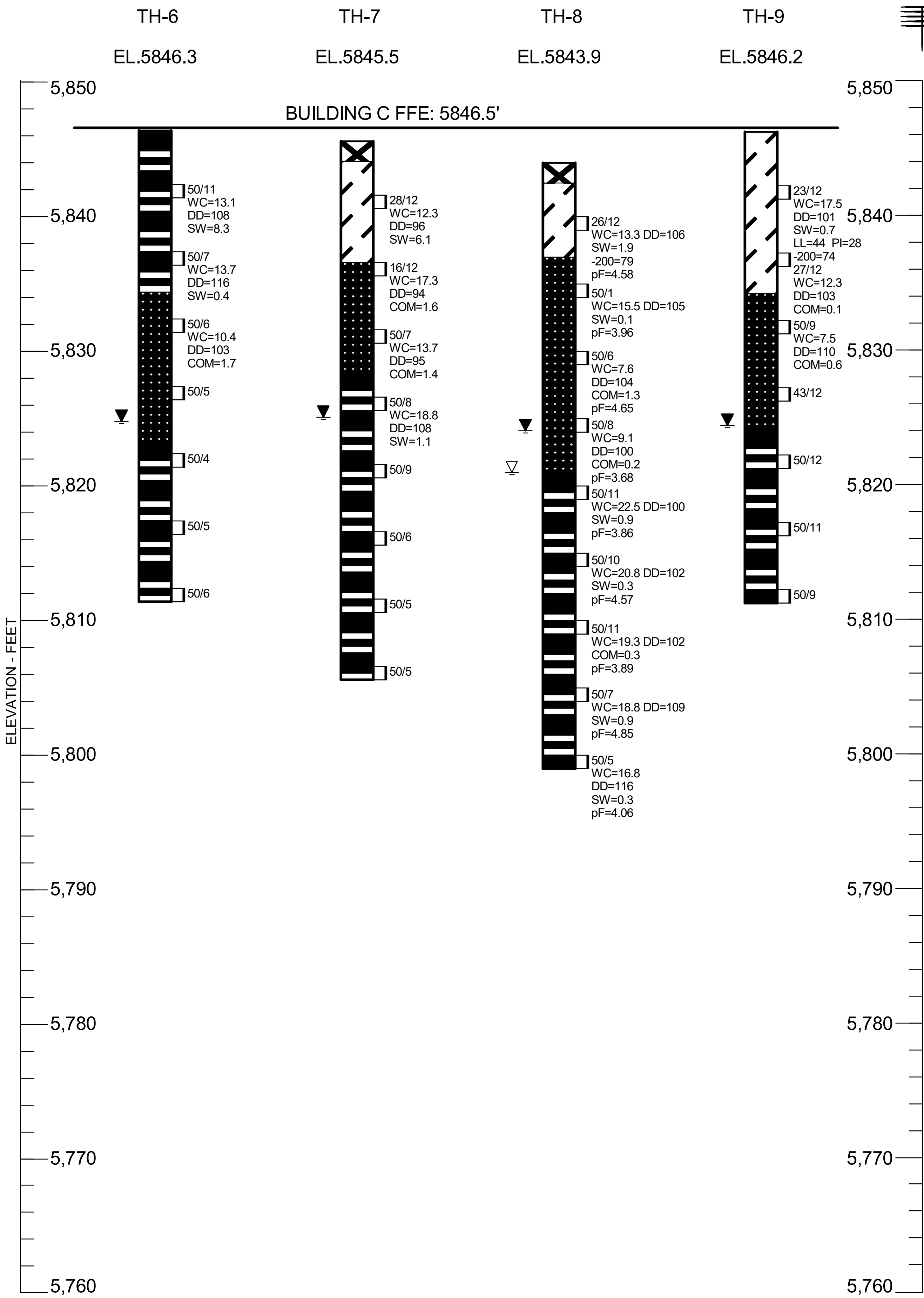
SUMMARY LOGS OF EXPLORATORY BORINGS



SUMMARY LOGS OF EXPLORATORY BORINGS

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

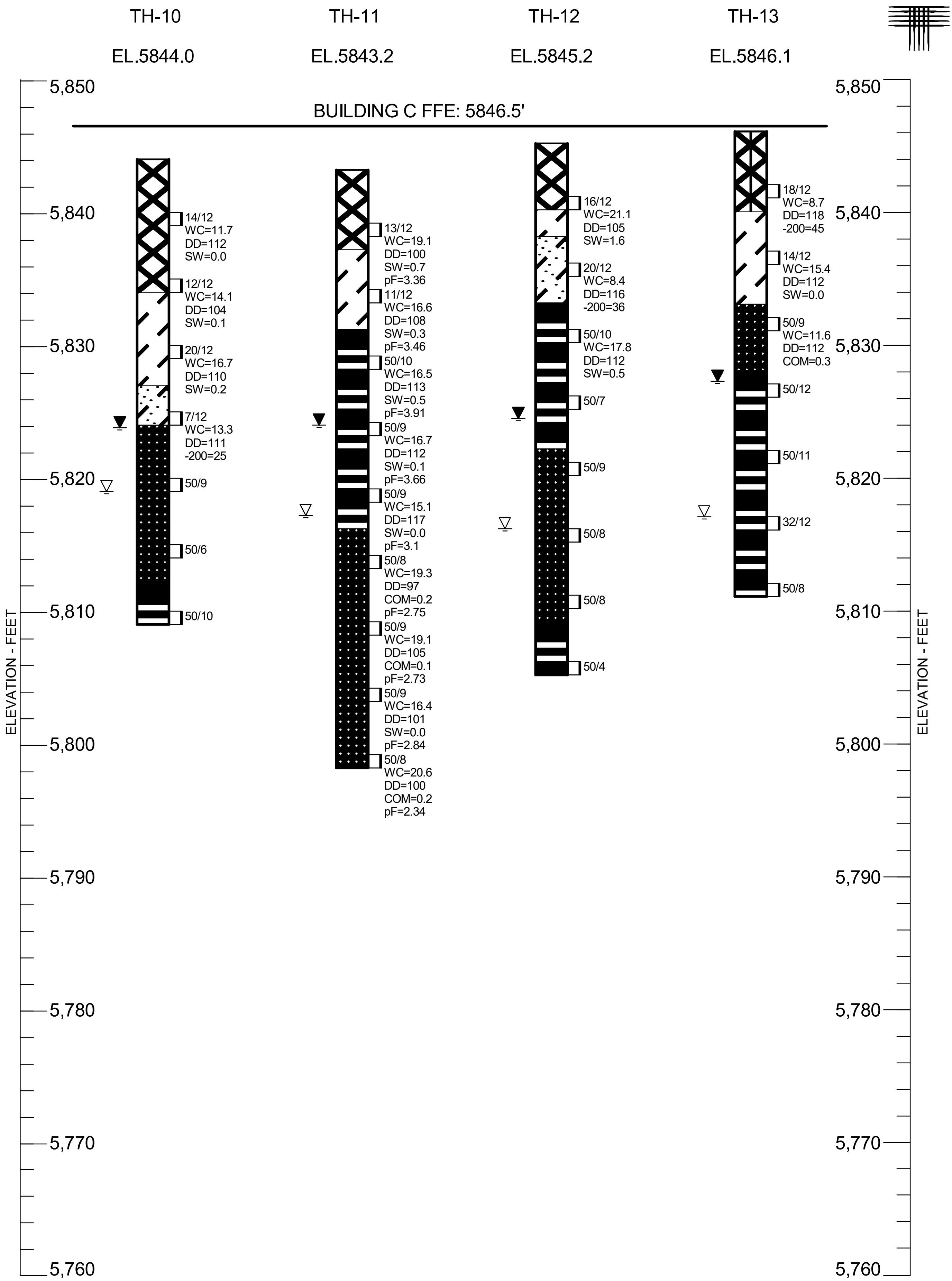
FIG. A-1



SUMMARY LOGS OF EXPLORATORY BORINGS

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

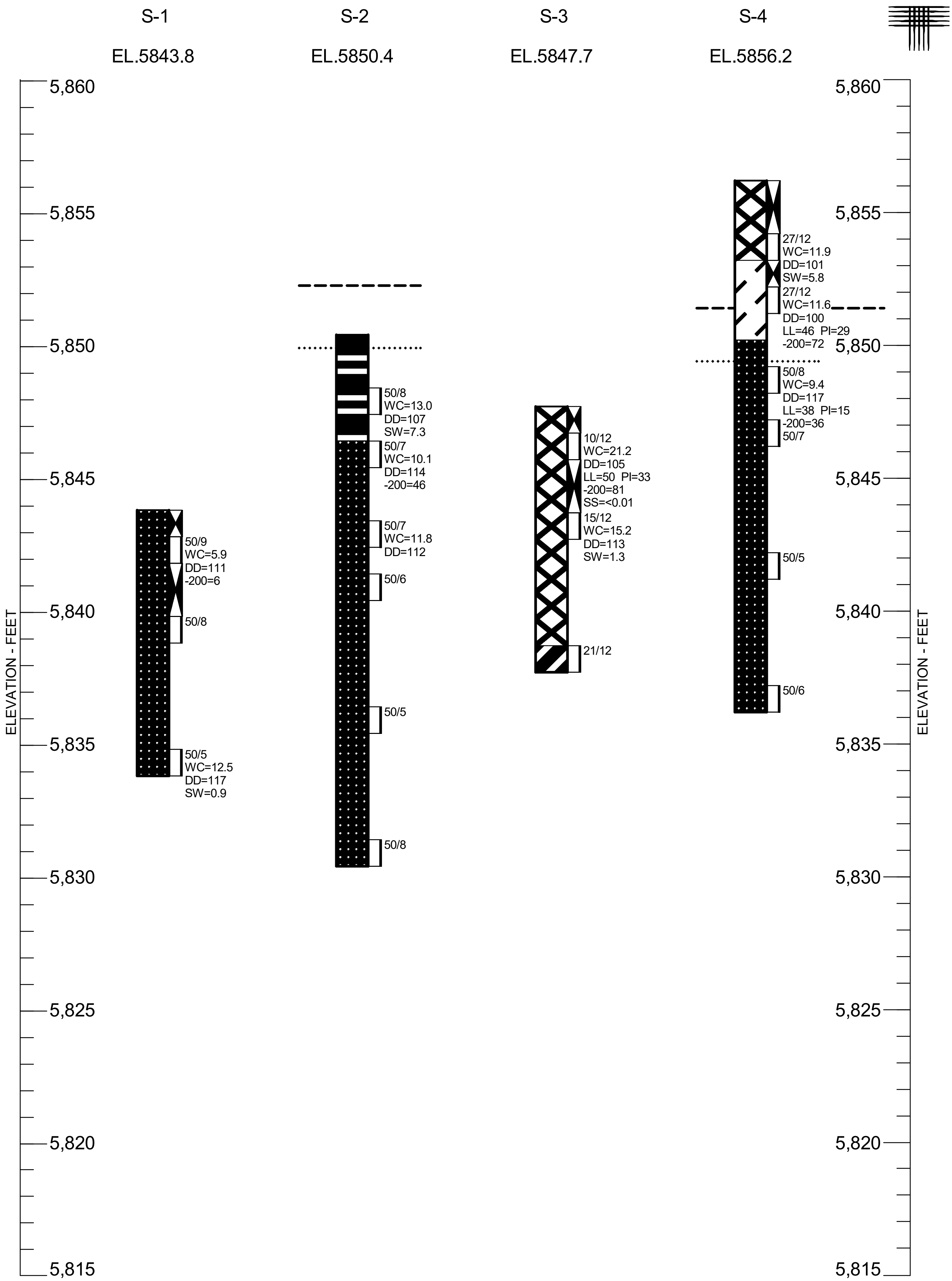
FIG. A-2



SUMMARY LOGS OF EXPLORATORY BORINGS

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

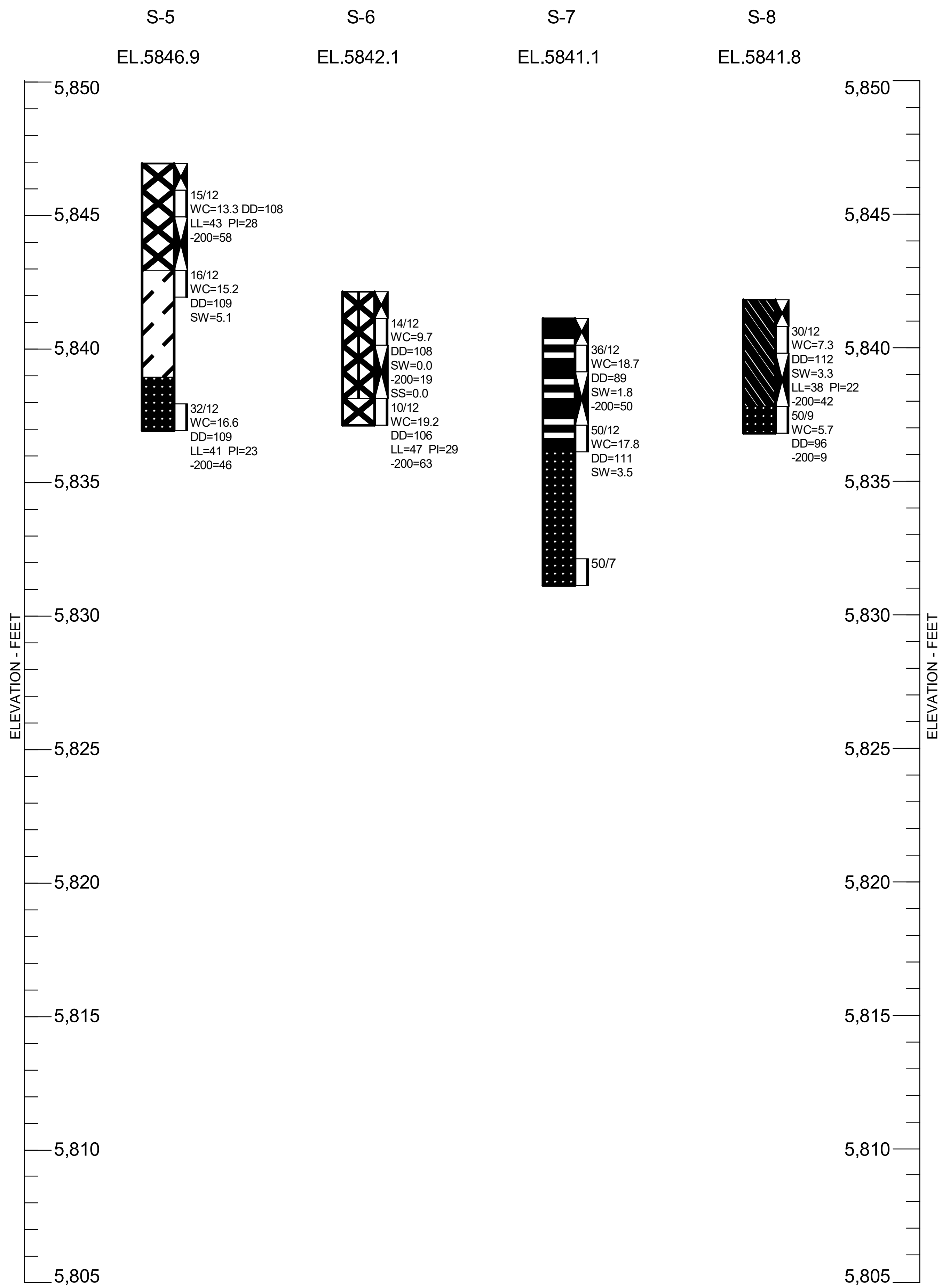
FIG. A-3



SUMMARY LOGS OF EXPLORATORY BORINGS

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

FIG. A-4



SUMMARY LOGS OF EXPLORATORY BORINGS

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

LEGEND:

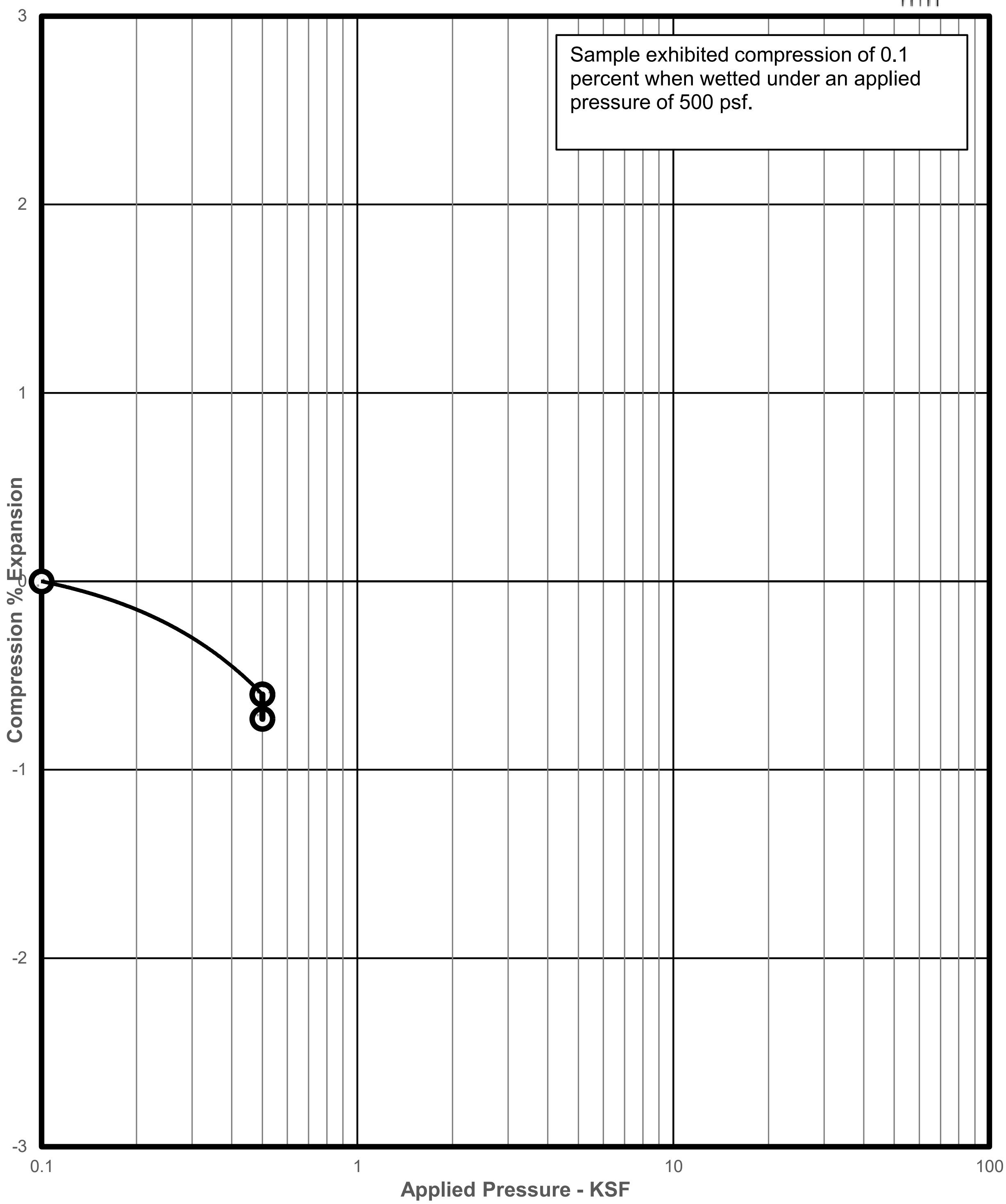
- FILL, CLAY, SANDY OR SAND, CLAYEY, STIFF TO VERY STIFF, MEDIUM DENSE, DRY TO MOIST, BROWN, DARK BROWN.
- CLAY, SANDY, STIFF TO VERY STIFF, SLIGHTLY MOIST, BROWN (CL).
- SAND, CLAYEY, MEDIUM DENSE, SLIGHTLY MOIST, BROWN, GRAY (SC).
- BEDROCK, SANDSTONE, MEDIUM HARD TO VERY HARD, SLIGHTLY MOIST TO MOIST, BROWN, RED, GRAY.
- BEDROCK, CLAYSTONE, MEDIUM HARD TO VERY HARD, SLIGHTLY MOIST TO MOIST, BROWN, GRAY, REDDISH GRAY, RUST.
- DRIVE SAMPLE. THE SYMBOL 50/9 INDICATES 50 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2.5-INCH O.D. SAMPLER 9 INCHES.
- BULK SAMPLE COLLECTED FROM AUGER CUTTINGS.
- GROUNDWATER LEVEL MEASURED AT TIME OF DRILLING.
- GROUNDWATER LEVEL MEASURED AFTER DRILLING ON APRIL 19, 2023.
- INDICATES FINISHED FLOOR ELEVATION.
- INDICATES TOP OF WALL.
- INDICATES BOTTOM OF WALL.

NOTES:

1. THE BORINGS WERE DRILLED BETWEEN MARCH 15 AND 20, 2023 USING A 4-INCH DIAMETER, CONTINUOUS-FLIGHT AUGER AND A CME-45, TRUCK-MOUNTED DRILL RIG.
2. BORING LOCATIONS AND ELEVATIONS ARE APPROXIMATE AND WERE DETERMINED BY A REPRESENTATIVE OF OUR FIRM USING A LEICA GS18 GPS UNIT REFERENCING THE NORTH AMERICAN DATUM OF 1983 (NAD83).
3. WC - INDICATES MOISTURE CONTENT (%).
   DD - INDICATES DRY DENSITY (PCF).
   SW - INDICATES SWELL WHEN WETTED UNDER APPROXIMATE OVERBURDEN PRESSURE (%).
   LL - INDICATES LIQUID LIMIT.
   PI - INDICATES PLASTICITY INDEX.
   -200 - INDICATES PASSING NO. 200 SIEVE (%).
   SS - INDICATES WATER-SOLUBLE SULFATE CONTENT (%).
   pF - INDICATES SOIL SUCTION VALUE (PSF).
4. THESE LOGS ARE SUBJECT TO THE EXPLANATIONS, LIMITATIONS, AND CONCLUSIONS AS CONTAINED IN THIS REPORT.

APPENDIX B

LABORATORY TEST RESULTS AND TABLE B-I



SAMPLE OF CLAYSTONE

FROM: TH-1 AT 4 FEET

DRY UNIT WEIGHT: 100 pcf

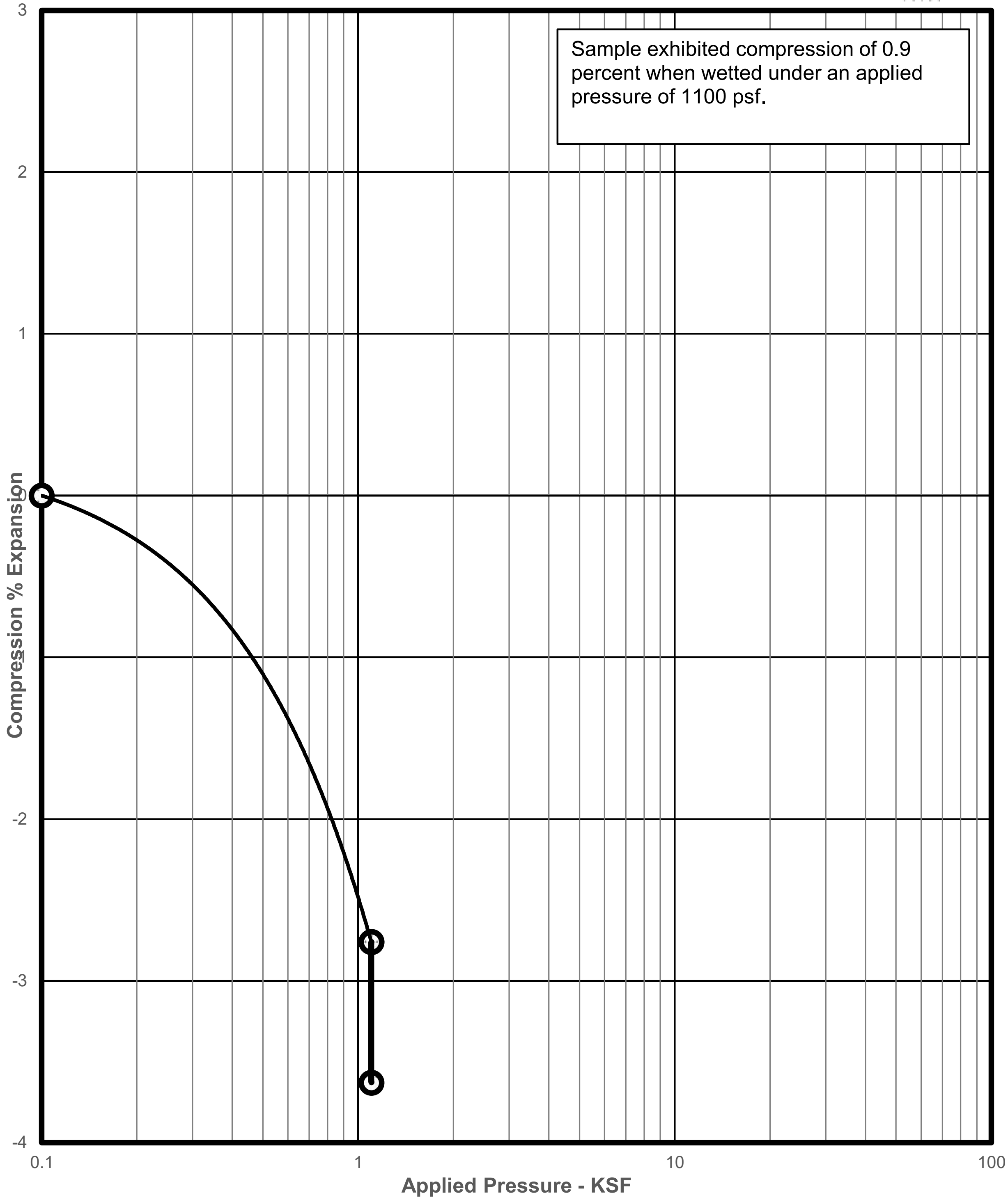
MOISTURE CONTENT: 19.0 %

SHEA PROPERTIES

HELIX INDUSTRIAL BUILDINGS C AND D

CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 1

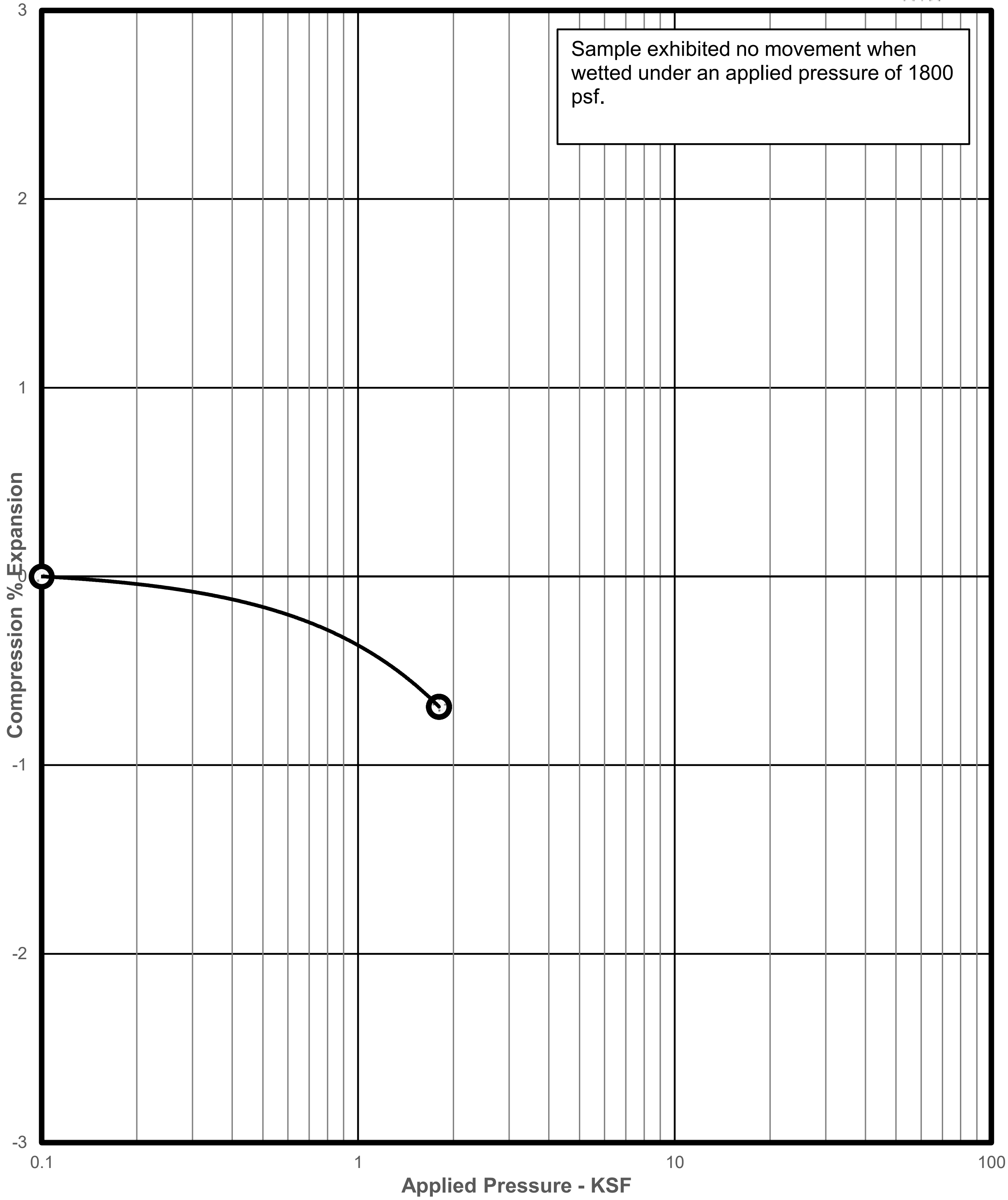


SAMPLE OF SANDSTONE
FROM: TH-1 AT 9 FEET

DRY UNIT WEIGHT: 90 pcf
MOISTURE CONTENT: 18.0 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 2

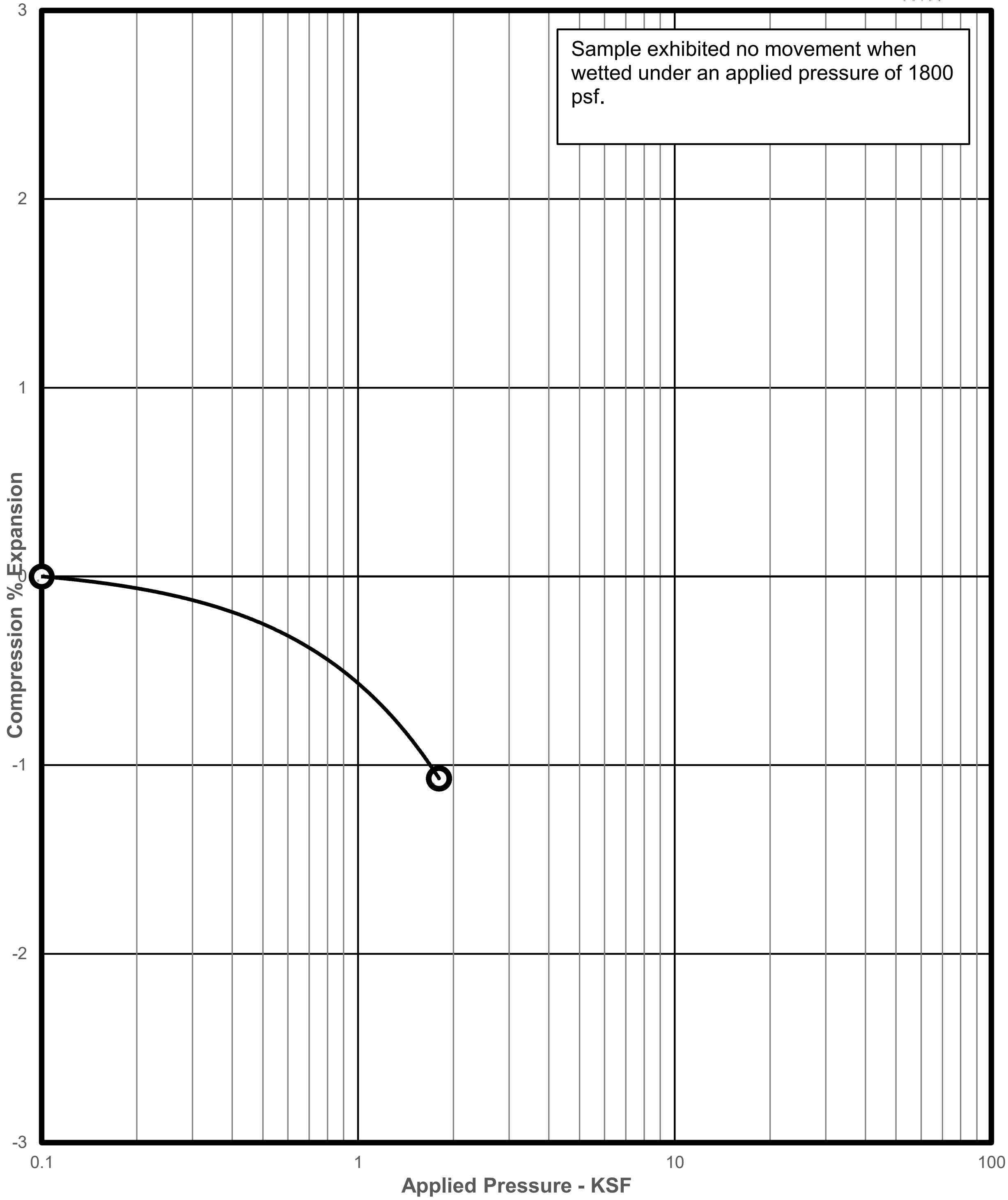


SAMPLE OF: SANDSTONE
FROM: TH-1 AT 14 FEET

DRY UNIT WEIGHT: 108 pcf
MOISTURE CONTENT: 13.6 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 3

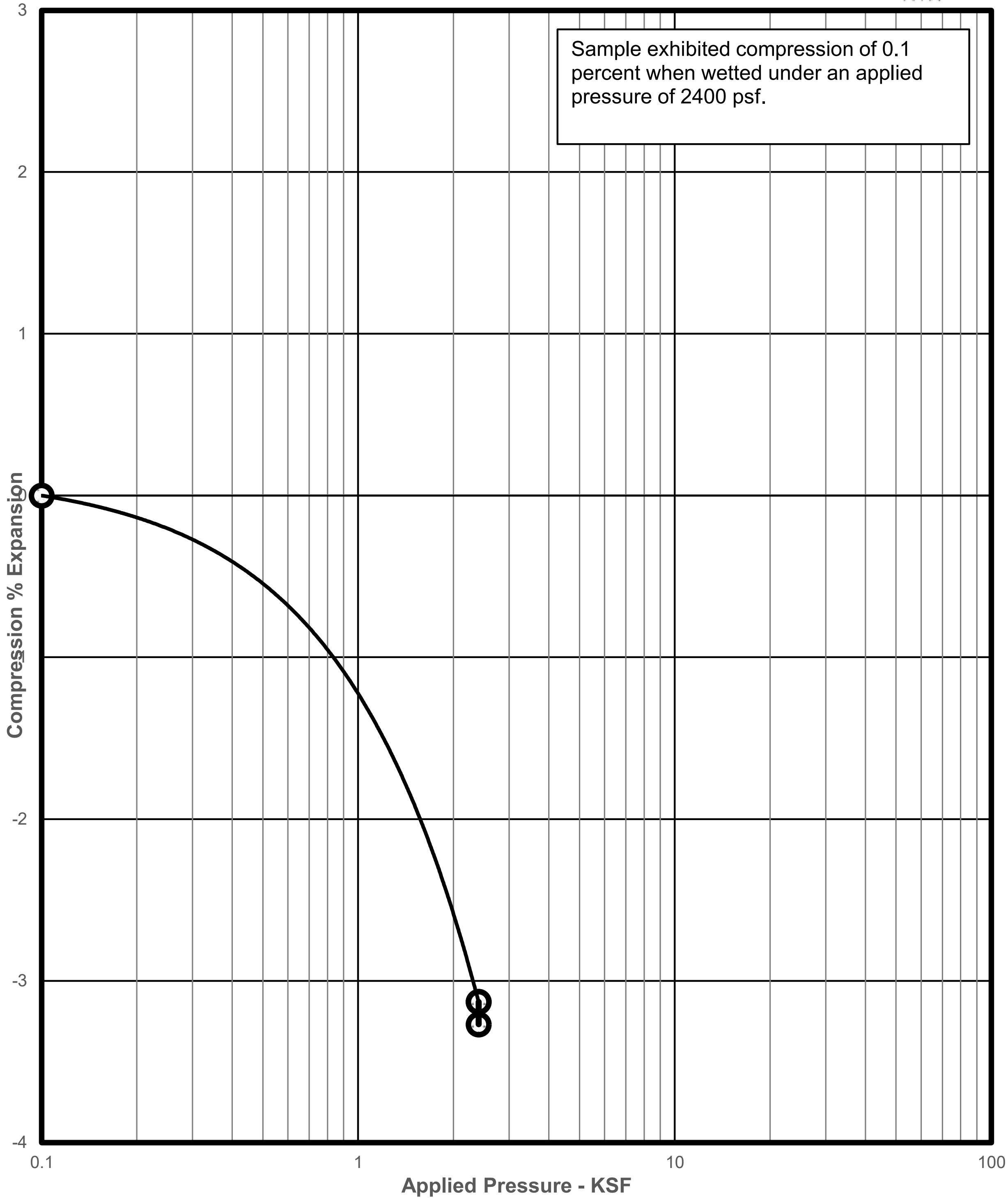


SAMPLE OF SANDSTONE
FROM: TH-2 AT 14 FEET

DRY UNIT WEIGHT: 107 pcf
MOISTURE CONTENT: 17.2 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 4

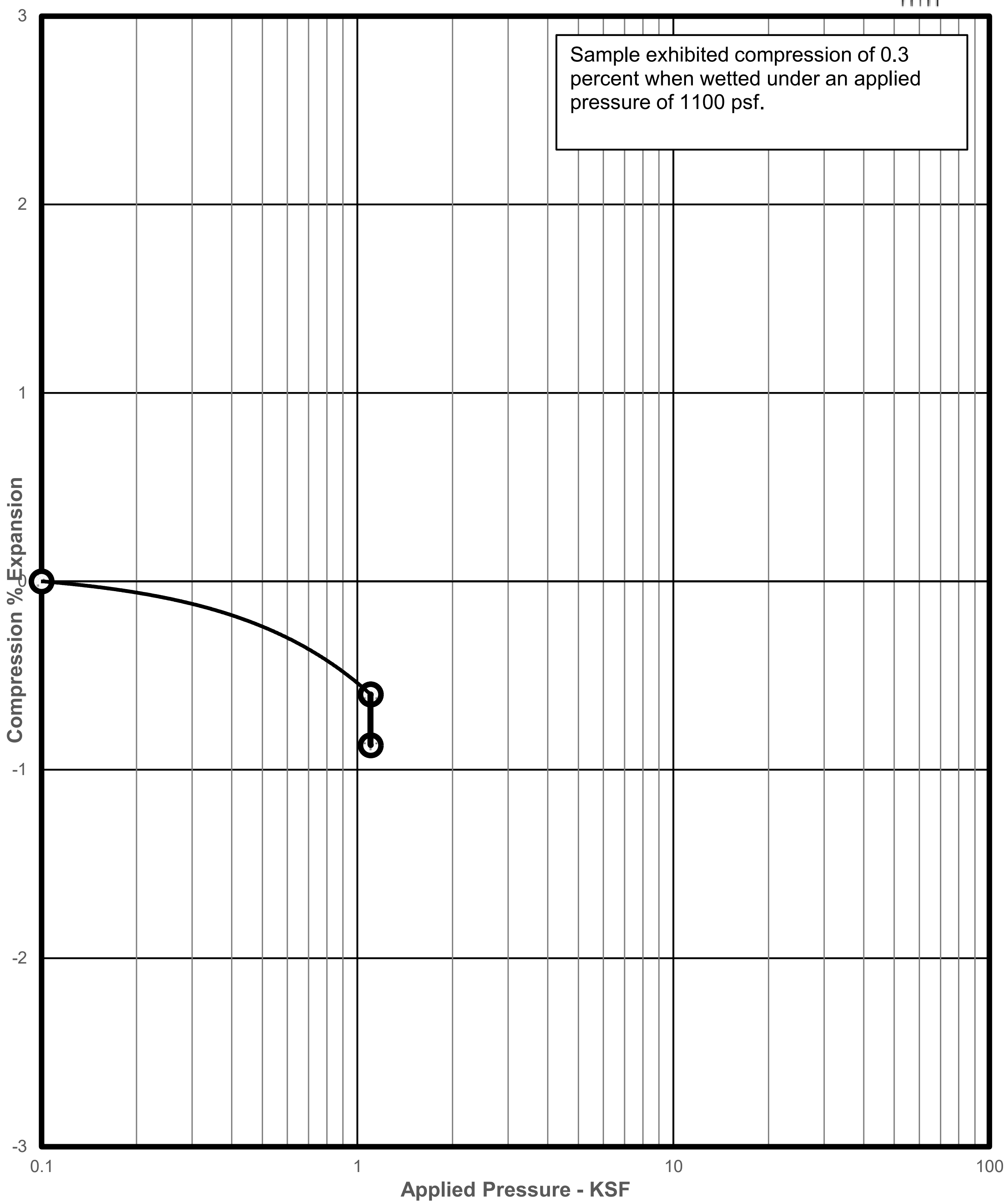


SAMPLE OF SANDSTONE
FROM: TH-2 AT 19 FEET

DRY UNIT WEIGHT: 109 pcf
MOISTURE CONTENT: 14.0 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 5

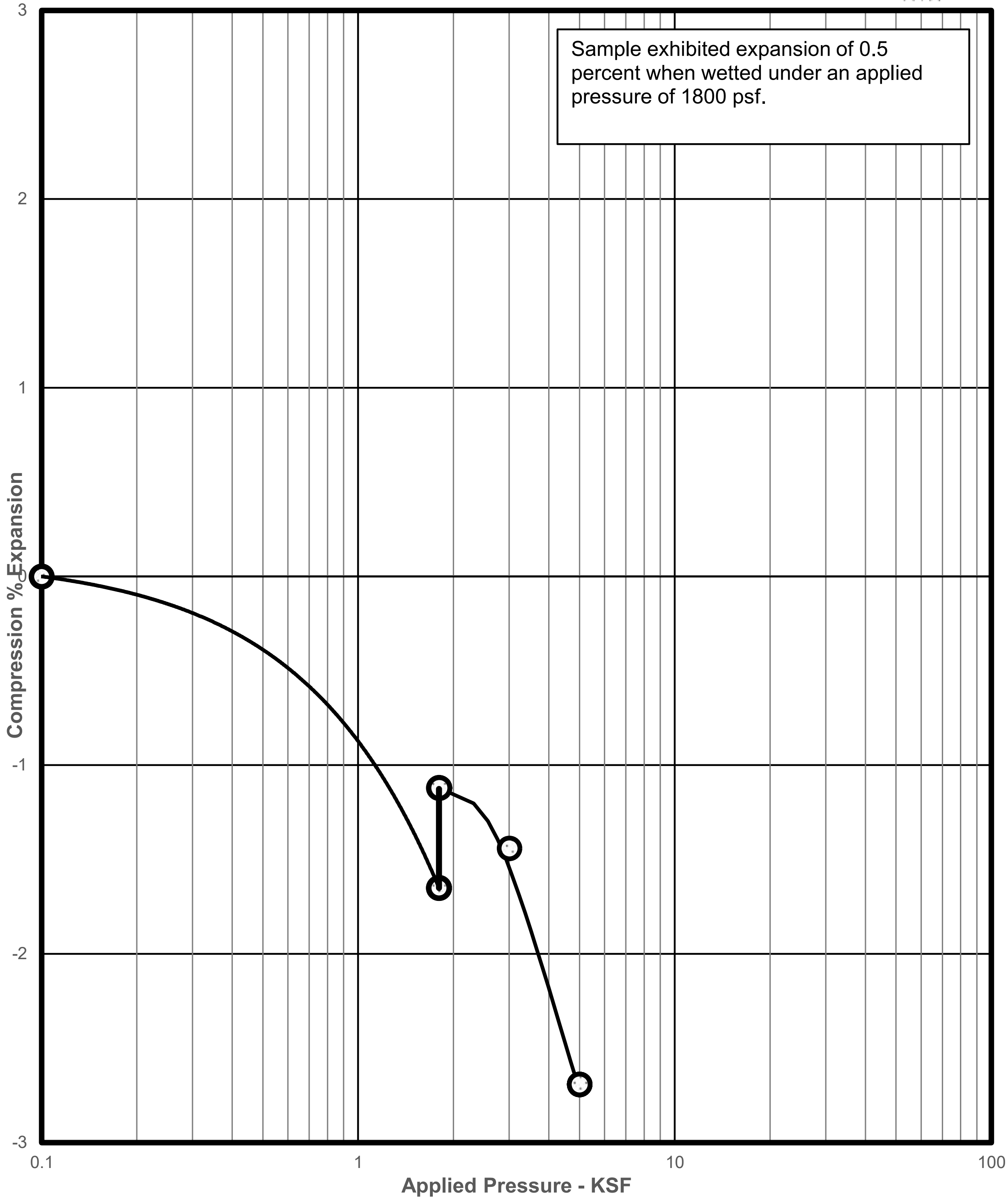


SAMPLE OF SANDSTONE
FROM: TH-3 AT 9 FEET

DRY UNIT WEIGHT: 96 pcf
MOISTURE CONTENT: 4.4 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

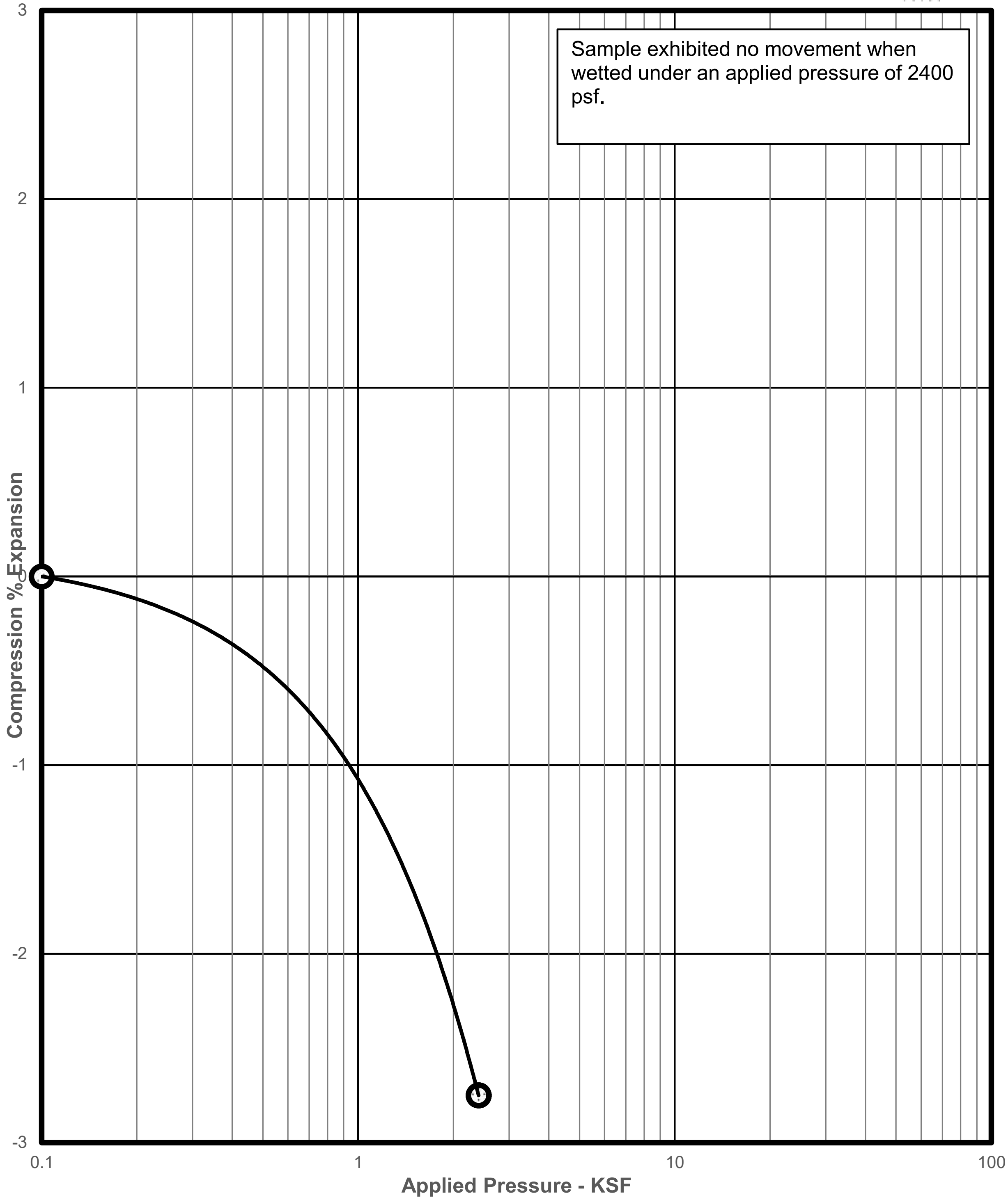
Swell Consolidation Test Results FIG. B- 6



SAMPLE OF SANDSTONE | DRY UNIT WEIGHT: 95 pcf
FROM: TH-3 AT 14 FEET | MOISTURE CONTENT: 20.0 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 7

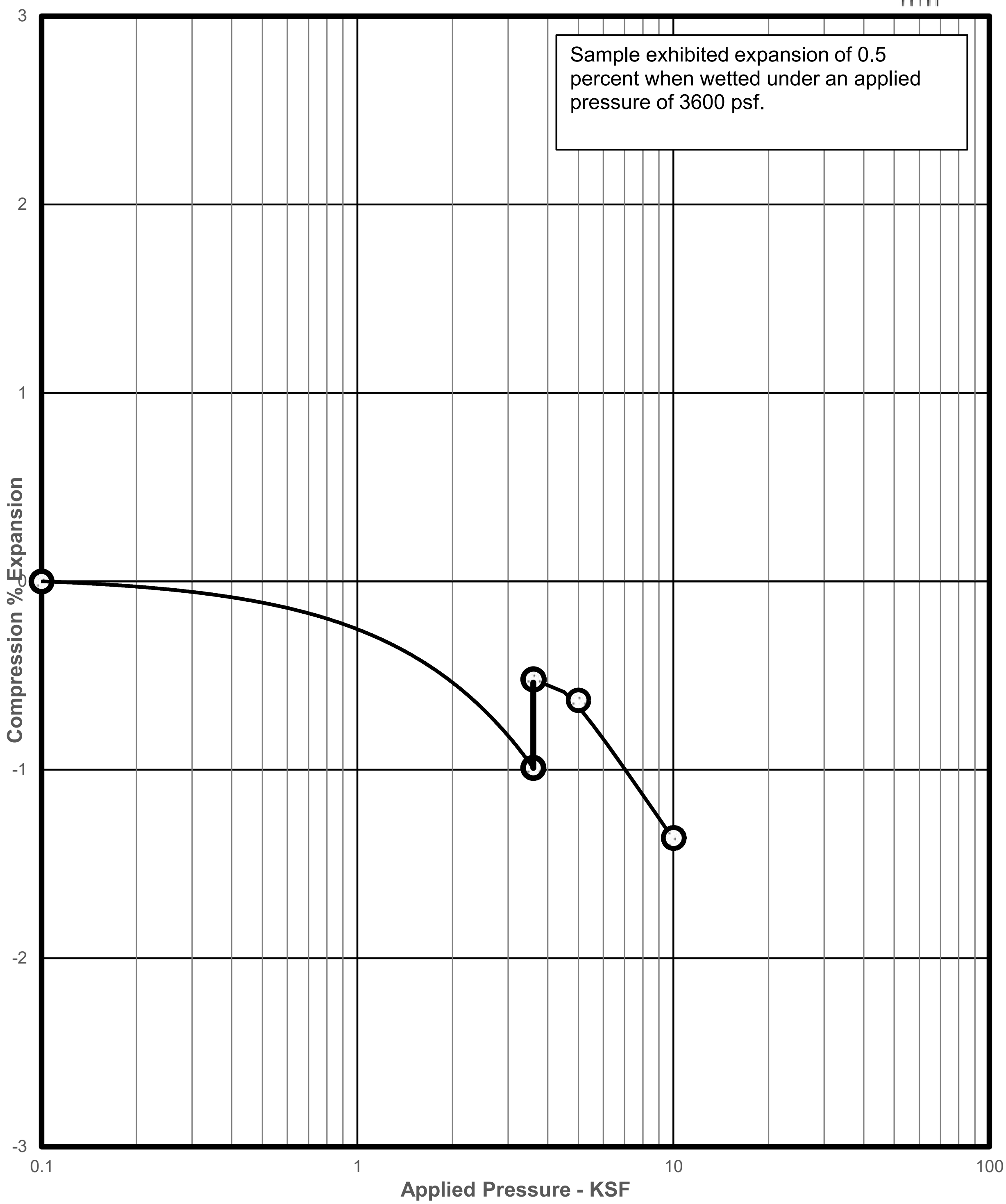


SAMPLE OF SANDSTONE
FROM: TH-3 AT 19 FEET

DRY UNIT WEIGHT: 111 pcf
MOISTURE CONTENT: 13.4 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 8

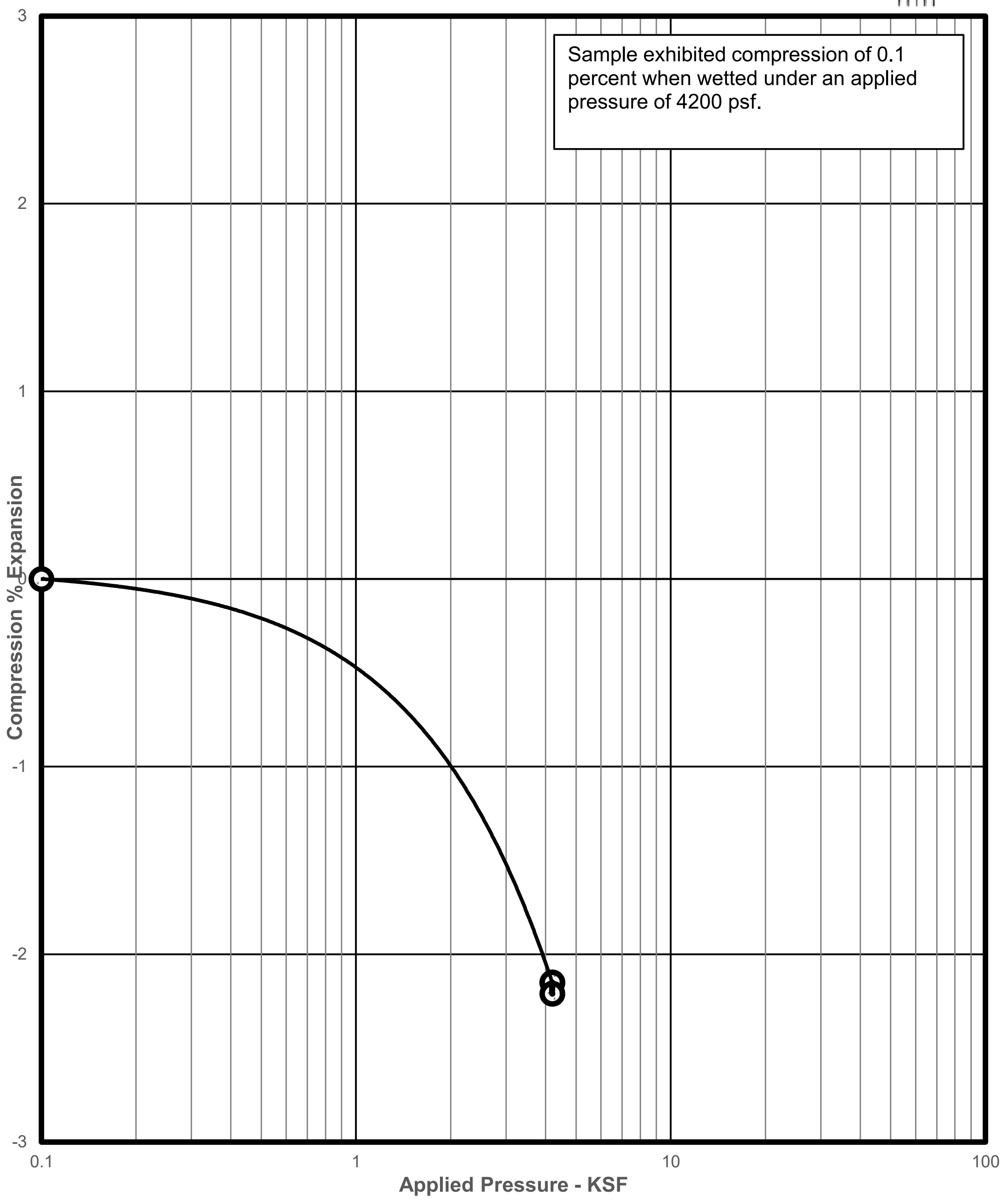


SAMPLE OF SANDSTONE
FROM: TH-3 AT 29 FEET

DRY UNIT WEIGHT: 112 pcf
MOISTURE CONTENT: 16.7 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 9

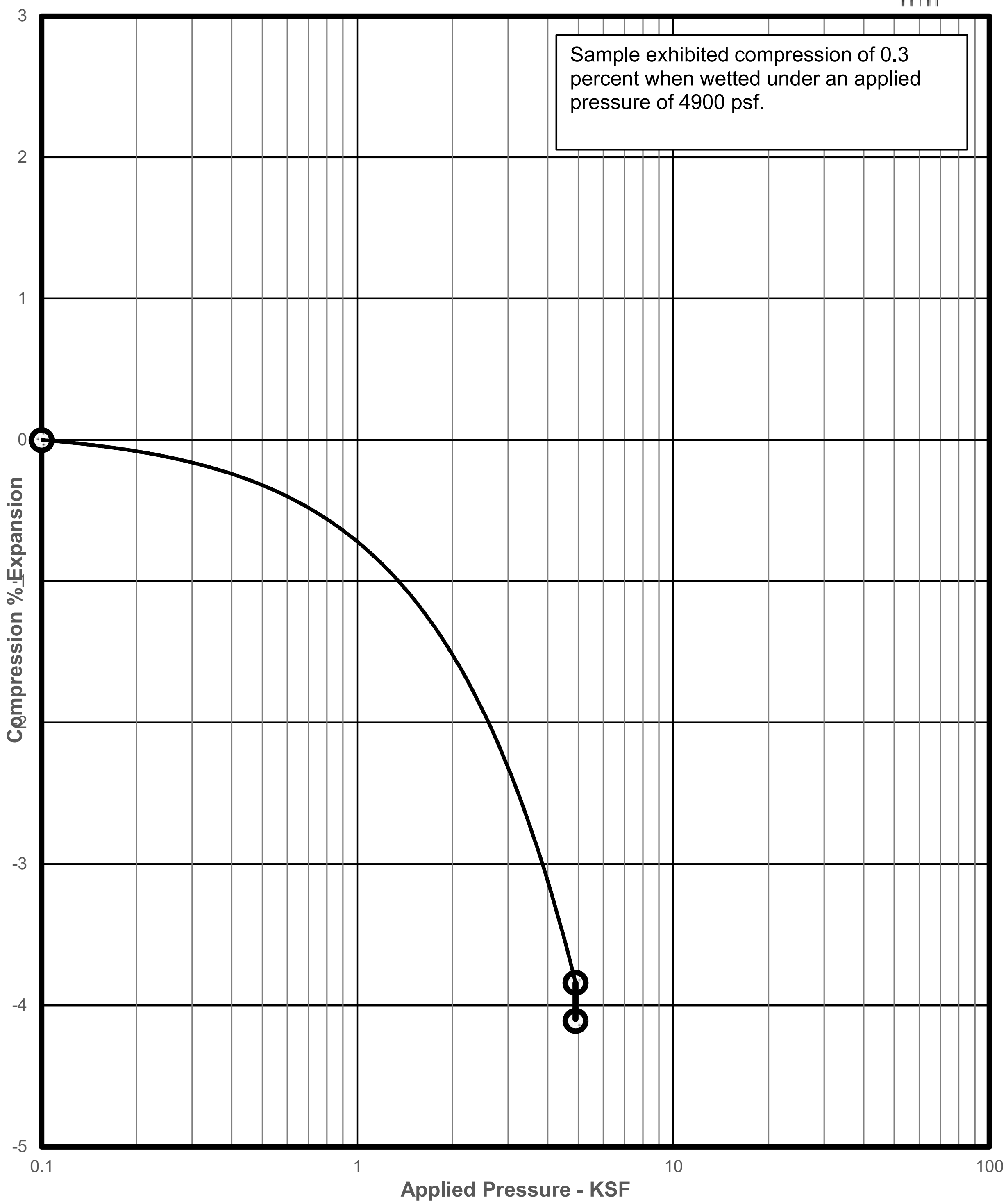


SAMPLE OF SANDSTONE
FROM: TH-3 AT 34 FEET

DRY UNIT WEIGHT: 105 pcf
MOISTURE CONTENT: 14.9 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 10



SAMPLE OF SANDSTONE

FROM: TH-3 AT 39 FEET

DRY UNIT WEIGHT: 102 pcf

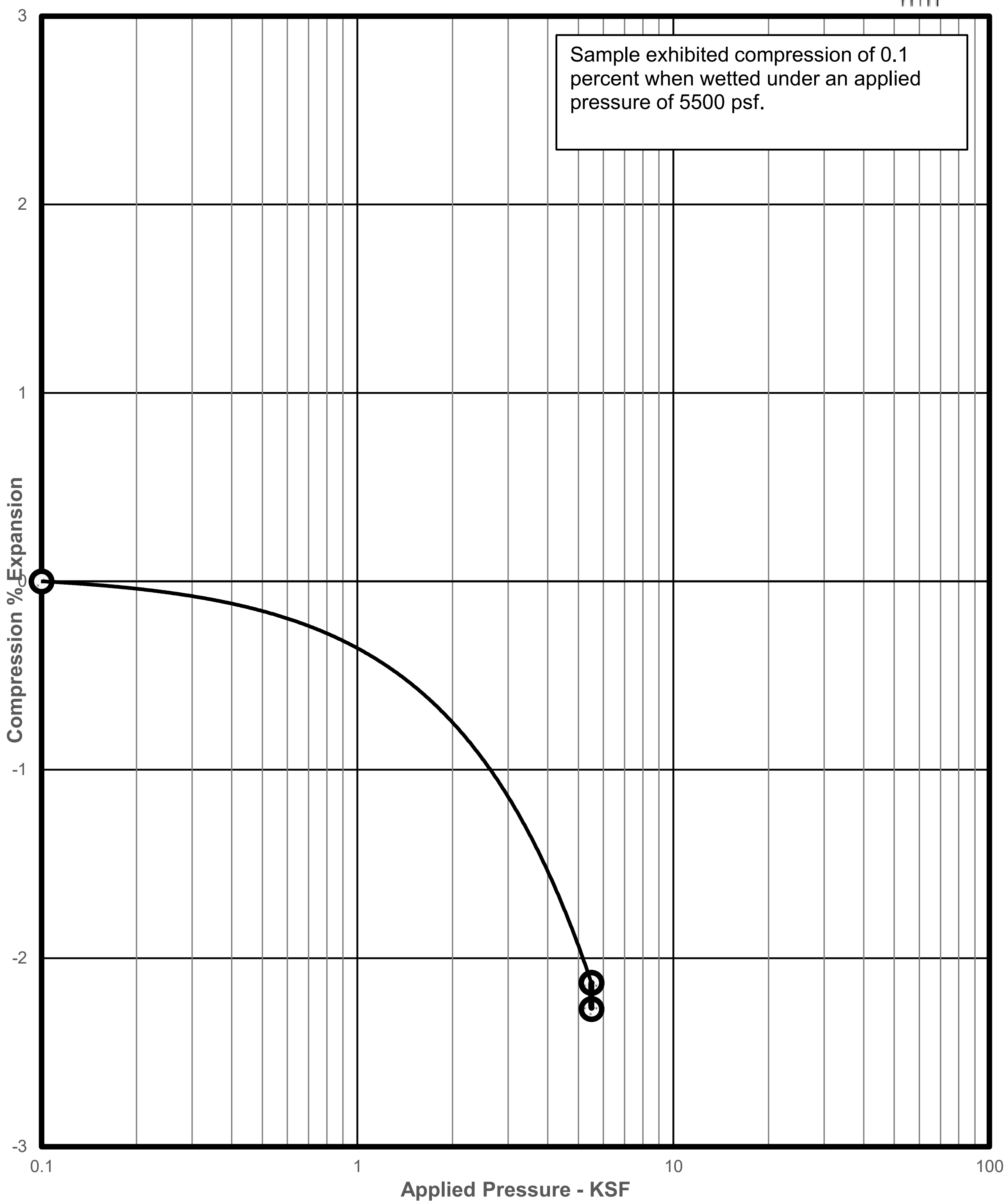
MOISTURE CONTENT: 16.2 %

SHEA PROPERTIES

HELIX INDUSTRIAL BUILDINGS C AND D

CTL|T PROJECT NO. DN51,883-125-R1

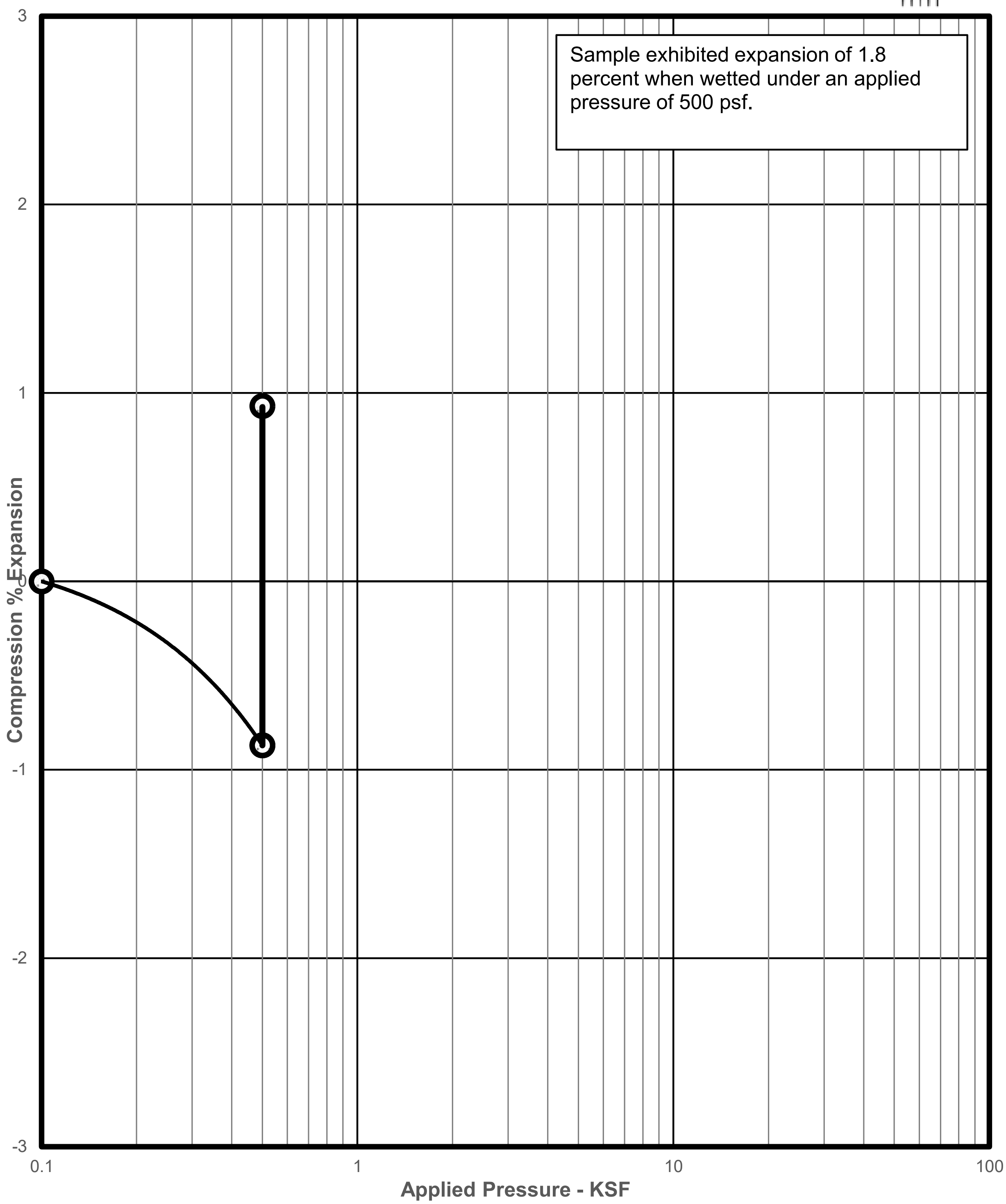
Swell Consolidation Test Results FIG. B- 11



SAMPLE OF	SANDSTONE	DRY UNIT WEIGHT:	106	pcf
FROM:	TH-3 AT 44 FEET	MOISTURE CONTENT:	15.2	%

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 12

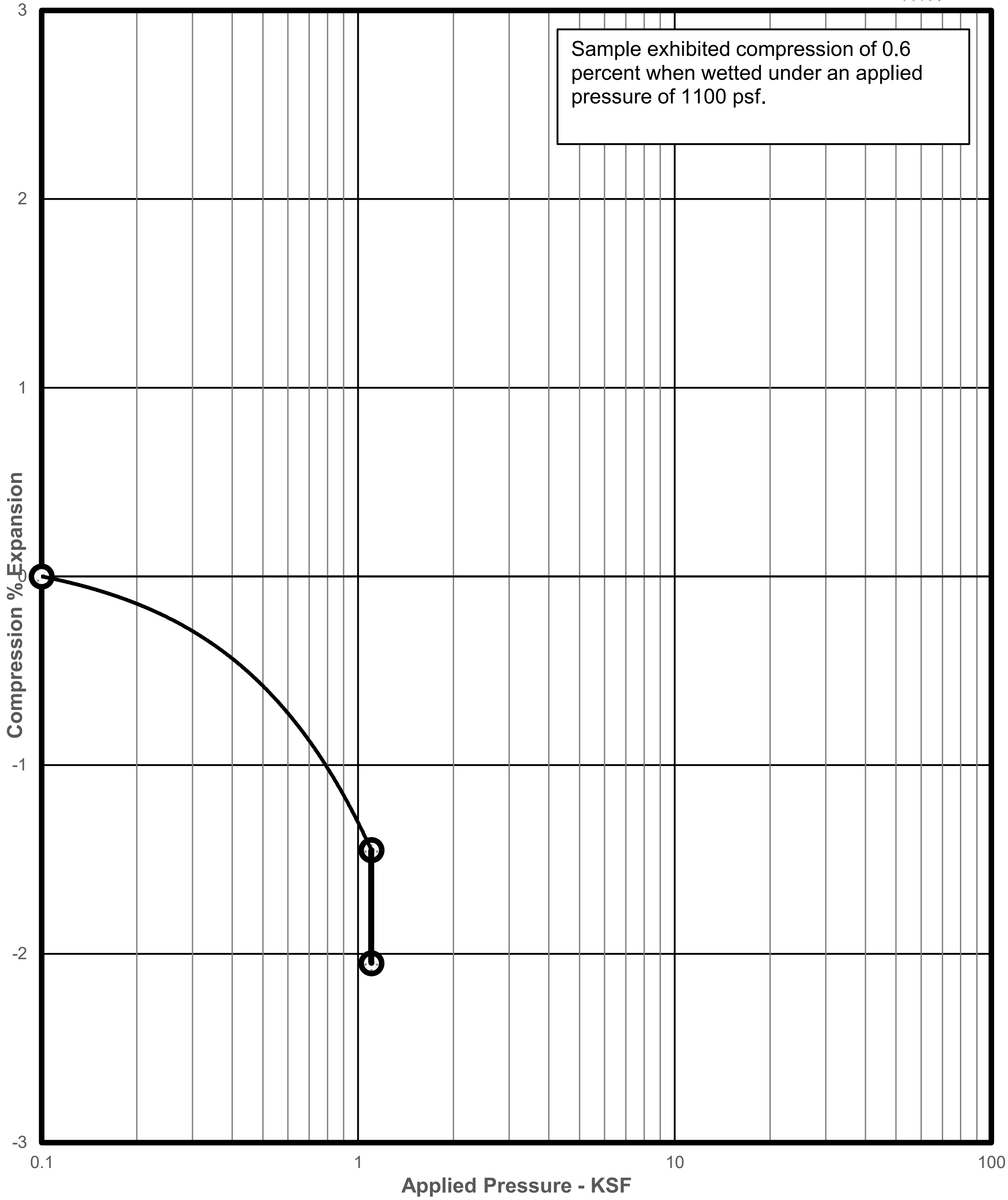


SAMPLE OF CLAYSTONE
FROM: TH-4 AT 4 FEET

DRY UNIT WEIGHT: 90 pcf
MOISTURE CONTENT: 25.9 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 13

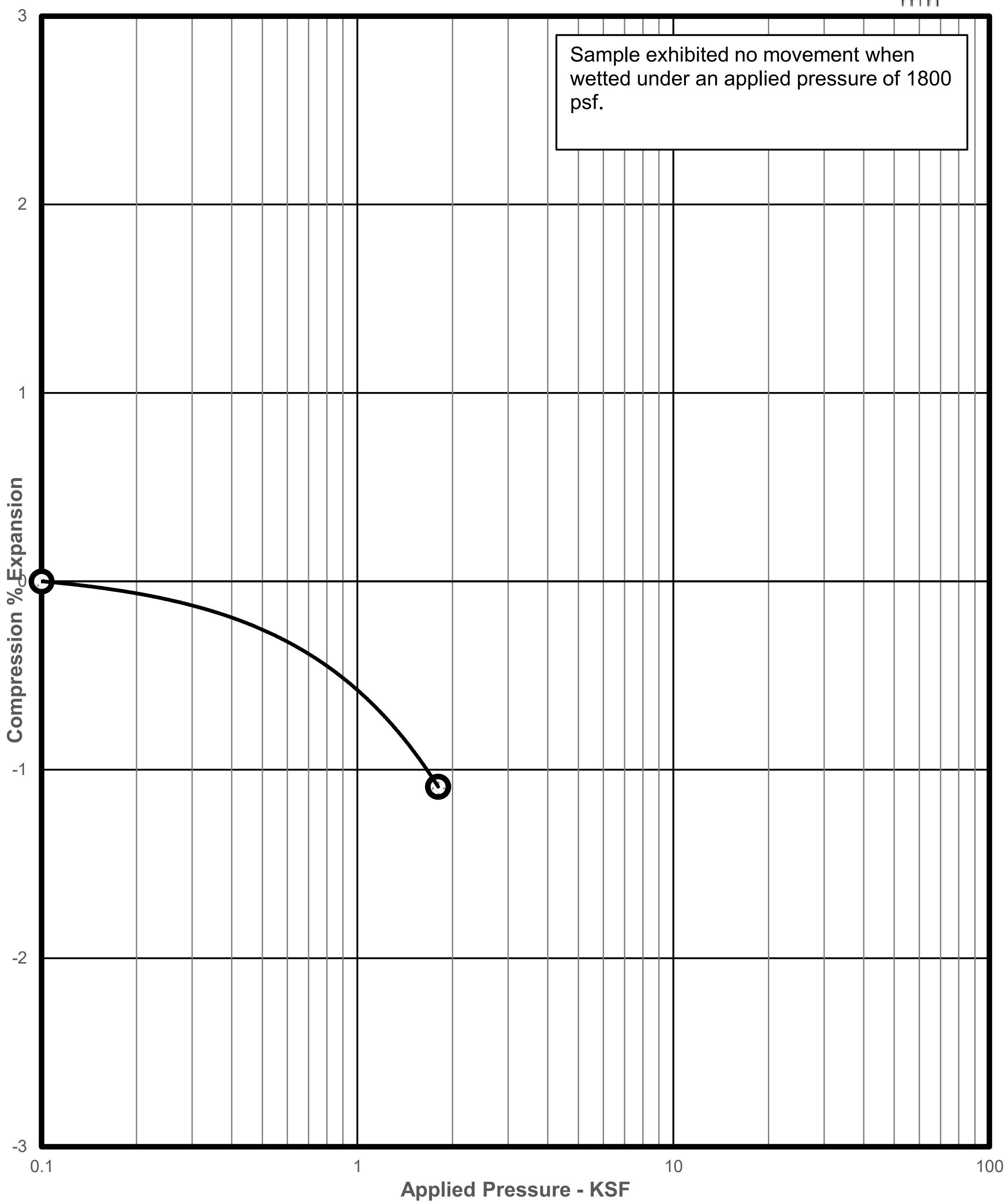


SAMPLE OF CLAYSTONE
FROM: TH-4 AT 9 FEET

DRY UNIT WEIGHT: 99 pcf
MOISTURE CONTENT: 14.3 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 14

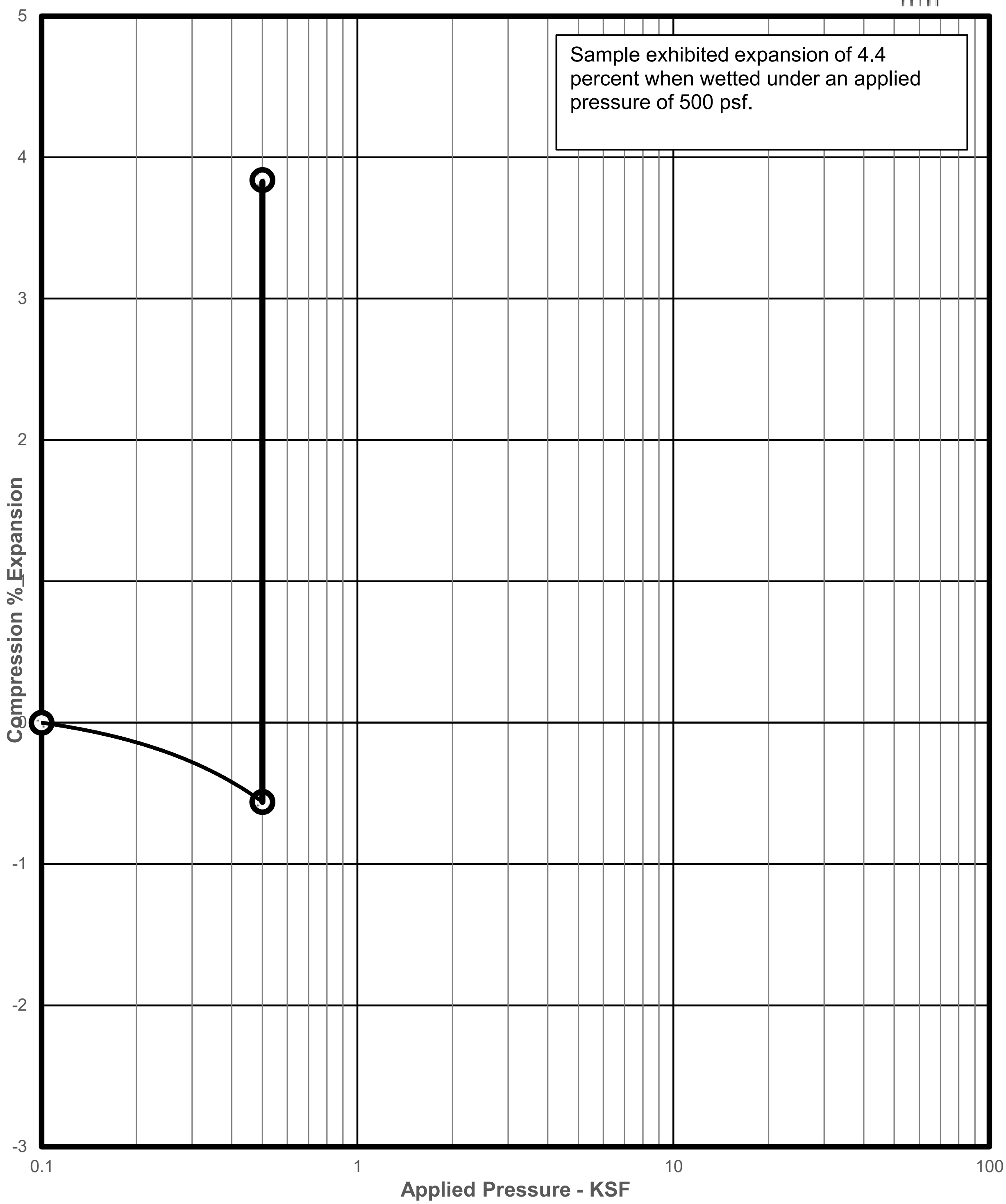


SAMPLE OF SANDSTONE
FROM: TH-4 AT 14 FEET

DRY UNIT WEIGHT: 104 pcf
MOISTURE CONTENT: 11.7 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 15



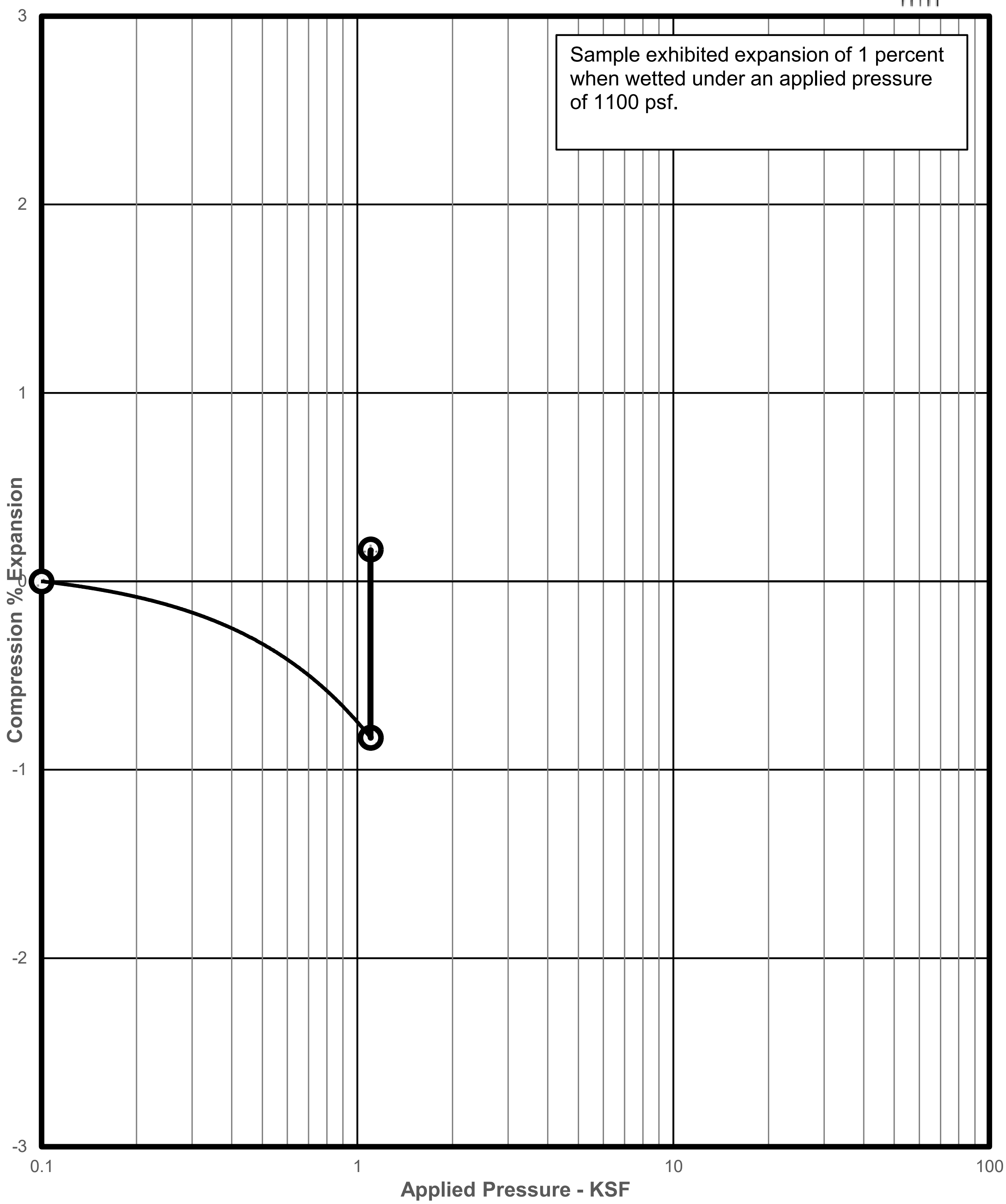
SAMPLE OF CLAYSTONE
FROM: TH-5 AT 4 FEET

DRY UNIT WEIGHT: 103 pcf
MOISTURE CONTENT: 18.6 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results

FIG. B- 16



SAMPLE OF CLAYSTONE

FROM: TH-5 AT 9 FEET

DRY UNIT WEIGHT: 110 pcf

MOISTURE CONTENT: 18.4 %

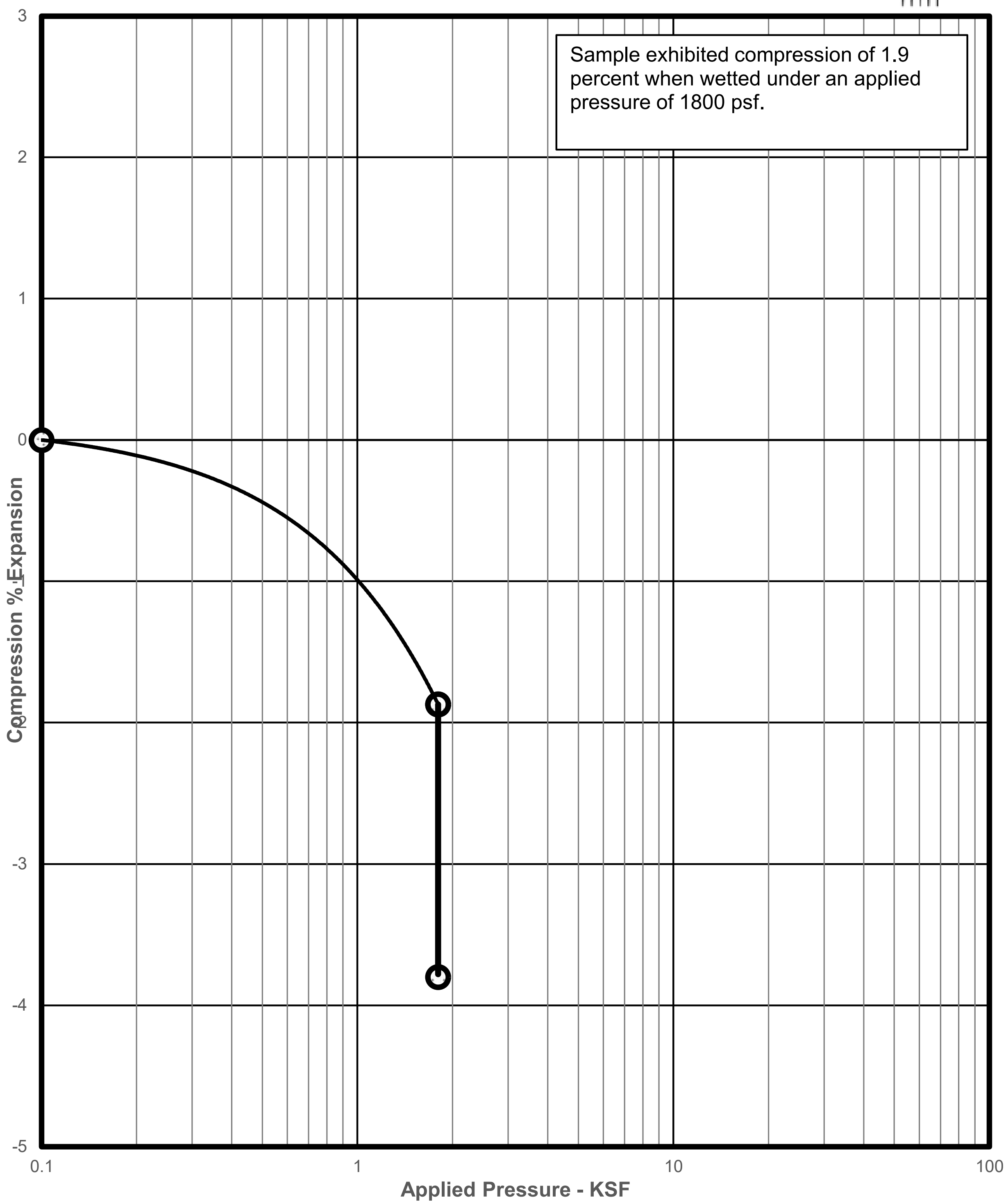
SHEA PROPERTIES

HELIX INDUSTRIAL BUILDINGS C AND D

CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results

FIG. B- 17

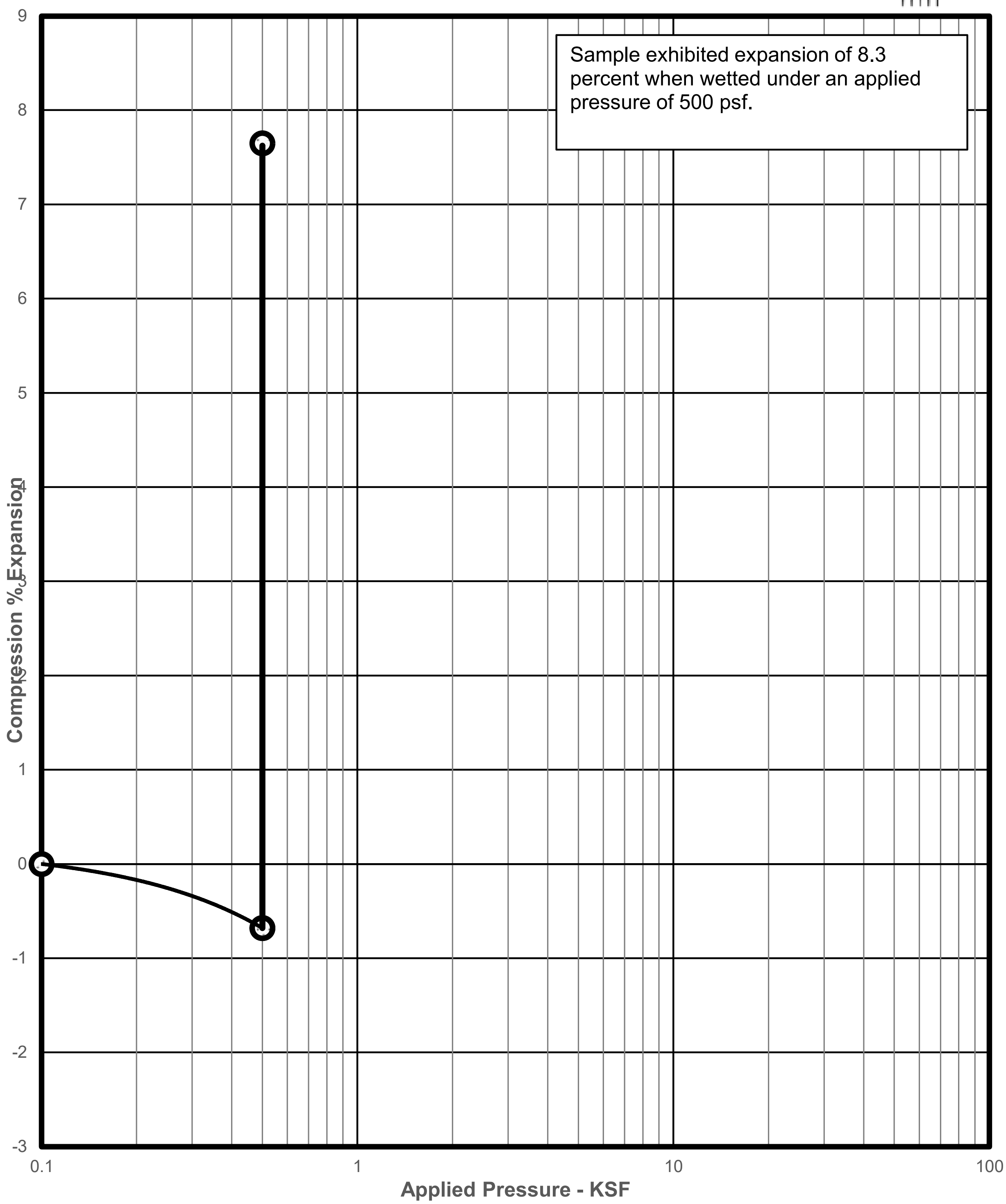


SAMPLE OF SANDSTONE
FROM: TH-5 AT 14 FEET

DRY UNIT WEIGHT: 107 pcf
MOISTURE CONTENT: 9.4 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 18



SAMPLE OF CLAYSTONE

FROM: TH-6 AT 4 FEET

DRY UNIT WEIGHT: 108 pcf

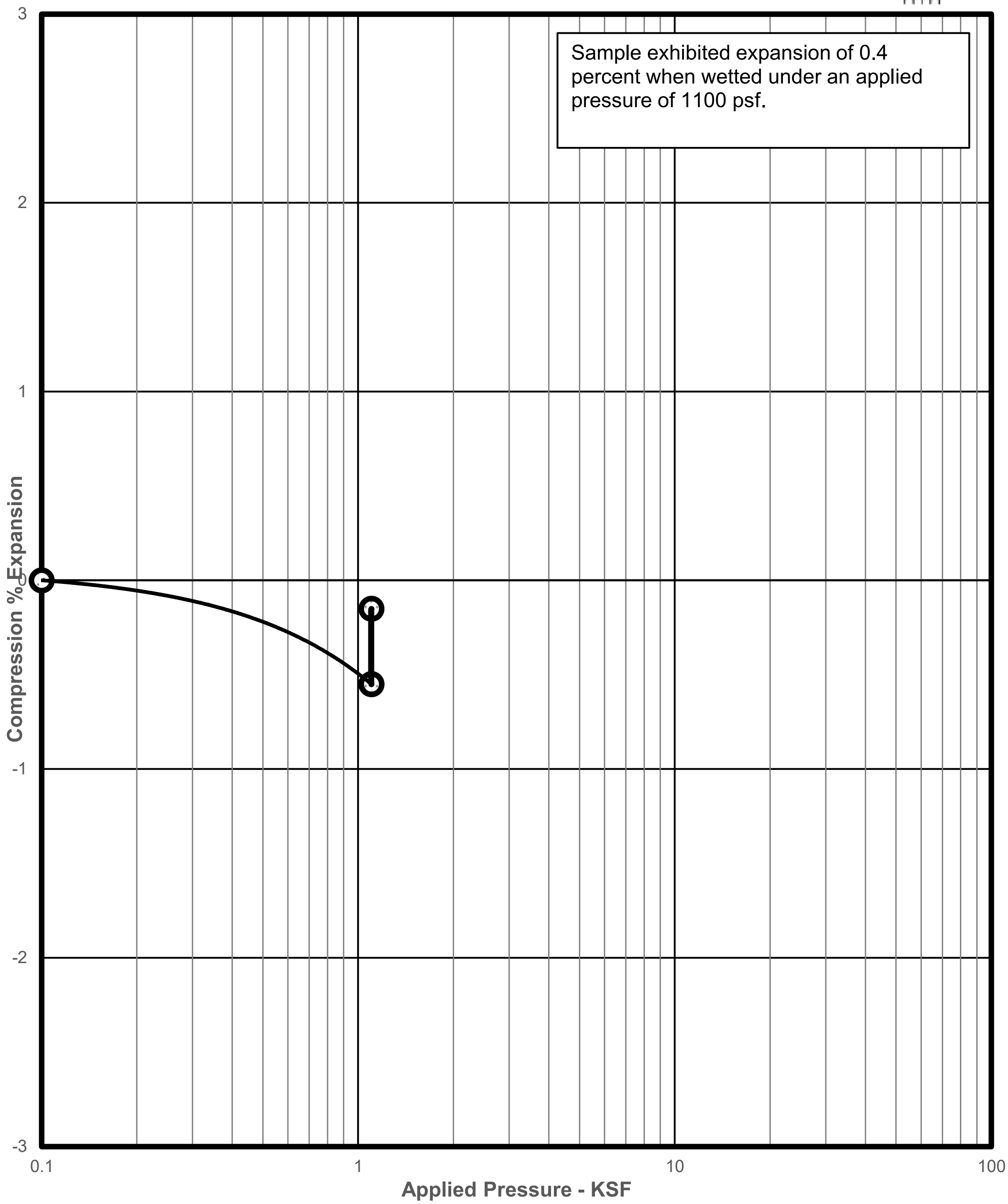
MOISTURE CONTENT: 13.1 %

SHEA PROPERTIES

HELIX INDUSTRIAL BUILDINGS C AND D

CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 19

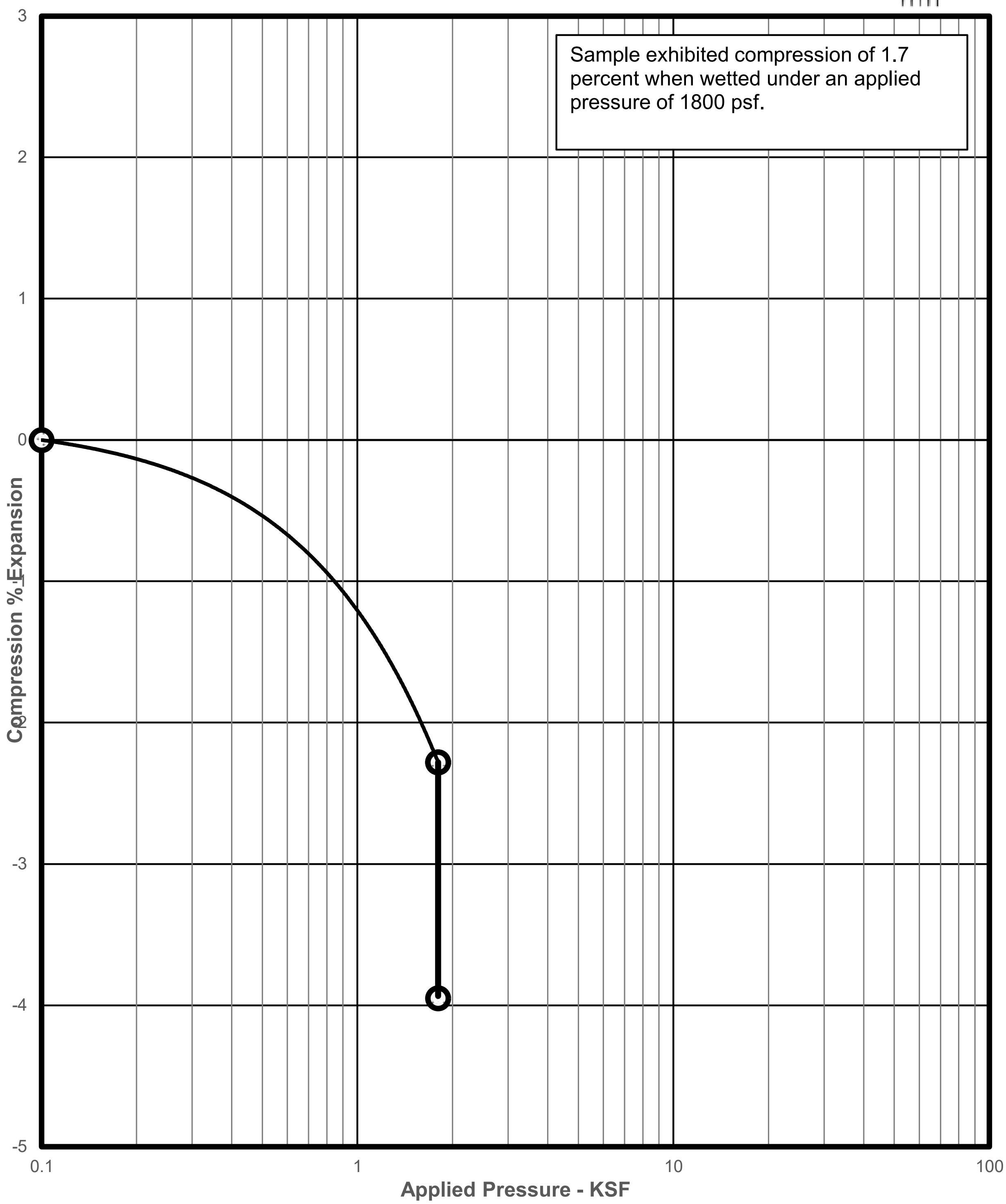


SAMPLE OF CLAYSTONE
FROM: TH-6 AT 9 FEET

DRY UNIT WEIGHT: 116 pcf
MOISTURE CONTENT: 13.7 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 20



SAMPLE OF SANDSTONE

FROM: TH-6 AT 14 FEET

DRY UNIT WEIGHT: 103 pcf

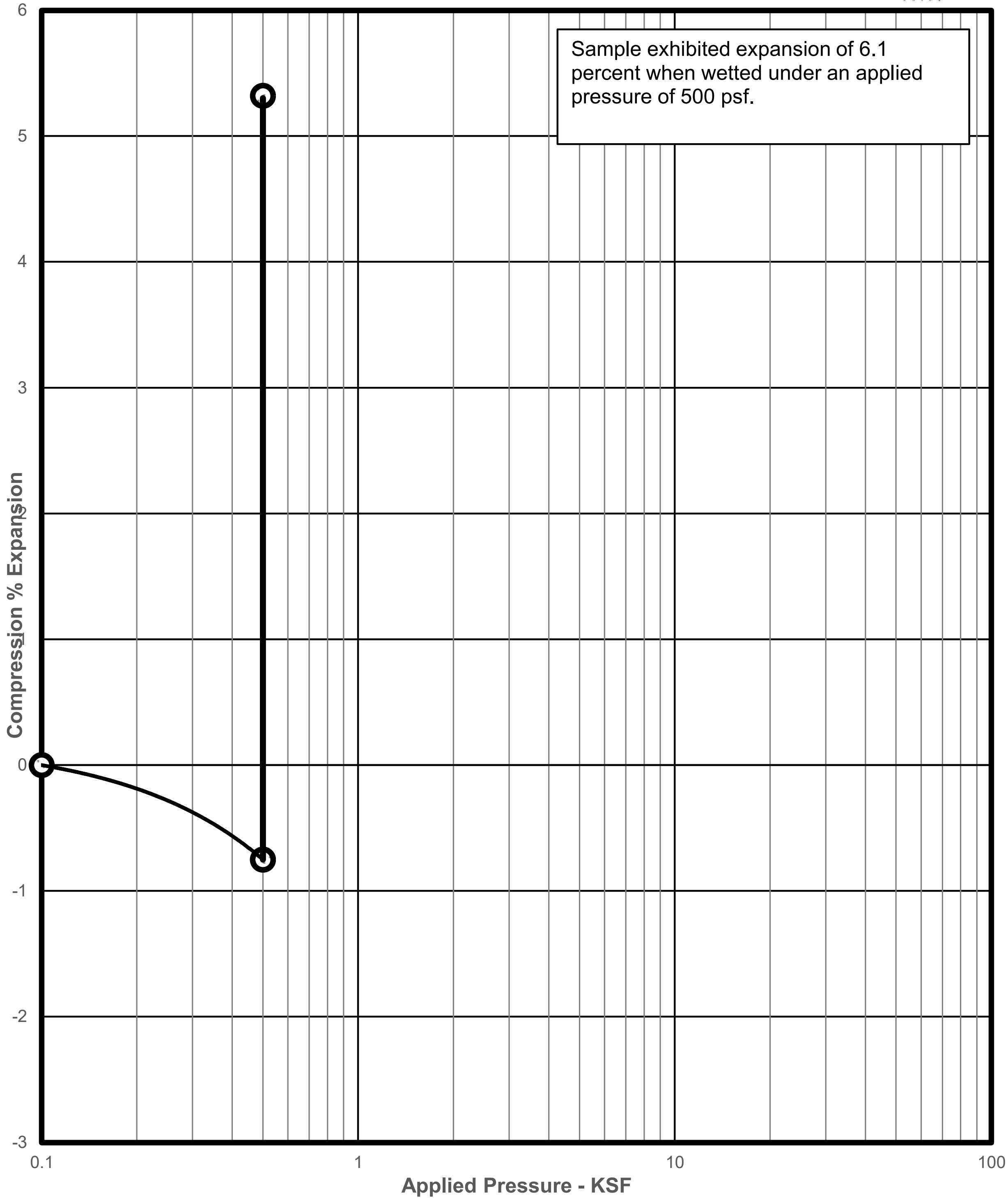
MOISTURE CONTENT: 10.4 %

SHEA PROPERTIES

HELIX INDUSTRIAL BUILDINGS C AND D

CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 21

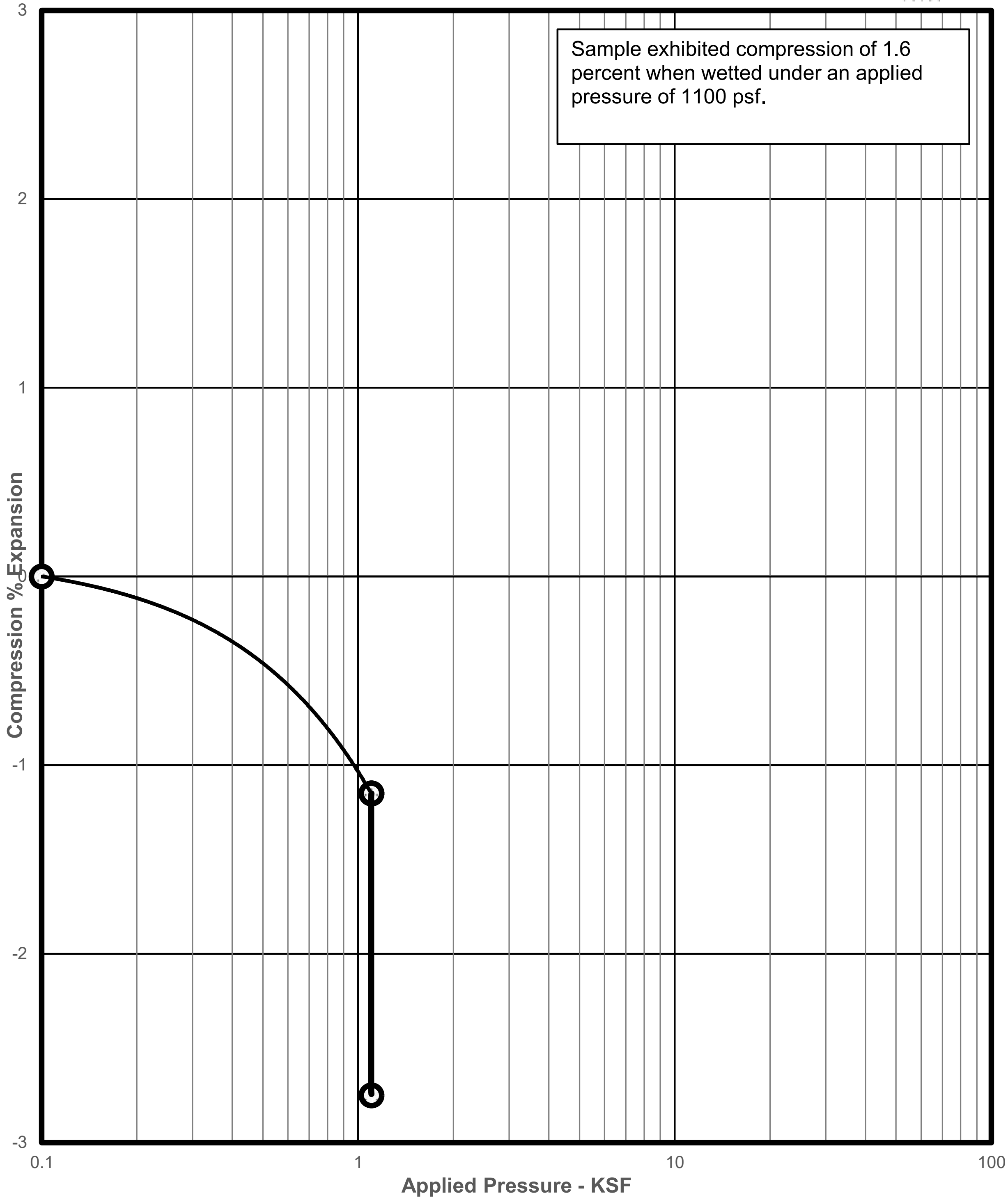


SAMPLE OF: CLAY, SANDY (CL)
FROM: TH-7 AT 4 FEET

DRY UNIT WEIGHT: 96 pcf
MOISTURE CONTENT: 12.3 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 22

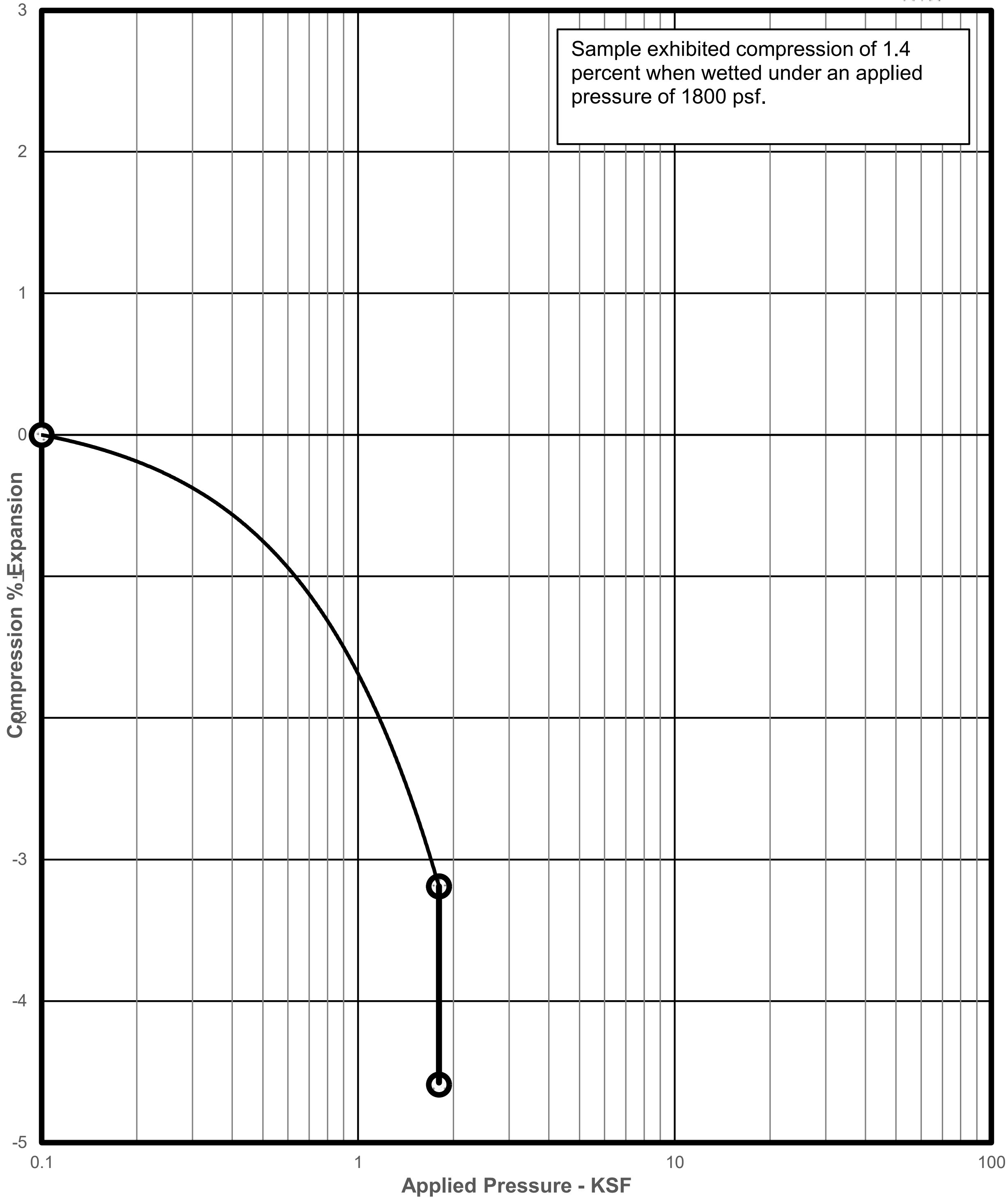


SAMPLE OF SANDSTONE
FROM: TH-7 AT 9 FEET

DRY UNIT WEIGHT: 94 pcf
MOISTURE CONTENT: 17.3 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 23



SAMPLE OF SANDSTONE

FROM: TH-7 AT 14 FEET

DRY UNIT WEIGHT: 95 pcf

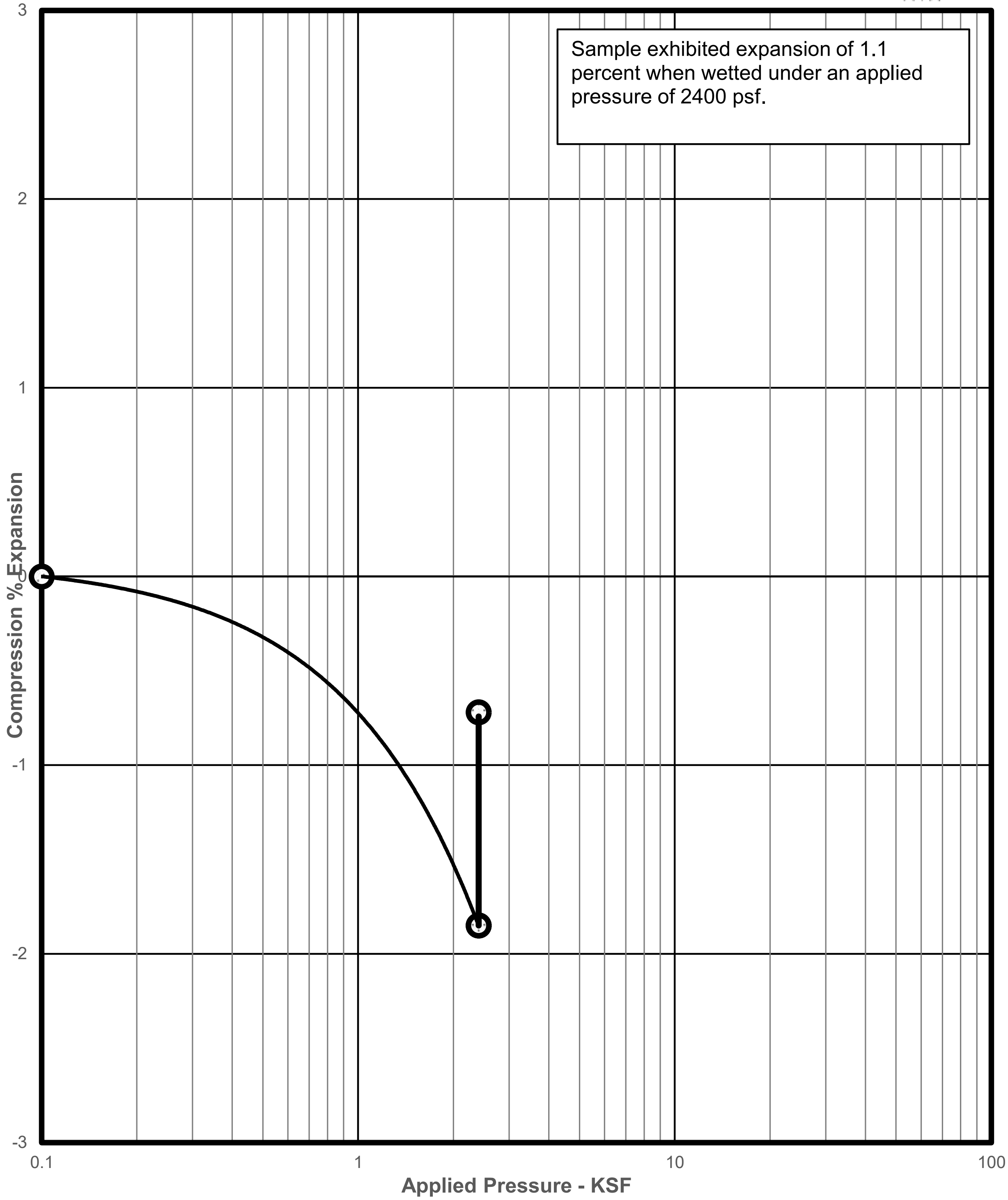
MOISTURE CONTENT: 13.7 %

SHEA PROPERTIES

HELIX INDUSTRIAL BUILDINGS C AND D

CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 24



SAMPLE OF CLAYSTONE

FROM: TH-7 AT 19 FEET

DRY UNIT WEIGHT: 108 pcf

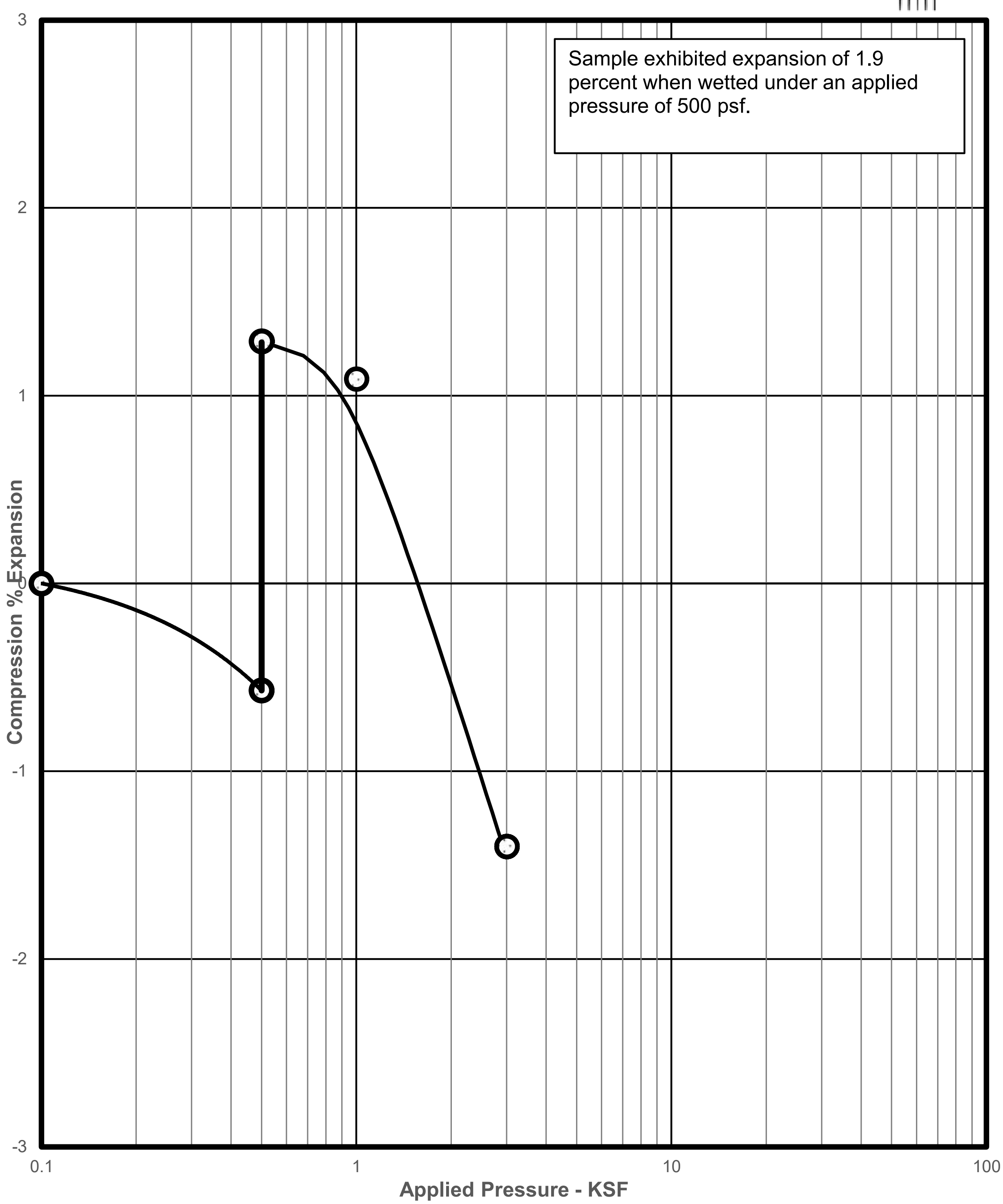
MOISTURE CONTENT: 18.8 %

SHEA PROPERTIES

HELIX INDUSTRIAL BUILDINGS C AND D

CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 25

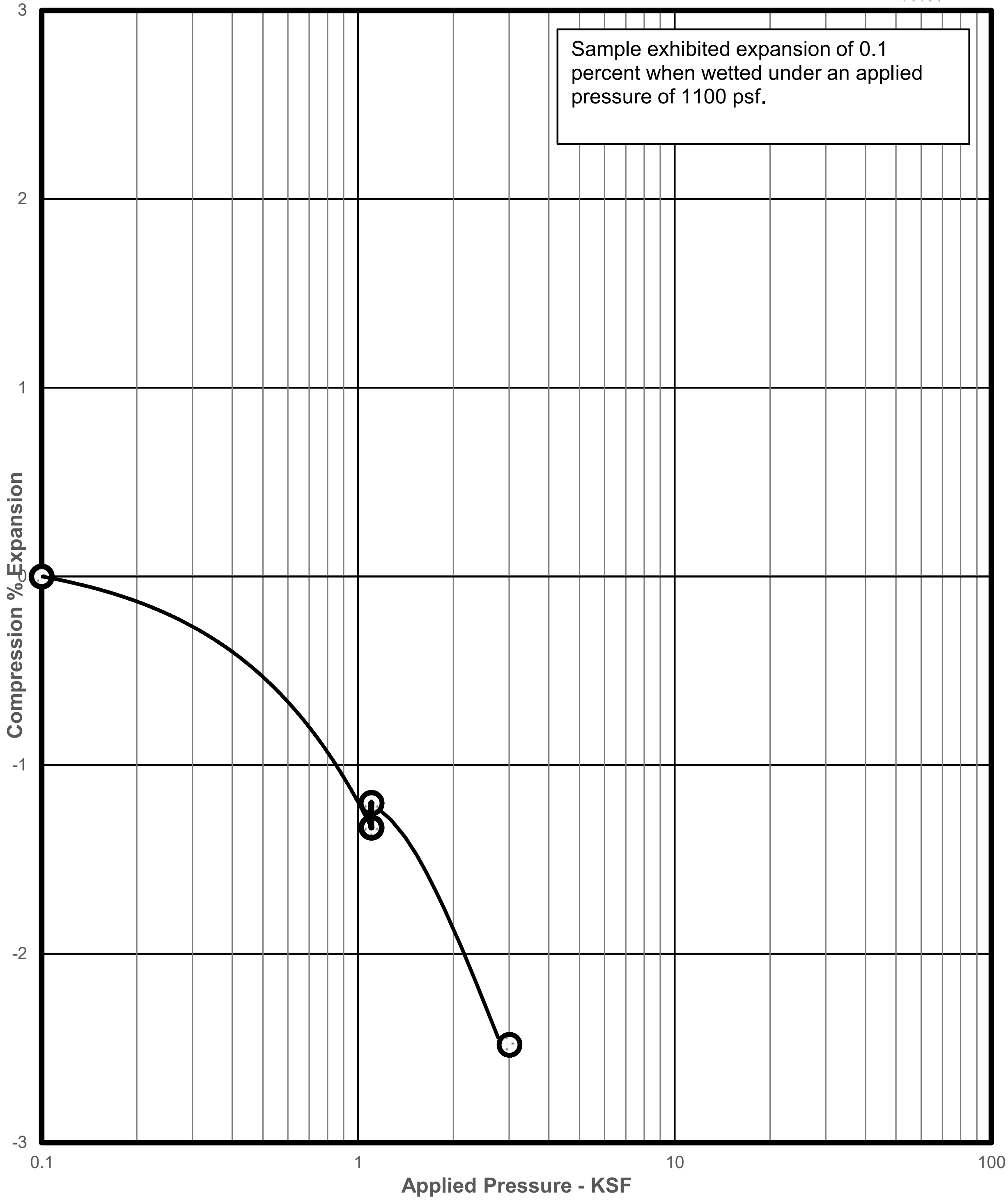


SAMPLE OF: CLAY, SANDY (CL)
FROM: TH-8 AT 4 FEET

DRY UNIT WEIGHT: 106 pcf
MOISTURE CONTENT: 13.3 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 26



SAMPLE OF SANDSTONE

FROM: TH-8 AT 9 FEET

DRY UNIT WEIGHT: 105 pcf

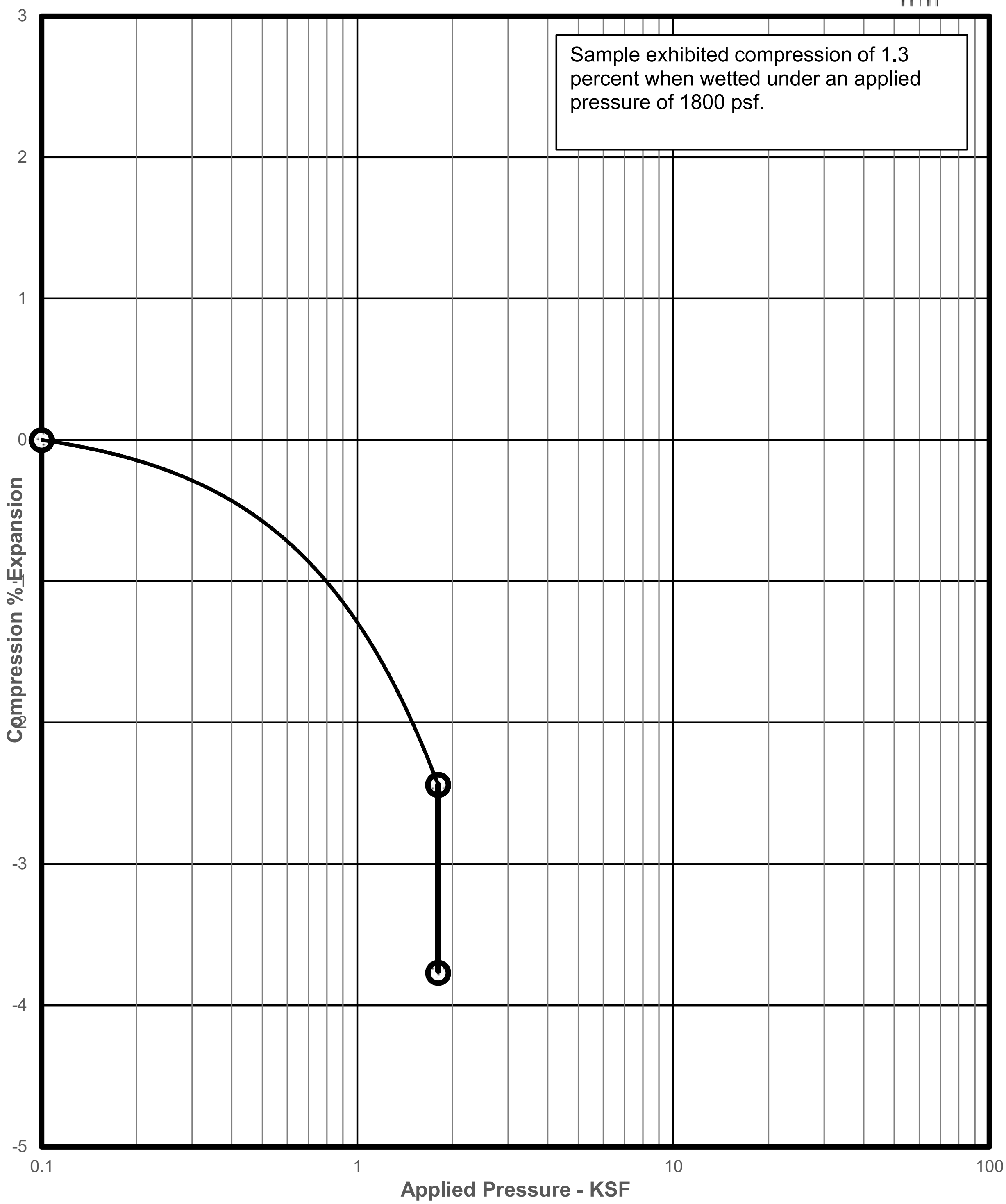
MOISTURE CONTENT: 15.5 %

SHEA PROPERTIES

HELIX INDUSTRIAL BUILDINGS C AND D

CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 27

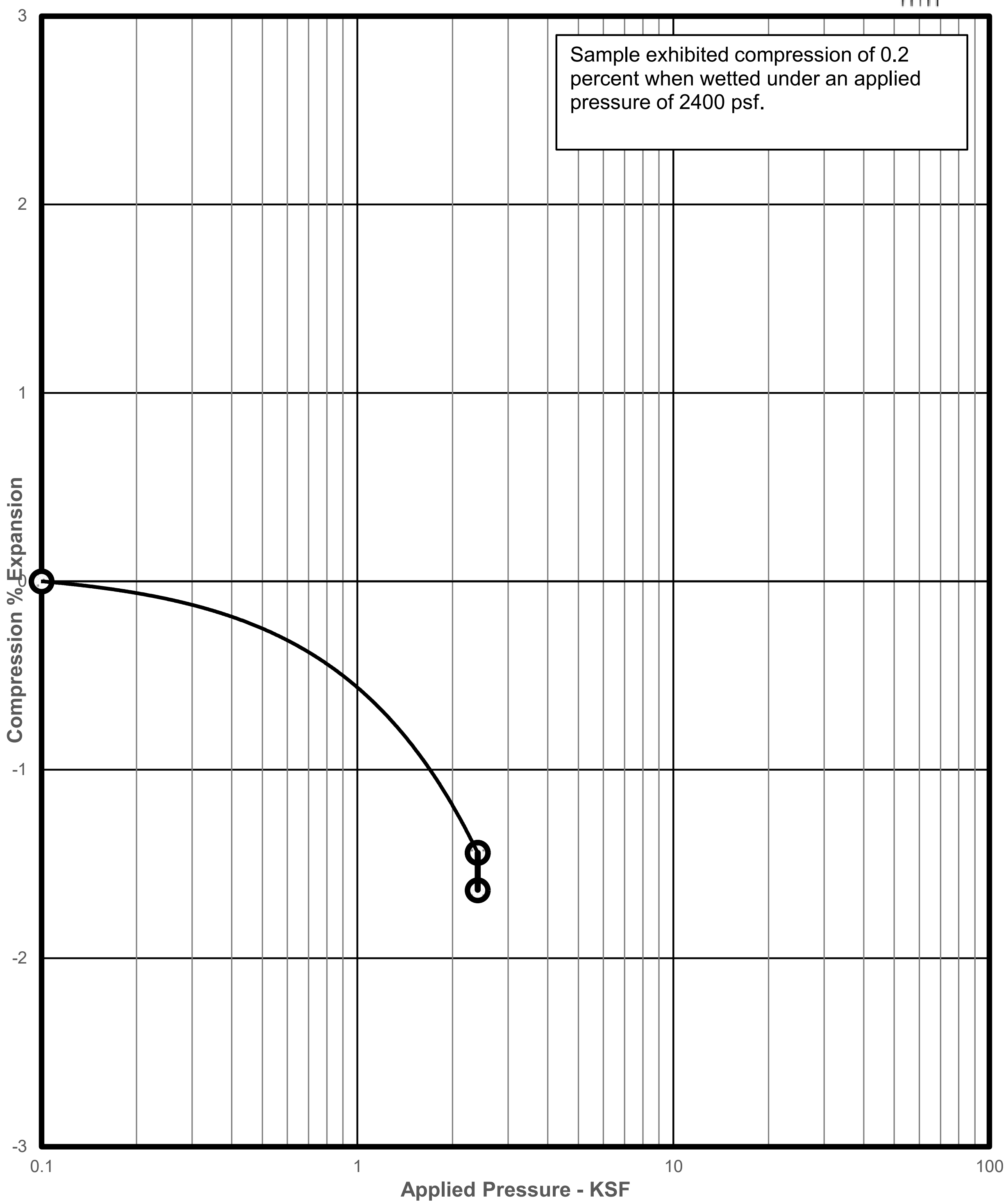


SAMPLE OF SANDSTONE
FROM: TH-8 AT 14 FEET

DRY UNIT WEIGHT: 104 pcf
MOISTURE CONTENT: 7.6 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 28

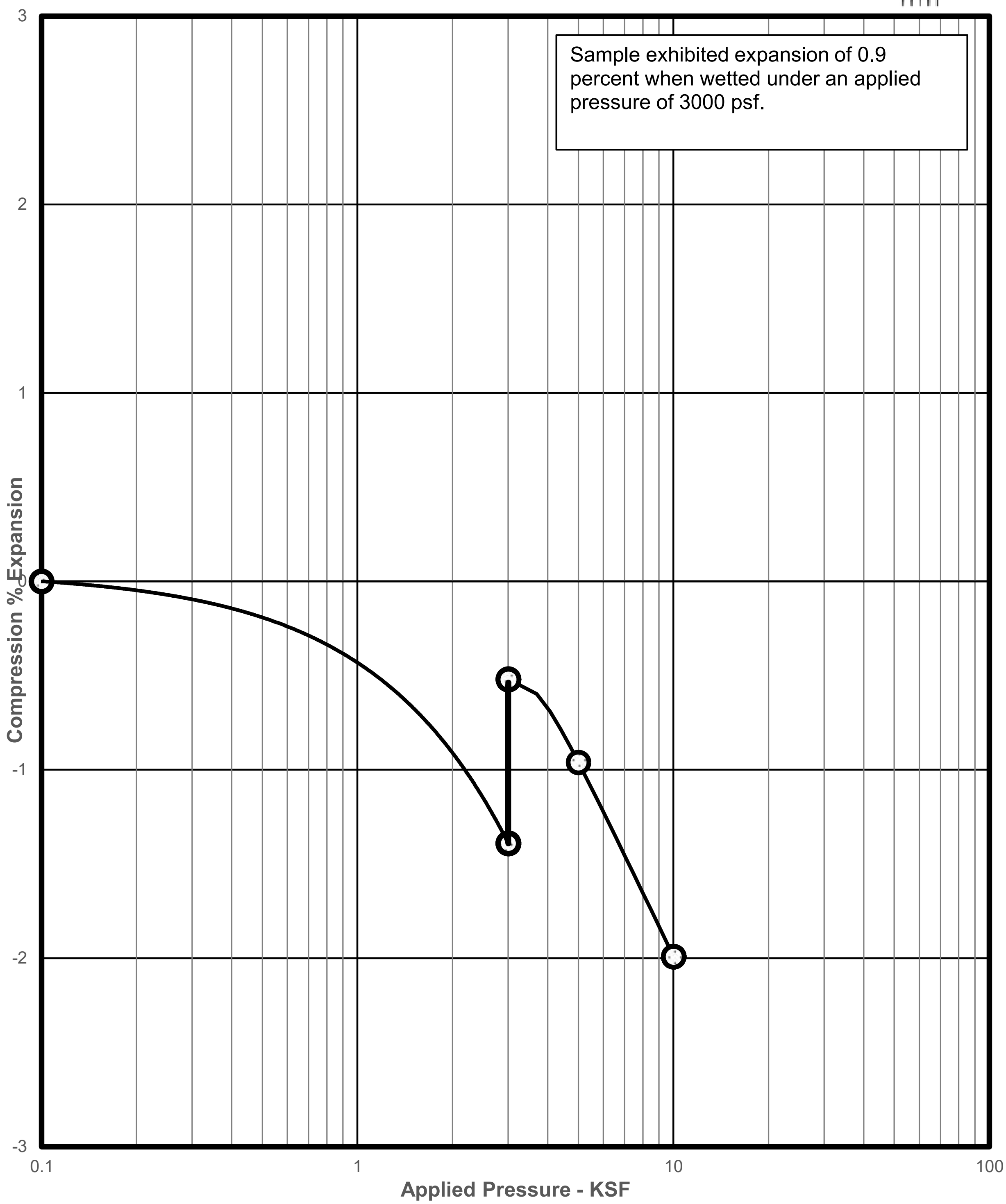


SAMPLE OF SANDSTONE
FROM: TH-8 AT 19 FEET

DRY UNIT WEIGHT: 100 pcf
MOISTURE CONTENT: 9.1 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 29



SAMPLE OF CLAYSTONE

FROM: TH-8 AT 24 FEET

DRY UNIT WEIGHT: 100 pcf

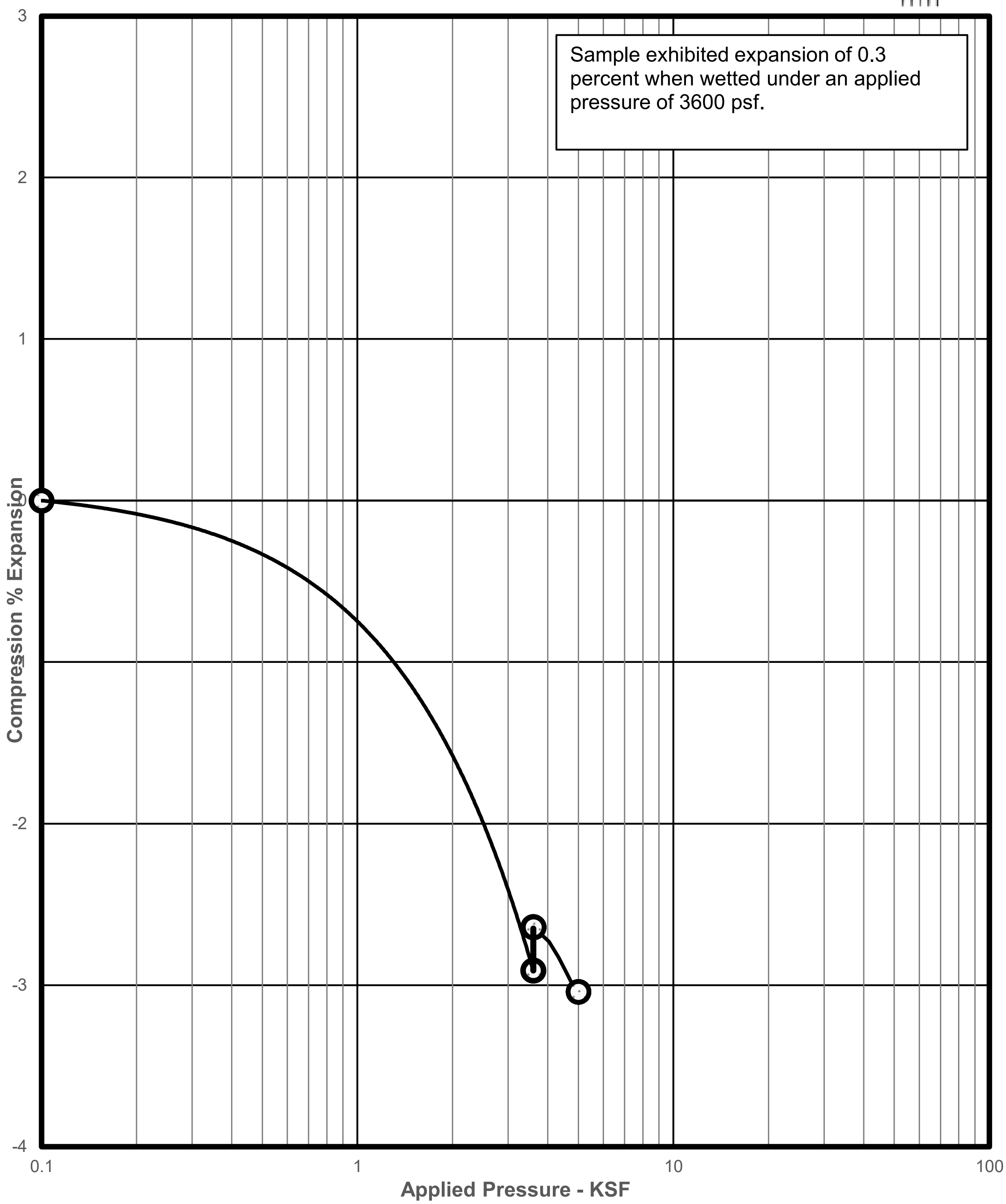
MOISTURE CONTENT: 22.5 %

SHEA PROPERTIES

HELIX INDUSTRIAL BUILDINGS C AND D

CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 30



SAMPLE OF CLAYSTONE

FROM: TH-8 AT 29 FEET

DRY UNIT WEIGHT: 102 pcf

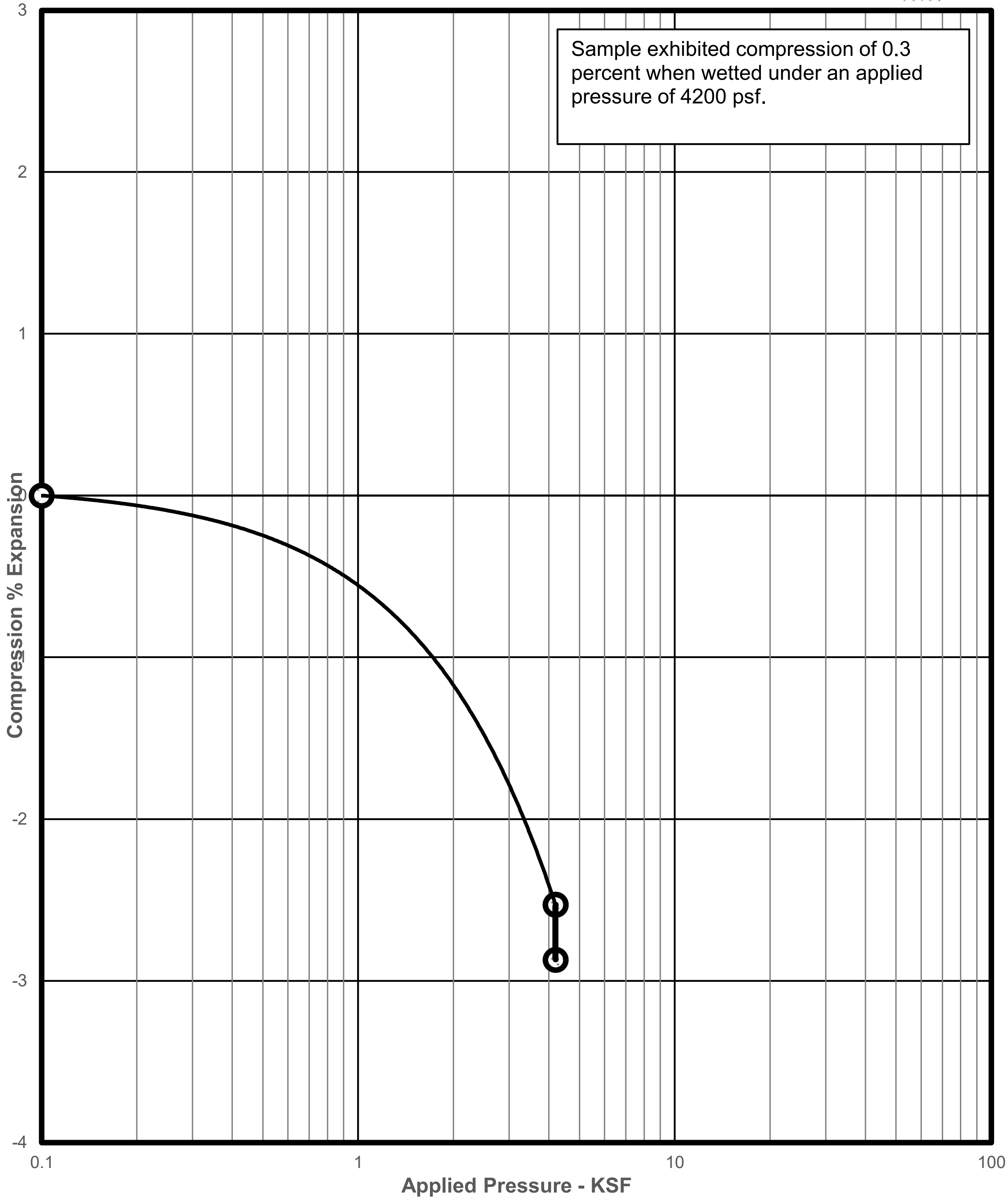
MOISTURE CONTENT: 20.8 %

SHEA PROPERTIES

HELIX INDUSTRIAL BUILDINGS C AND D

CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 31



SAMPLE OF CLAYSTONE

FROM: TH-8 AT 34 FEET

DRY UNIT WEIGHT: 102 pcf

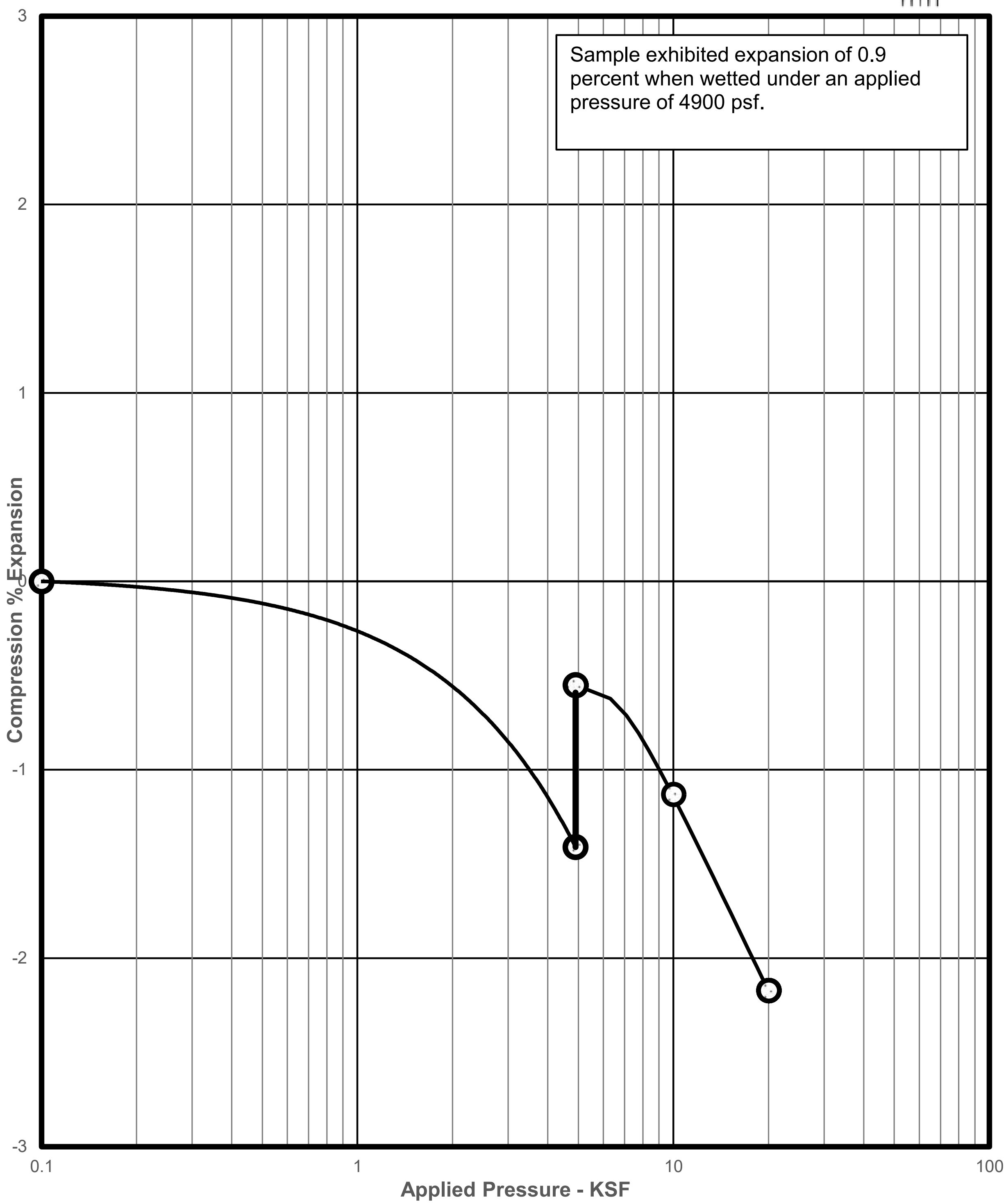
MOISTURE CONTENT: 19.3 %

SHEA PROPERTIES

HELIX INDUSTRIAL BUILDINGS C AND D

CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 32

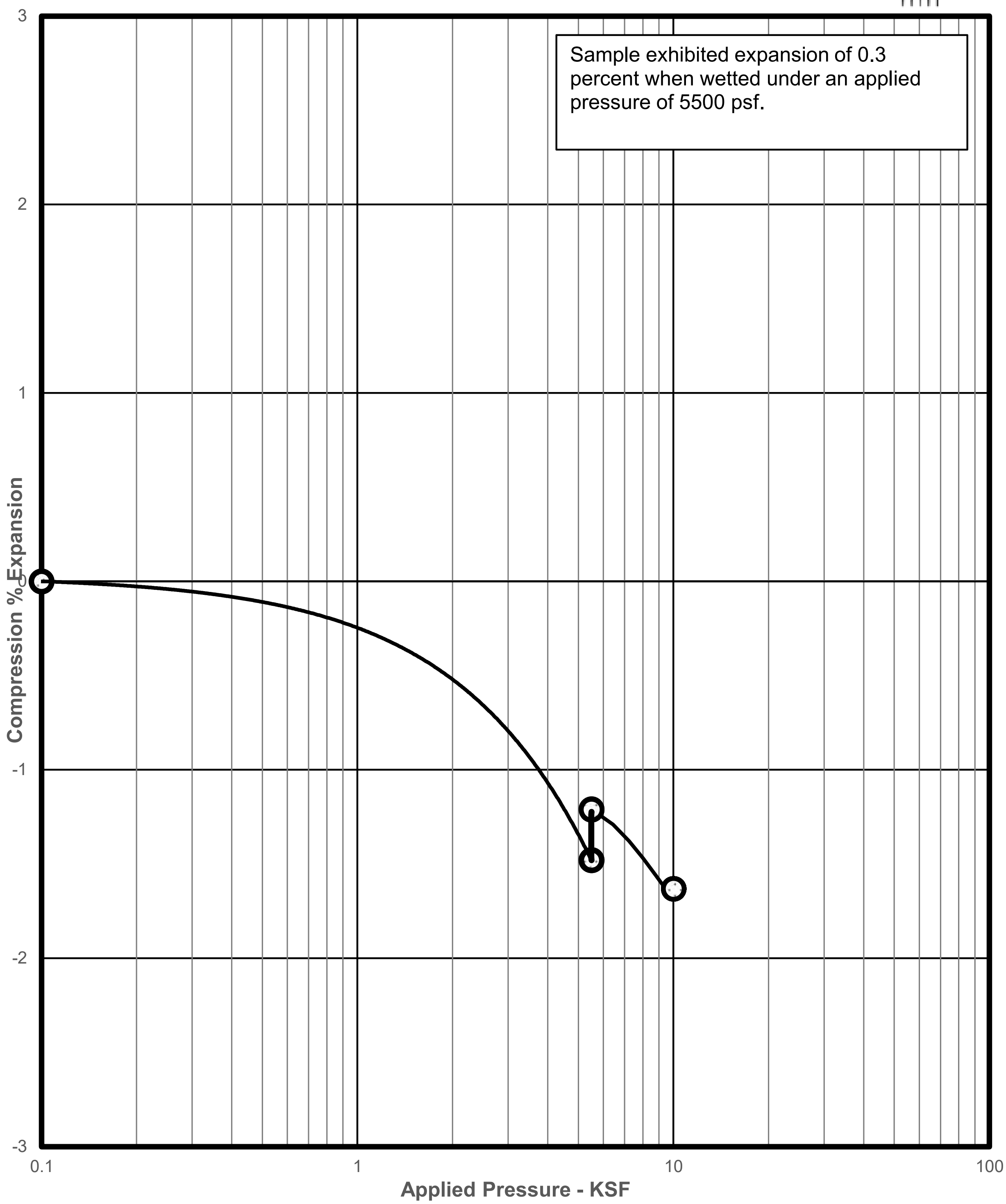


SAMPLE OF CLAYSTONE
FROM: TH-8 AT 39 FEET

DRY UNIT WEIGHT: 109 pcf
MOISTURE CONTENT: 18.8 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

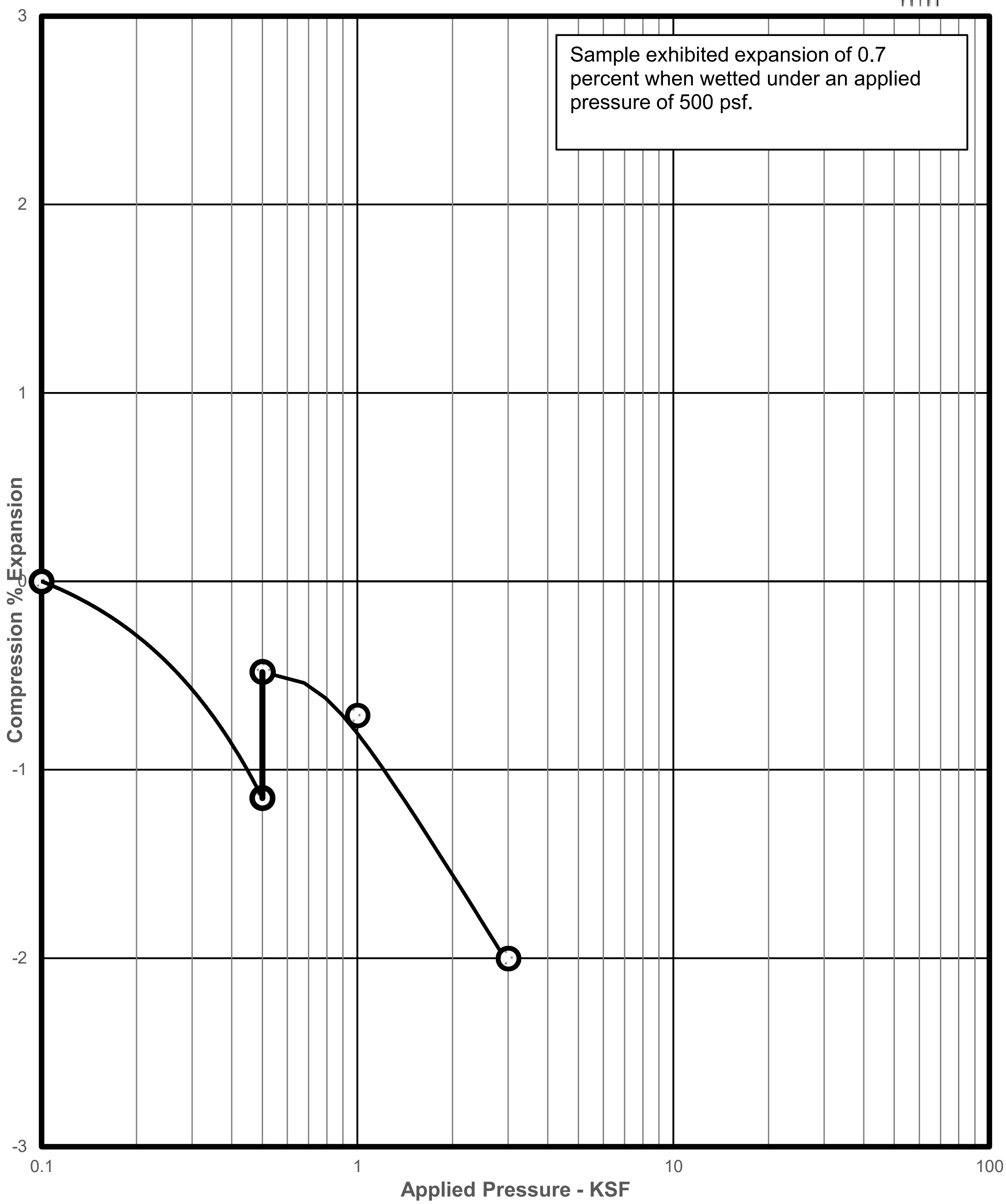
Swell Consolidation Test Results FIG. B- 33



SAMPLE OF	CLAYSTONE	DRY UNIT WEIGHT:	116	pcf
FROM:	TH-8 AT 44 FEET	MOISTURE CONTENT:	16.8	%

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 34

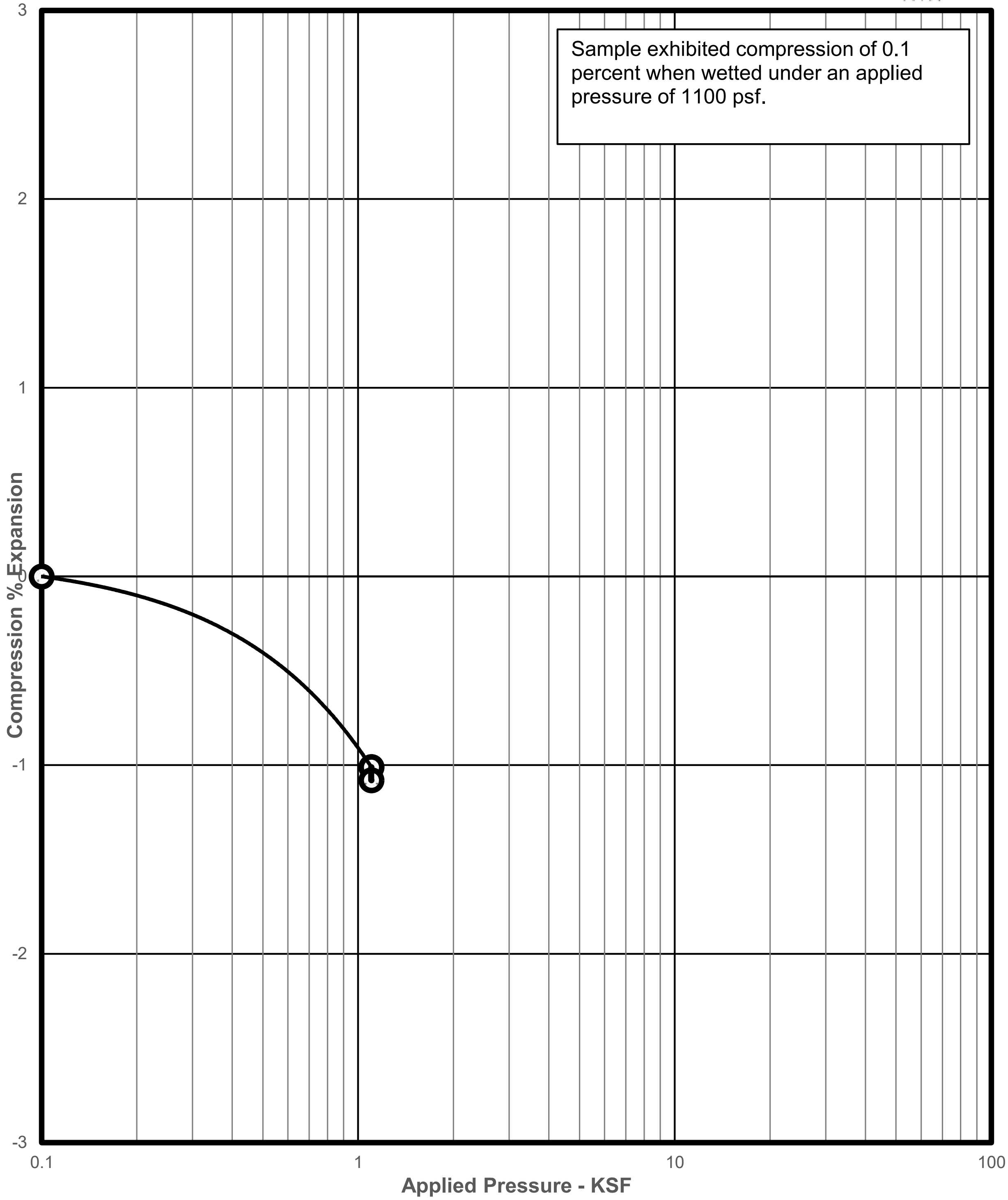


SAMPLE OF: CLAY, SANDY (CL)
FROM: TH-9 AT 4 FEET

DRY UNIT WEIGHT: 101 pcf
MOISTURE CONTENT: 17.5 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 35



SAMPLE OF: CLAY, SANDY (CL)

FROM: TH-9 AT 9 FEET

DRY UNIT WEIGHT: 103 pcf

MOISTURE CONTENT: 12.3 %

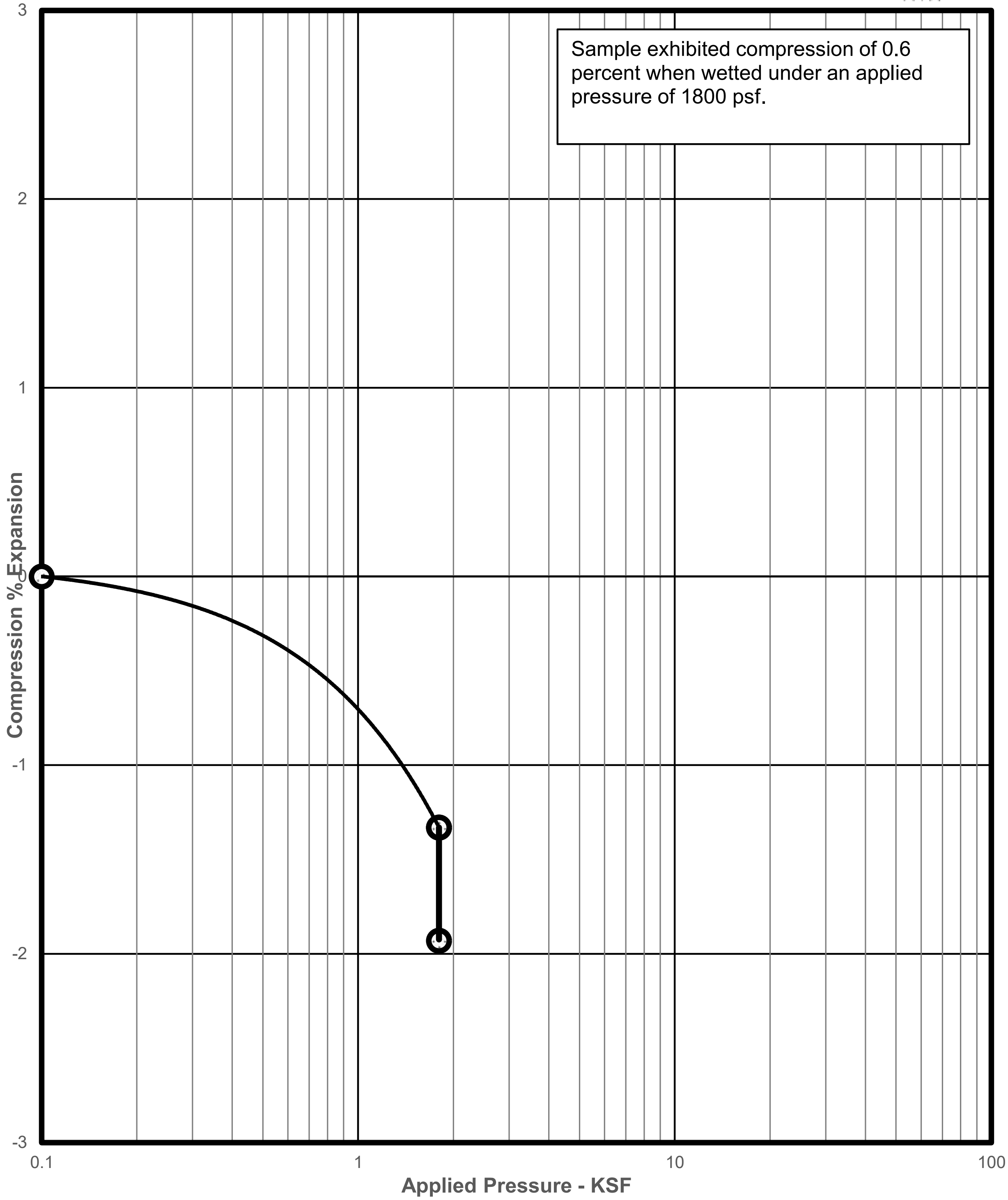
SHEA PROPERTIES

HELIX INDUSTRIAL BUILDINGS C AND D

CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results

FIG. B- 36

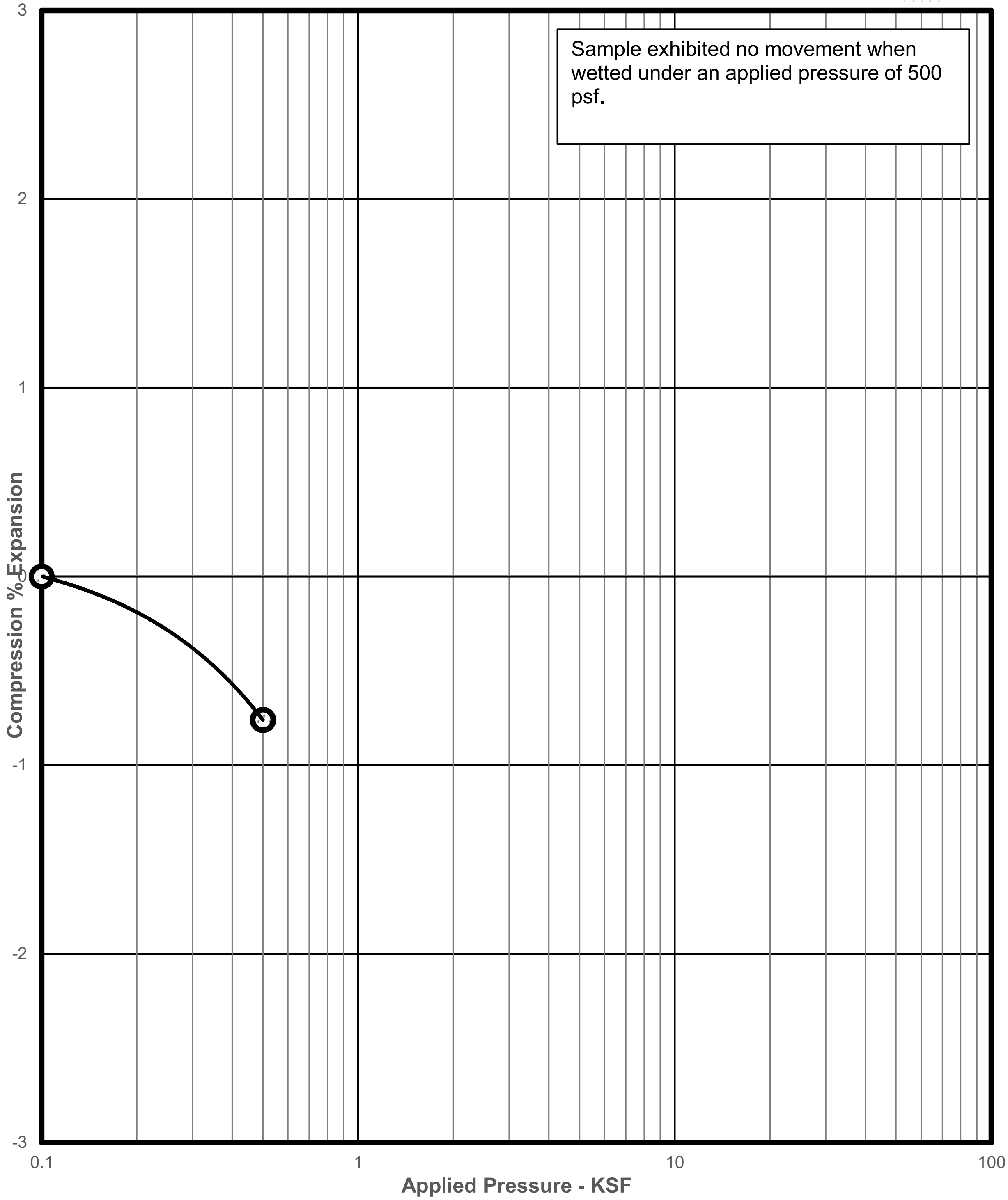


SAMPLE OF SANDSTONE
FROM: TH-9 AT 14 FEET

DRY UNIT WEIGHT: 110 pcf
MOISTURE CONTENT: 7.5 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 37

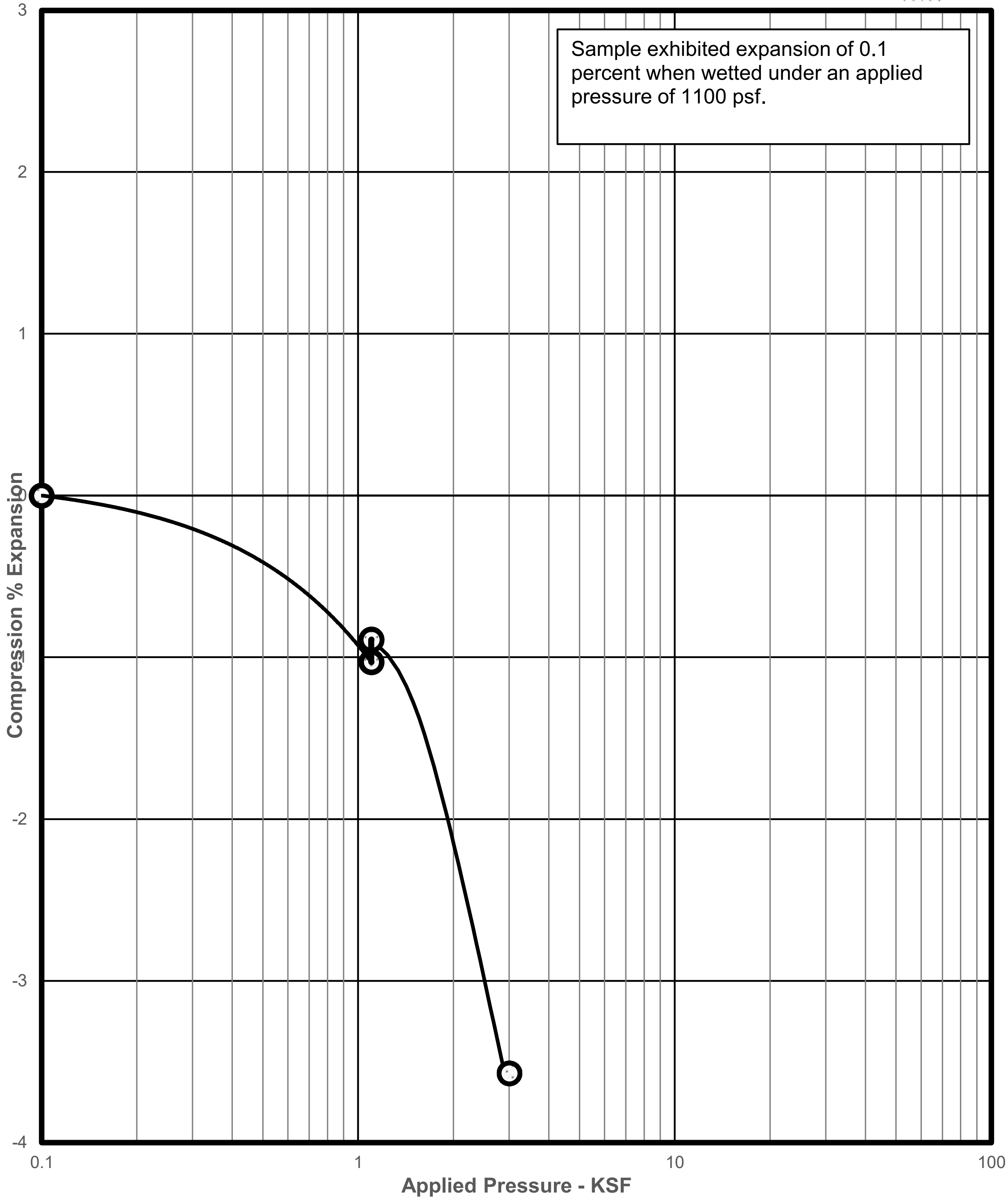


SAMPLE OF FILL, CLAY, SANDY
FROM: TH-10 AT 4 FEET

DRY UNIT WEIGHT: 112 pcf
MOISTURE CONTENT: 11.7 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 38

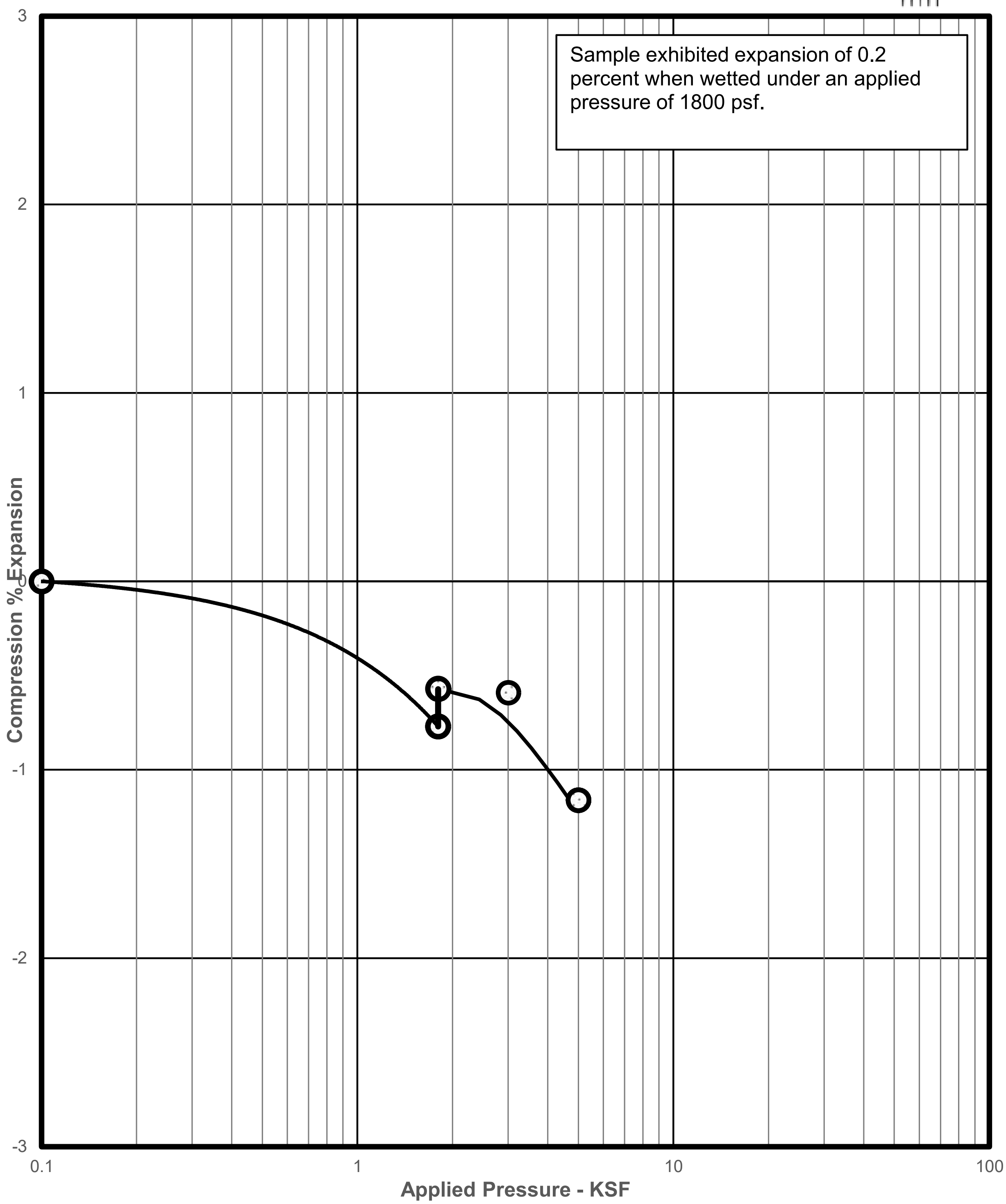


SAMPLE OF FILL, CLAY, SANDY
FROM: TH-10 AT 9 FEET

DRY UNIT WEIGHT: 104 pcf
MOISTURE CONTENT: 14.1 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 39

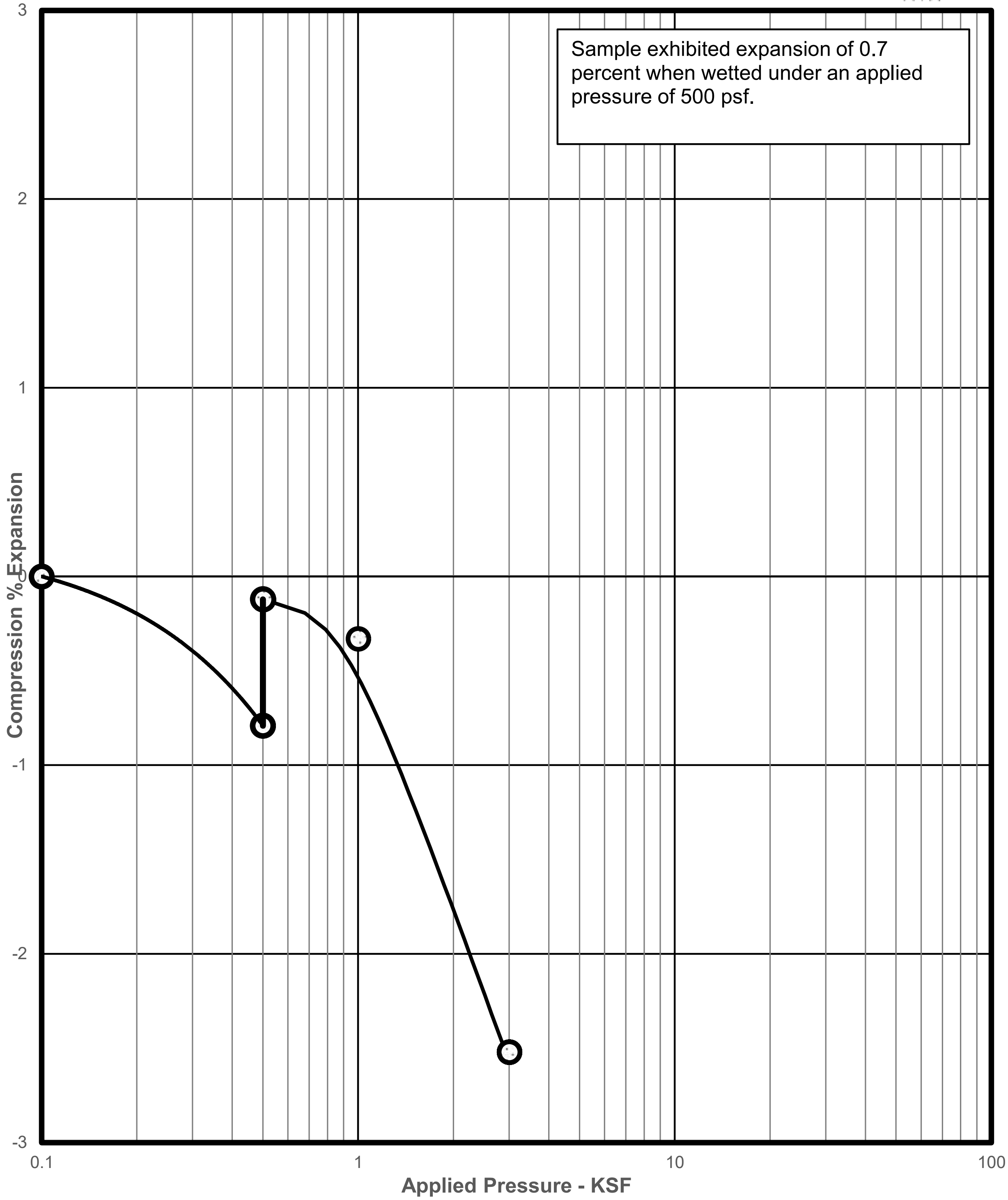


SAMPLE OF: CLAY, SANDY (CL)
FROM: TH-10 AT 14 FEET

DRY UNIT WEIGHT: 110 pcf
MOISTURE CONTENT: 16.7 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 40

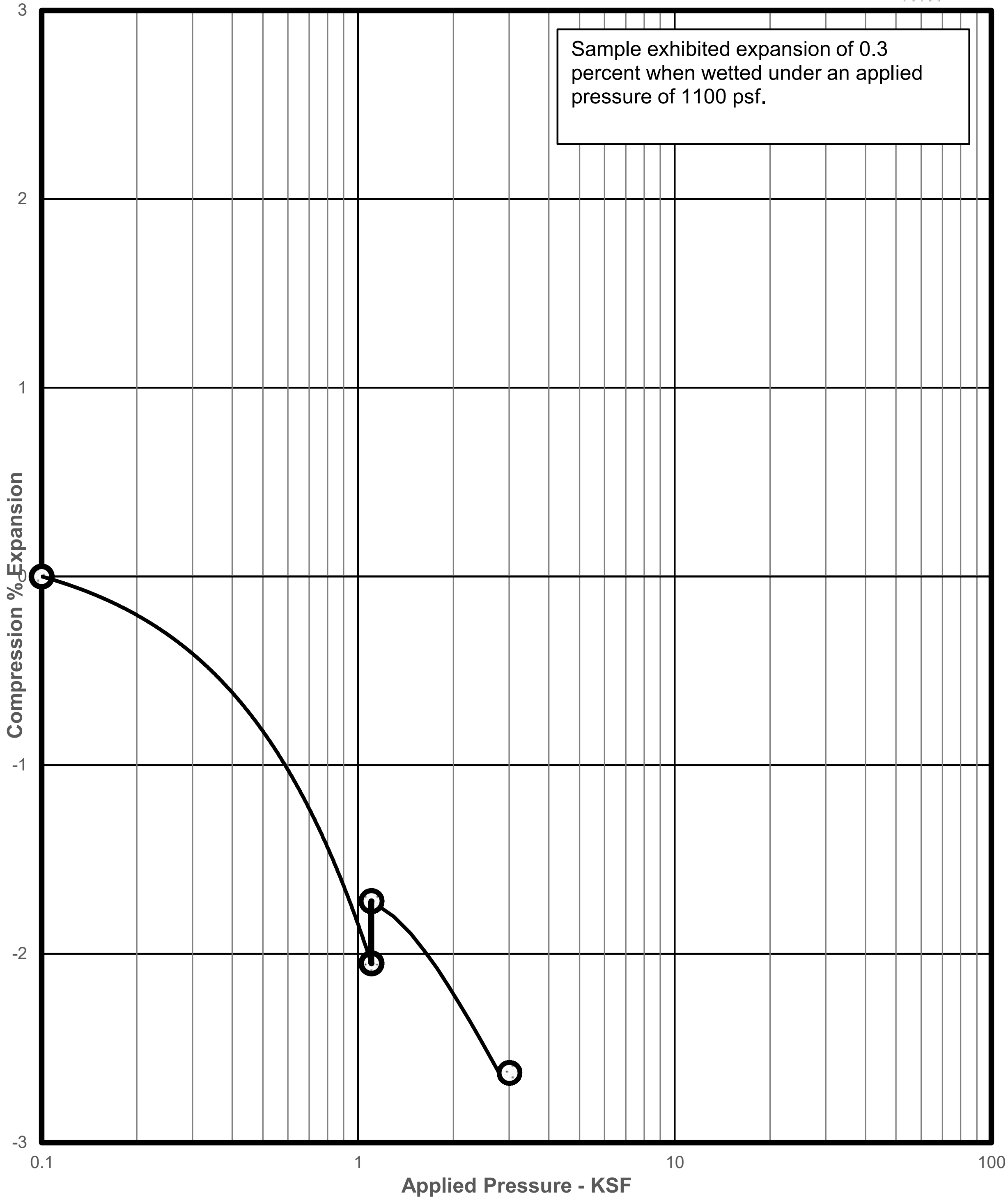


SAMPLE OF FILL, CLAY, SANDY
FROM: TH-11 AT 4 FEET

DRY UNIT WEIGHT: 100 pcf
MOISTURE CONTENT: 19.1 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

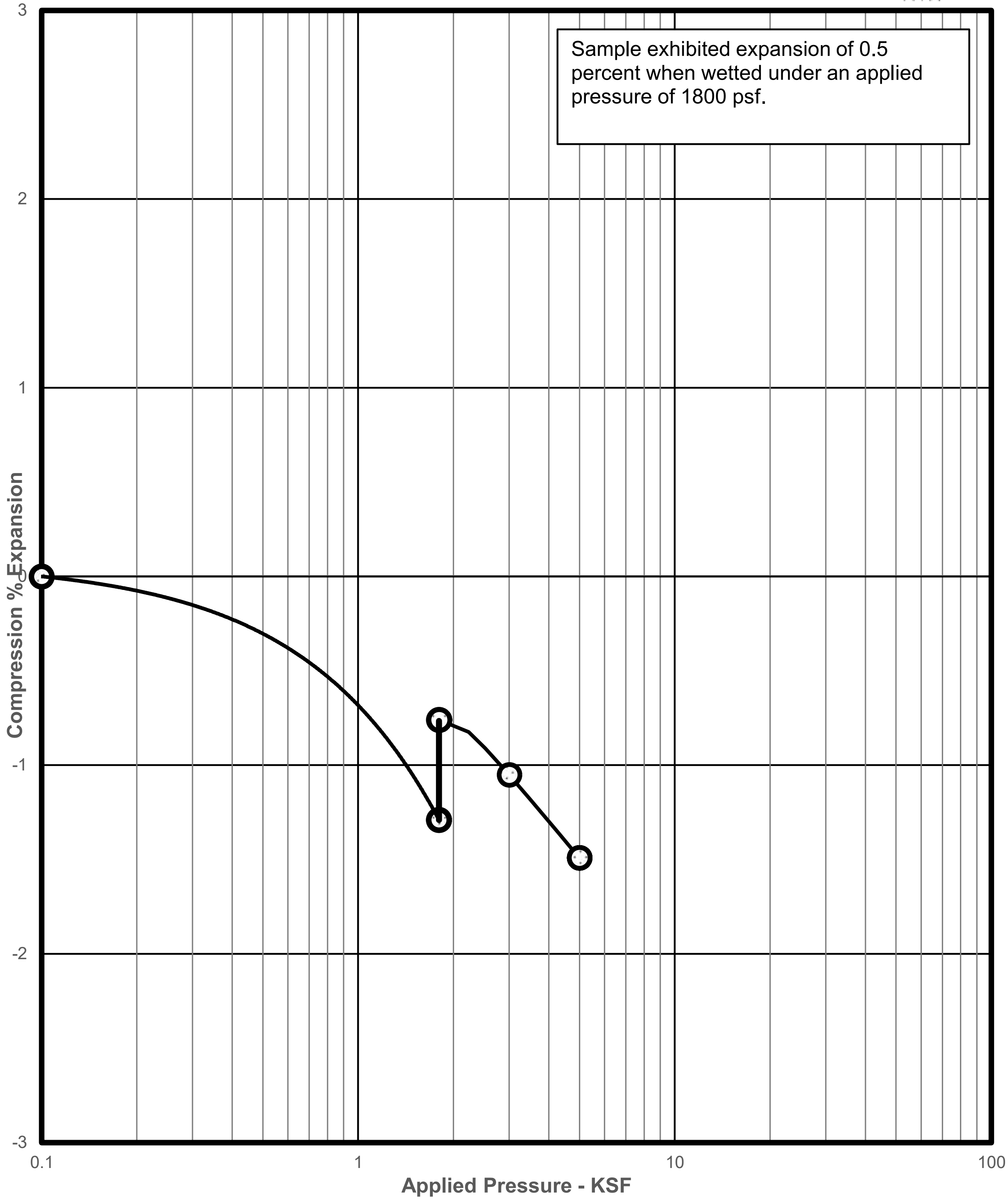
Swell Consolidation Test Results FIG. B- 41



SAMPLE OF: CLAY, SANDY (CL) DRY UNIT WEIGHT: 108 pcf
FROM: TH-11 AT 9 FEET MOISTURE CONTENT: 16.6 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 42

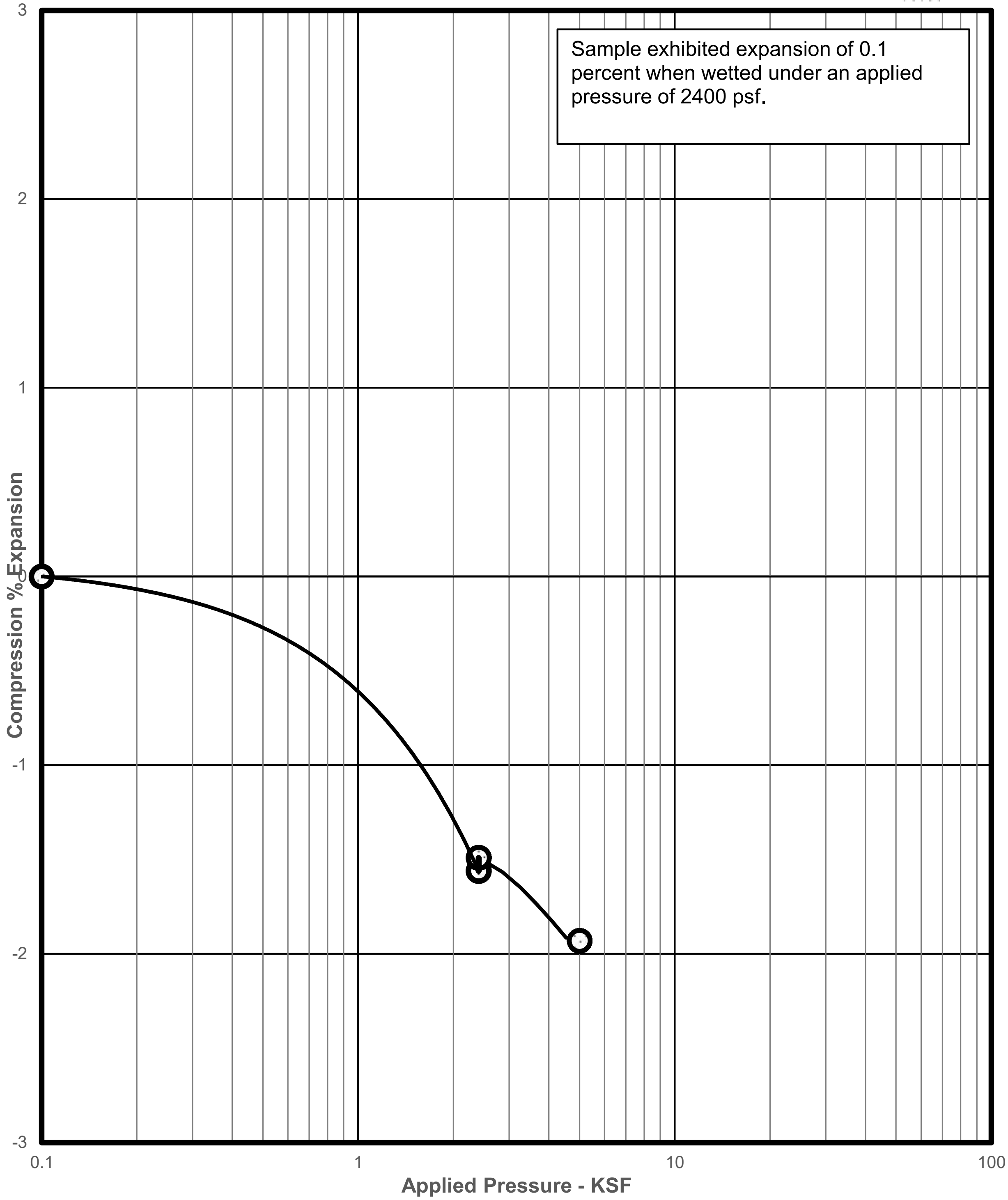


SAMPLE OF CLAYSTONE
FROM: TH-11 AT 14 FEET

DRY UNIT WEIGHT: 113 pcf
MOISTURE CONTENT: 16.5 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 43



SAMPLE OF CLAYSTONE

FROM: TH-11 AT 19 FEET

DRY UNIT WEIGHT: 112 pcf

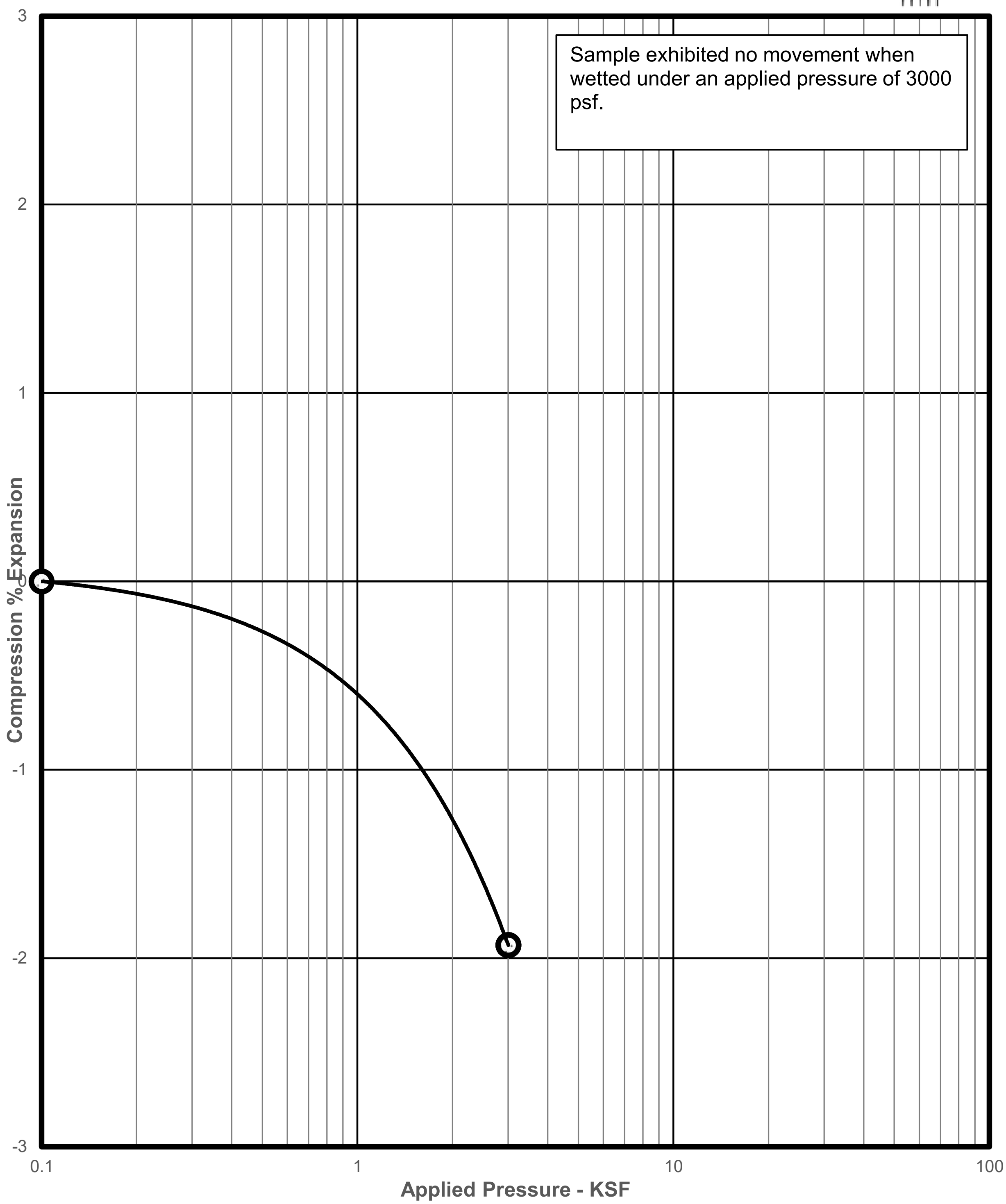
MOISTURE CONTENT: 16.7 %

SHEA PROPERTIES

HELIX INDUSTRIAL BUILDINGS C AND D

CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 44

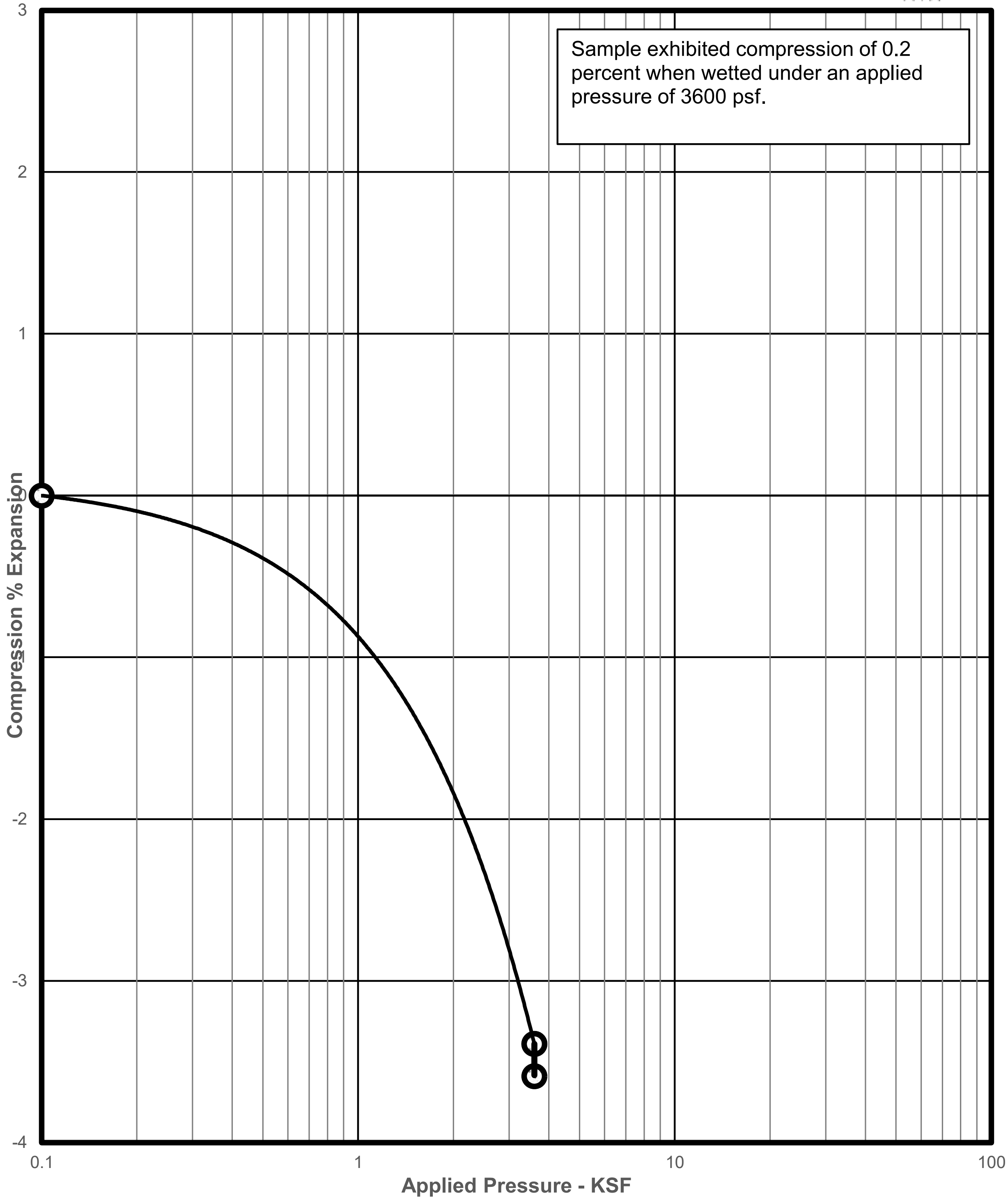


SAMPLE OF CLAYSTONE
FROM: TH-11 AT 24 FEET

DRY UNIT WEIGHT: 117 pcf
MOISTURE CONTENT: 15.1 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 45



SAMPLE OF SANDSTONE | DRY UNIT WEIGHT: 97 pcf

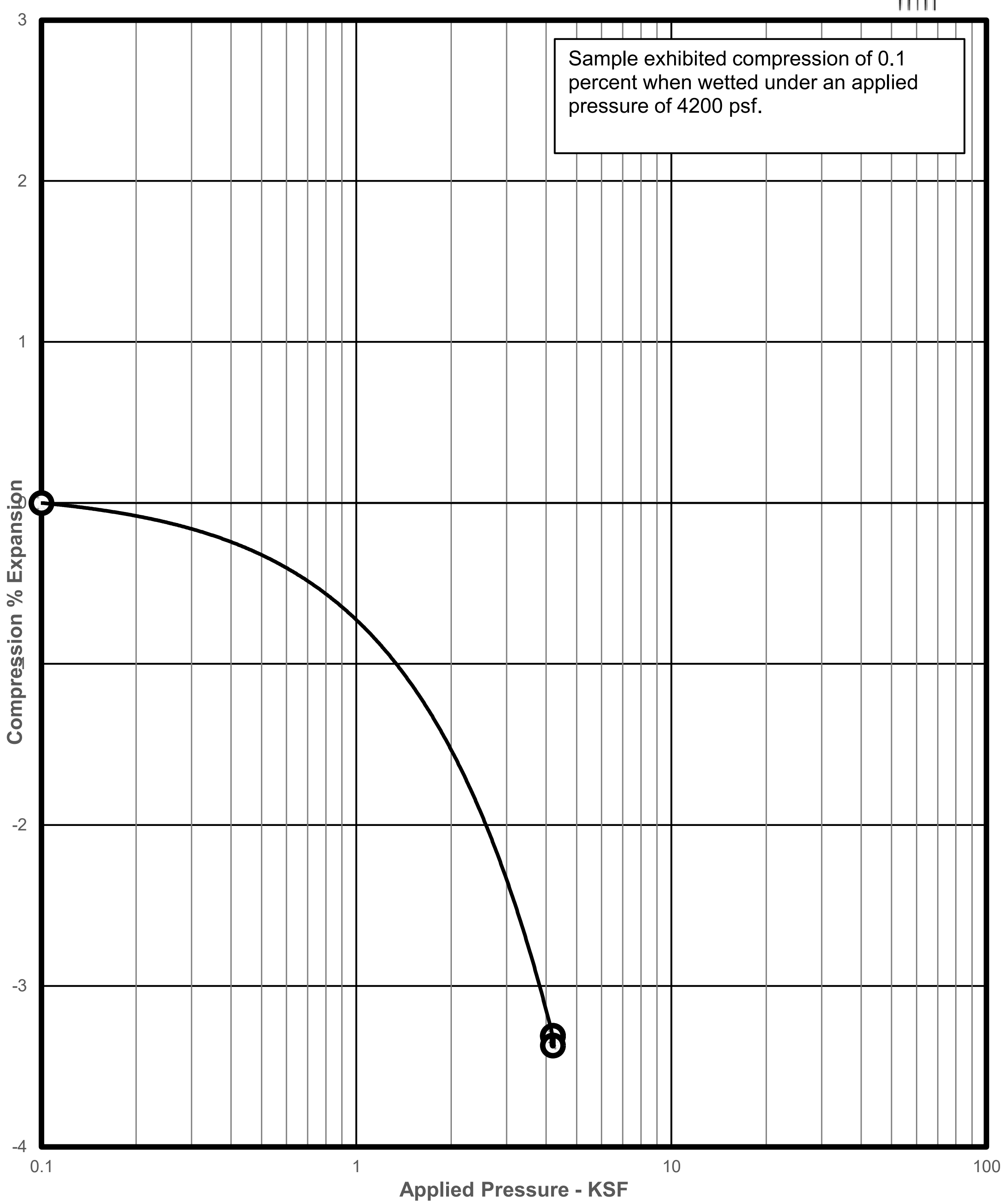
FROM: TH-11 AT 29 FEET | MOISTURE CONTENT: 19.3 %

SHEA PROPERTIES

HELIX INDUSTRIAL BUILDINGS C AND D

CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 46

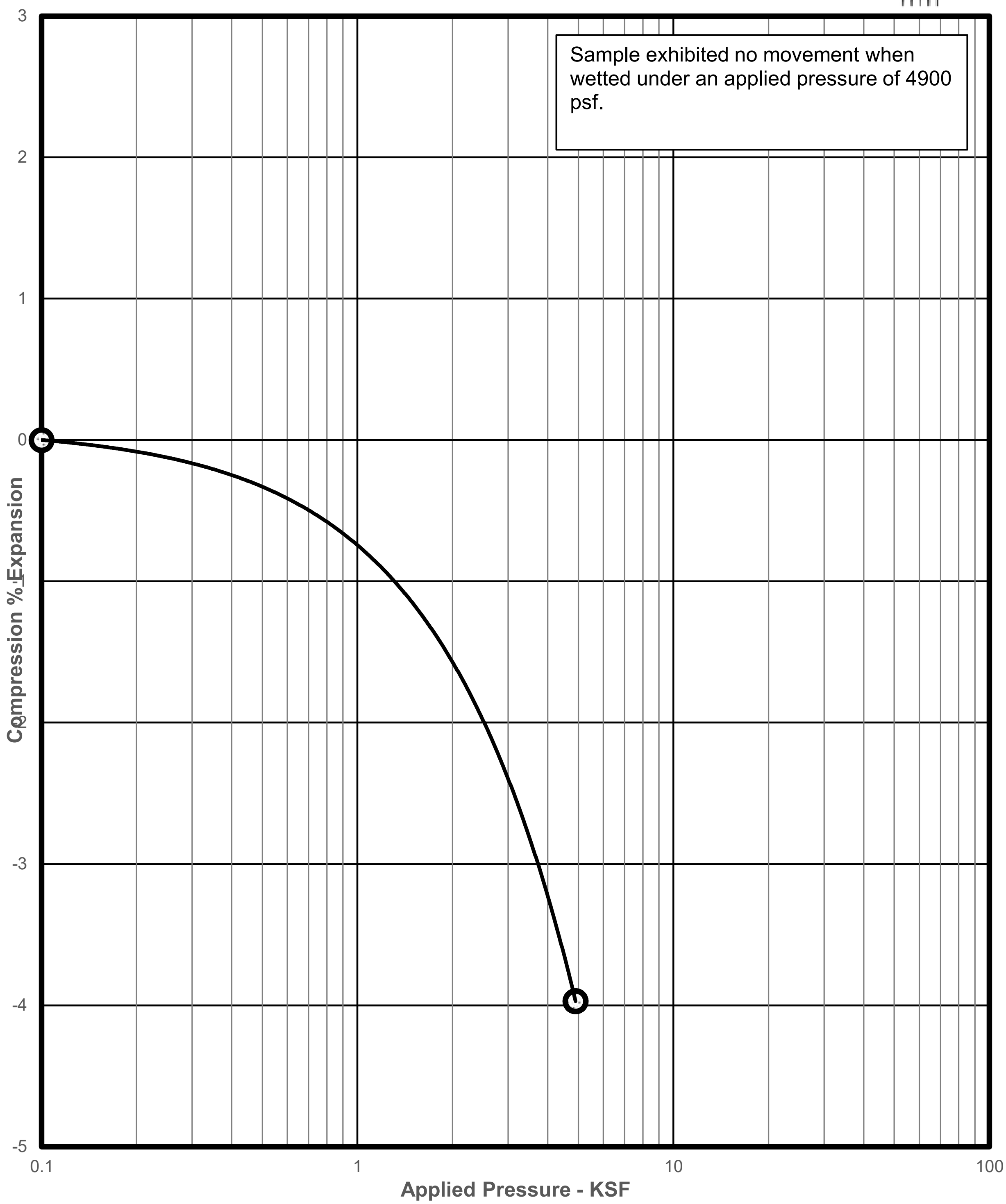


SAMPLE OF SANDSTONE
FROM: TH-11 AT 34 FEET

DRY UNIT WEIGHT: 105 pcf
MOISTURE CONTENT: 19.1 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 47



SAMPLE OF SANDSTONE — DRY UNIT WEIGHT: 101 pcf

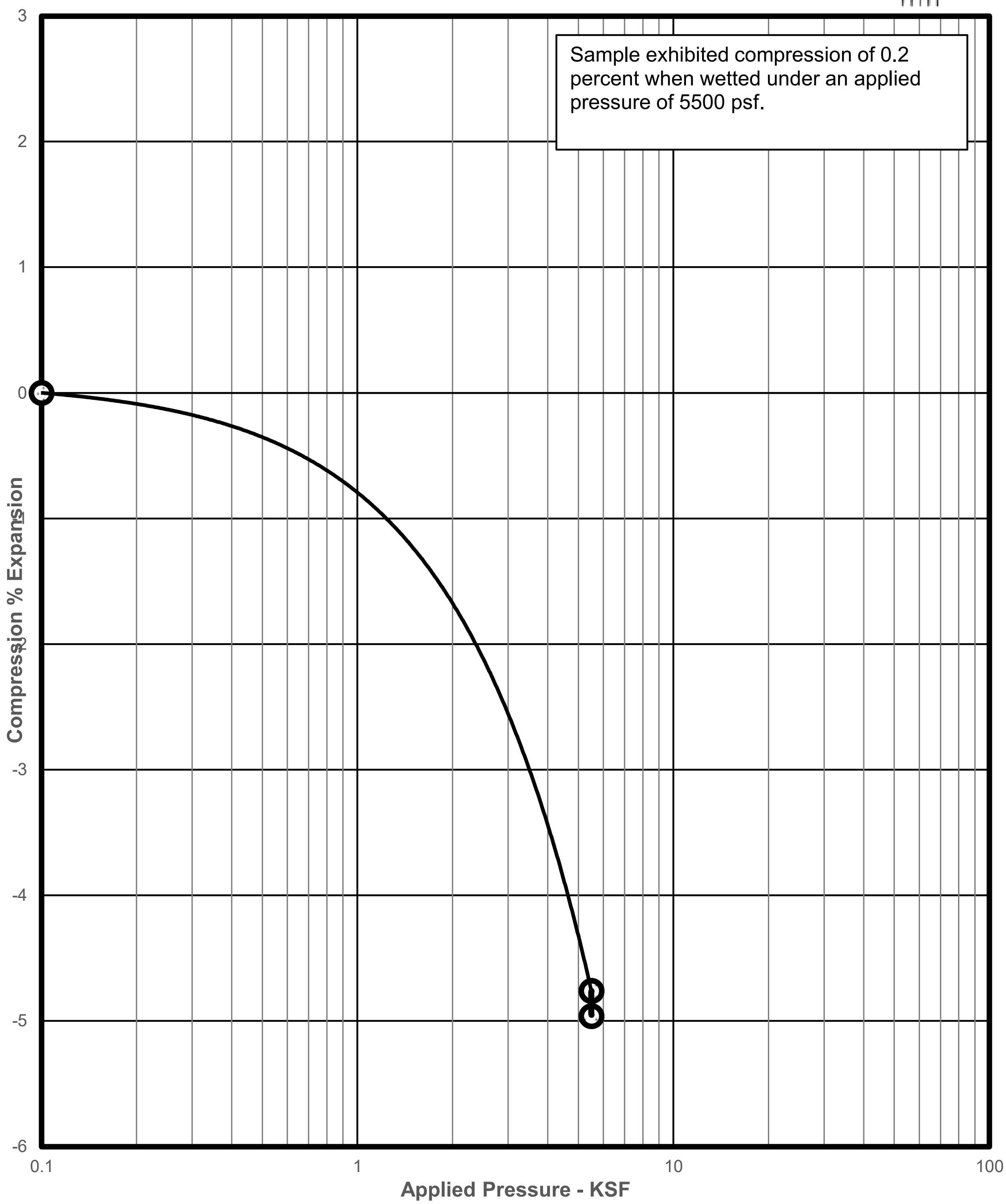
FROM: TH-11 AT 39 FEET — MOISTURE CONTENT: 16.4 %

SHEA PROPERTIES

HELIX INDUSTRIAL BUILDINGS C AND D

CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 48



SAMPLE OF SANDSTONE

FROM: TH-11 AT 44 FEET

DRY UNIT WEIGHT: 100 pcf

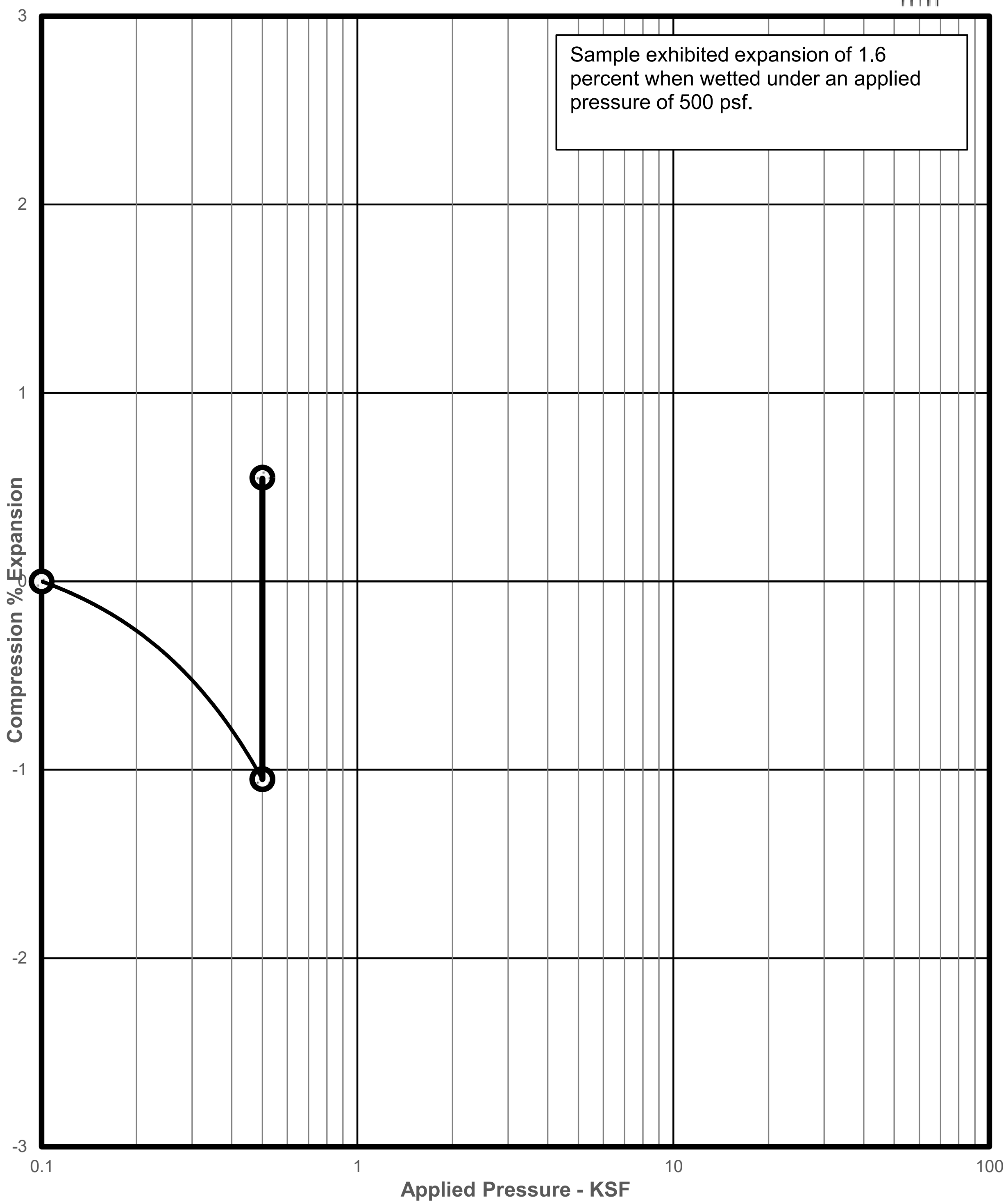
MOISTURE CONTENT: 20.6 %

SHEA PROPERTIES

HELIX INDUSTRIAL BUILDINGS C AND D

CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 49



SAMPLE OF FILL, CLAY, SANDY

FROM: TH-12 AT 4 FEET

DRY UNIT WEIGHT: 105 pcf

MOISTURE CONTENT: 21.1 %

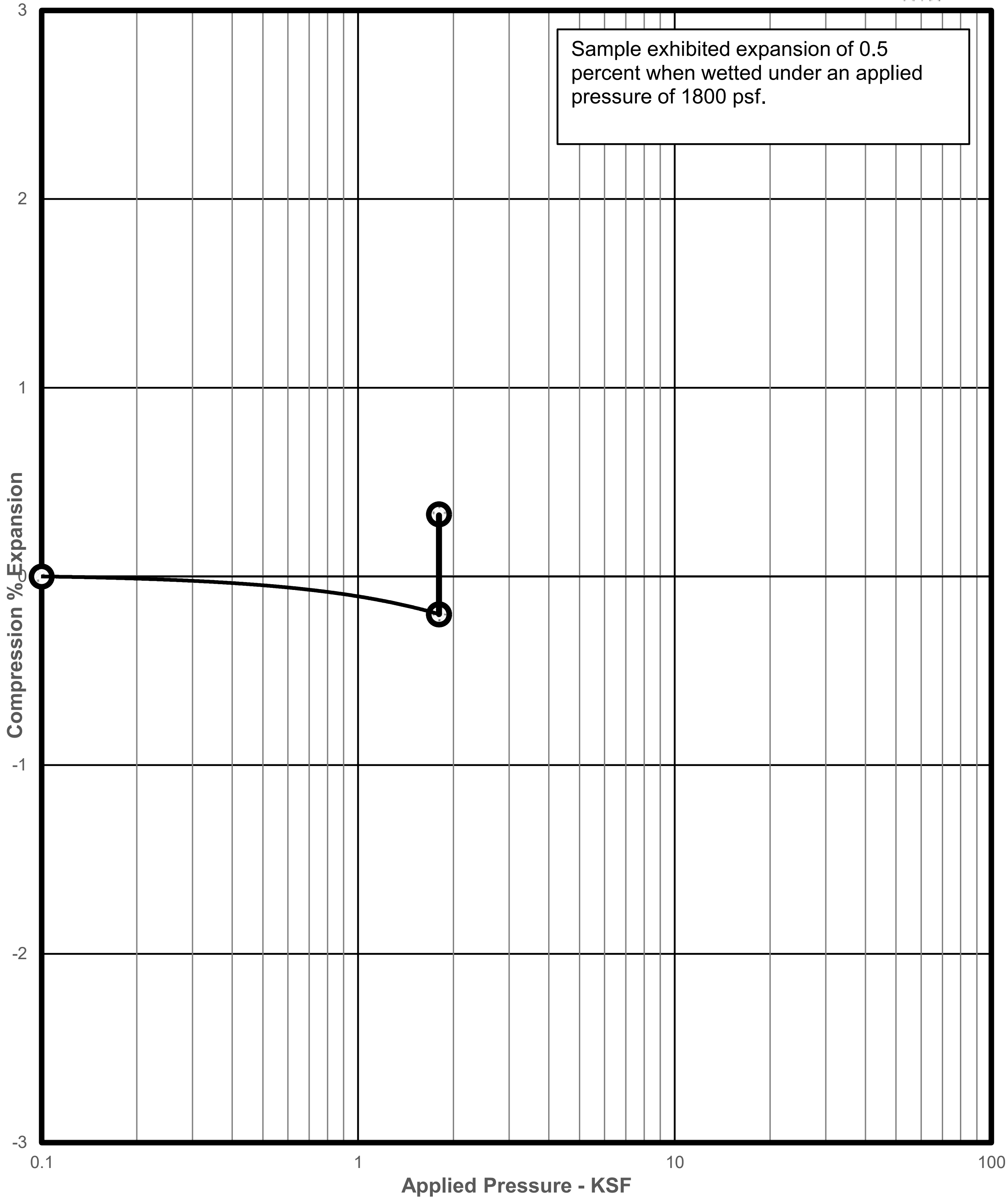
SHEA PROPERTIES

HELIX INDUSTRIAL BUILDINGS C AND D

CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results

FIG. B- 50

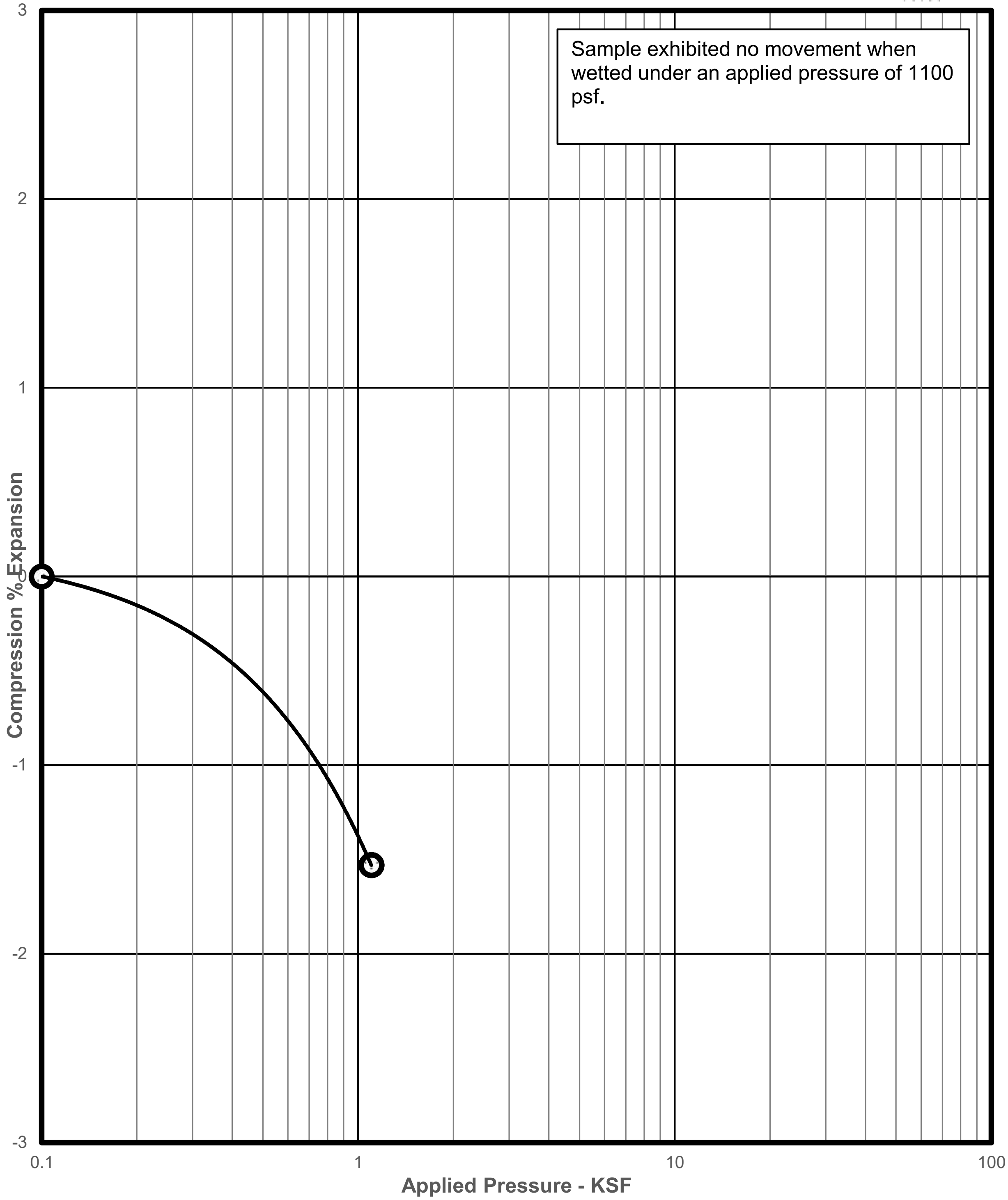


SAMPLE OF CLAYSTONE
FROM: TH-12 AT 14 FEET

DRY UNIT WEIGHT: 112 pcf
MOISTURE CONTENT: 17.8 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 51

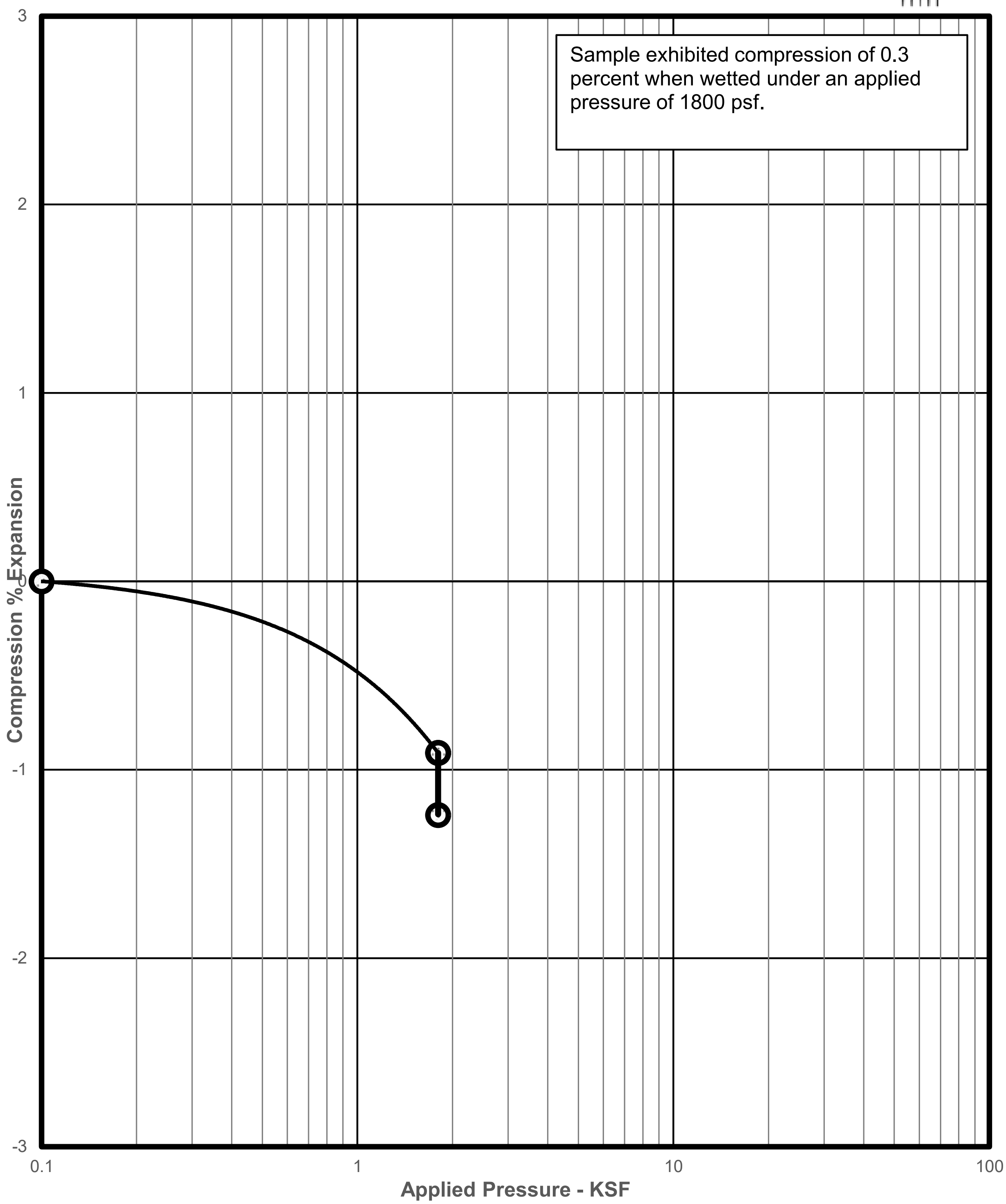


SAMPLE OF: CLAY, SANDY (CL)
FROM: TH-13 AT 9 FEET

DRY UNIT WEIGHT: 112 pcf
MOISTURE CONTENT: 15.4 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 52

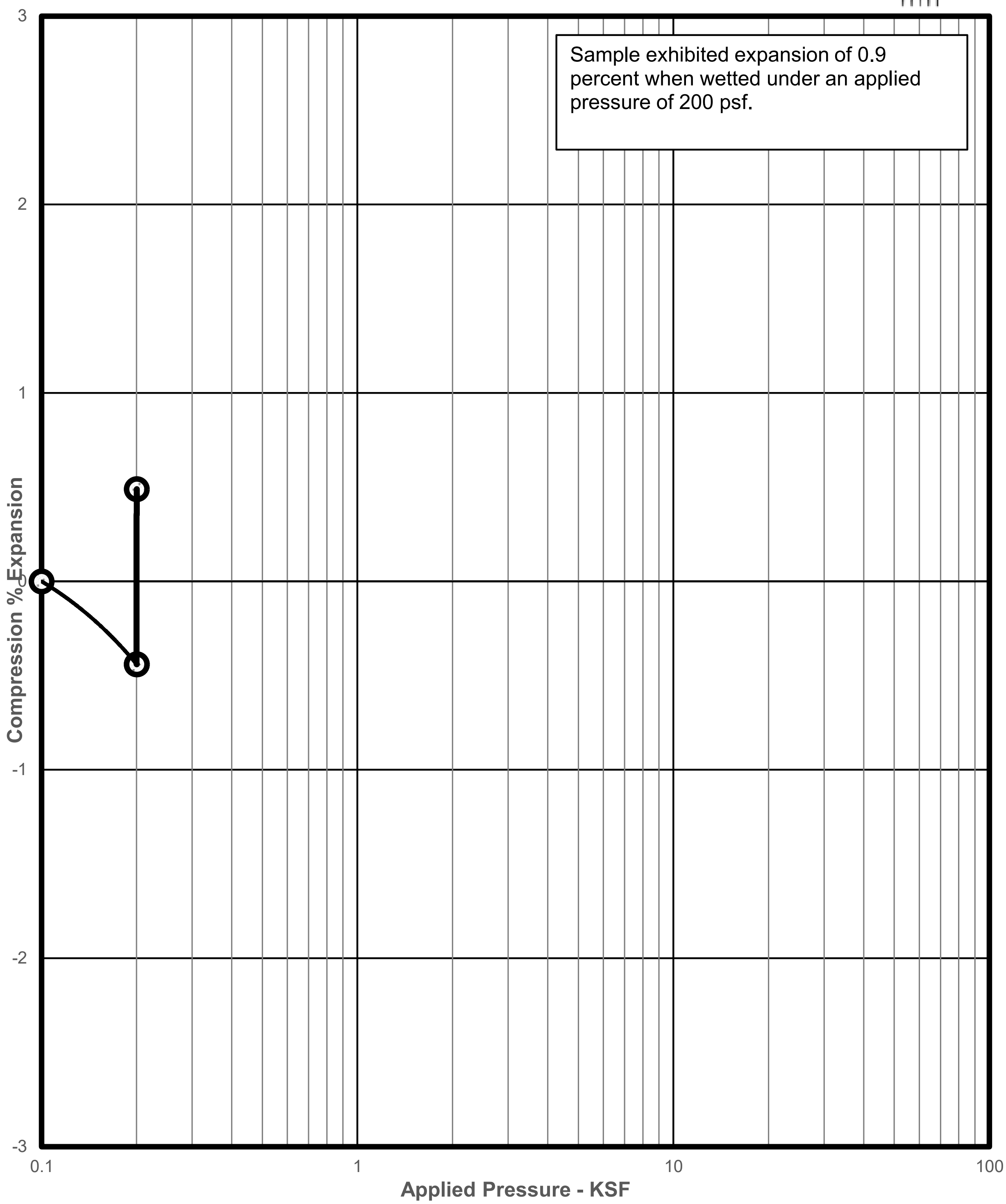


SAMPLE OF SANDSTONE
FROM: TH-13 AT 14 FEET

DRY UNIT WEIGHT: 112 pcf
MOISTURE CONTENT: 11.6 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 53

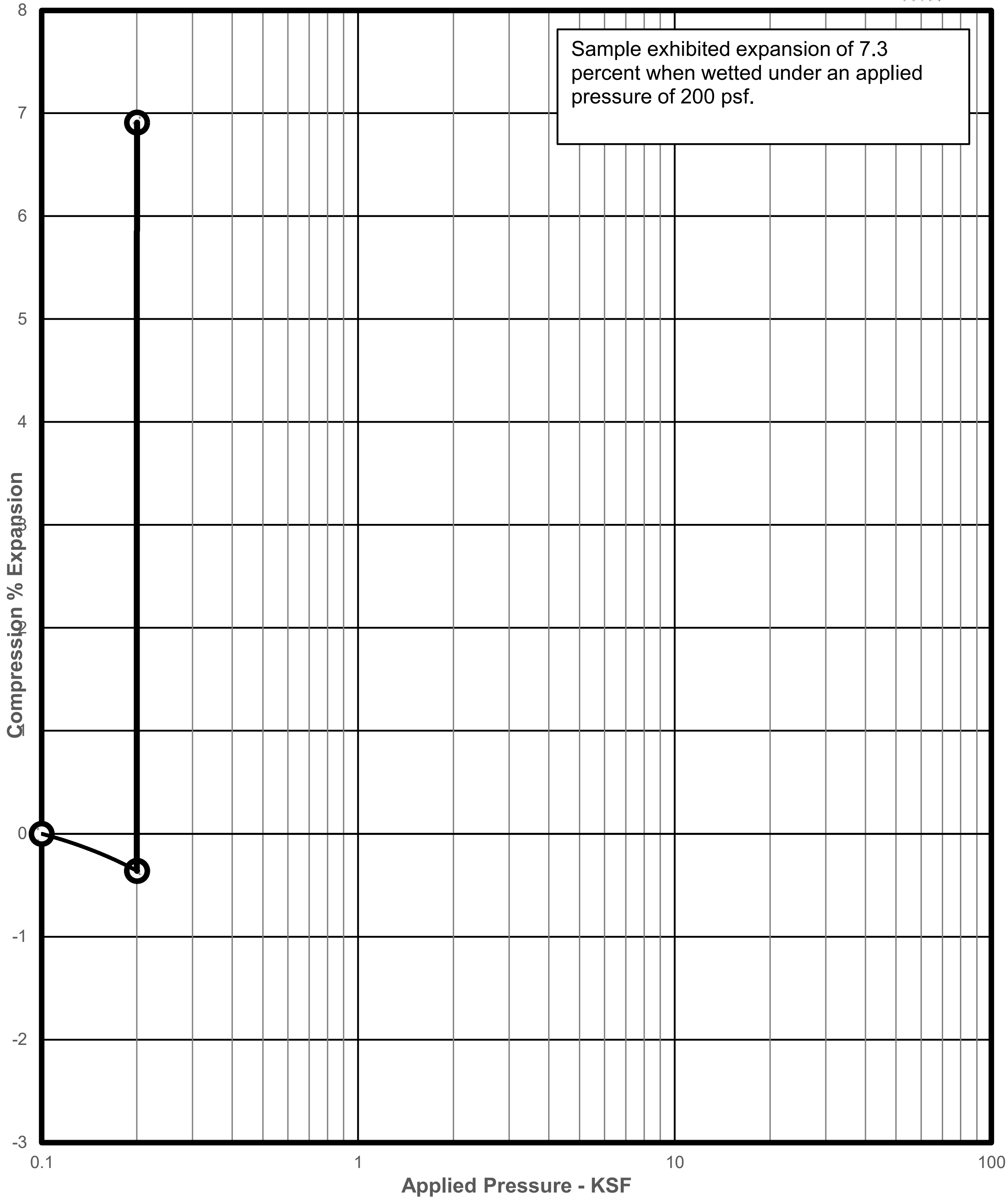


SAMPLE OF SANDSTONE
FROM: S-1 AT 9 FEET

DRY UNIT WEIGHT: 117 pcf
MOISTURE CONTENT: 12.5 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 54

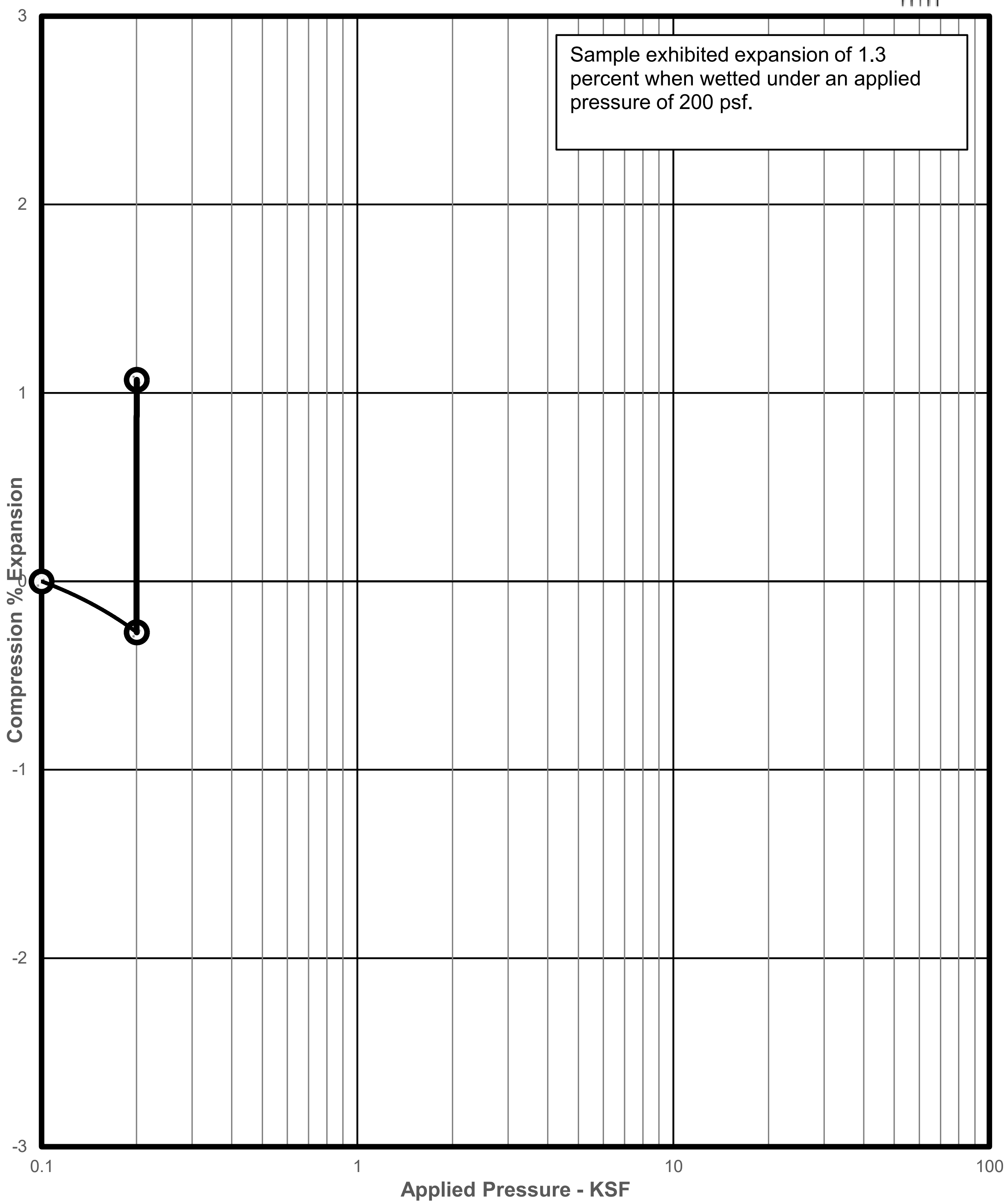


SAMPLE OF CLAYSTONE
FROM: S-2 AT 2 FEET

DRY UNIT WEIGHT: 107 pcf
MOISTURE CONTENT: 13.0 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 55

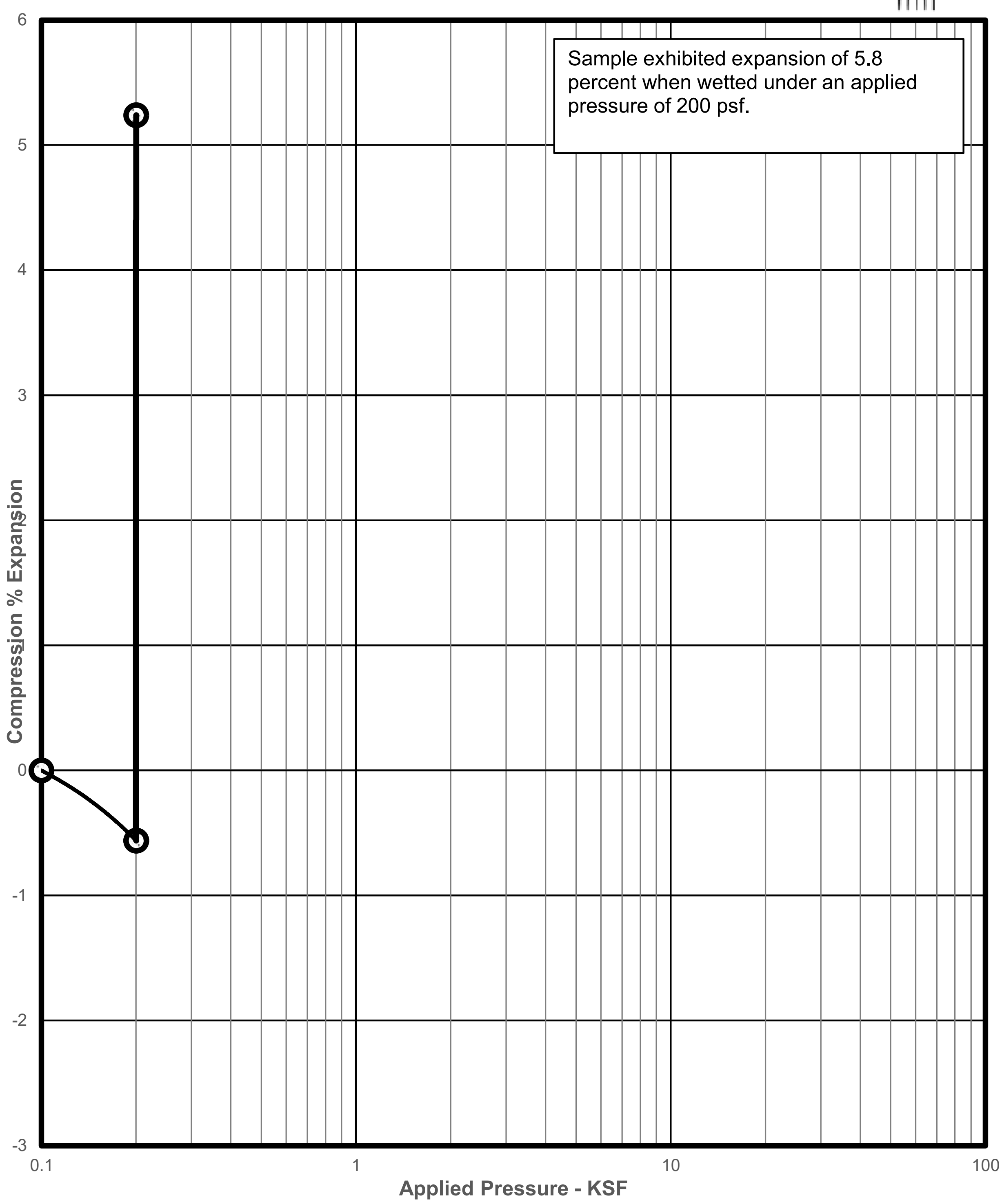


SAMPLE OF FILL, CLAY, SANDY
FROM: S-3 AT 4 FEET

DRY UNIT WEIGHT: 113 pcf
MOISTURE CONTENT: 15.2 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

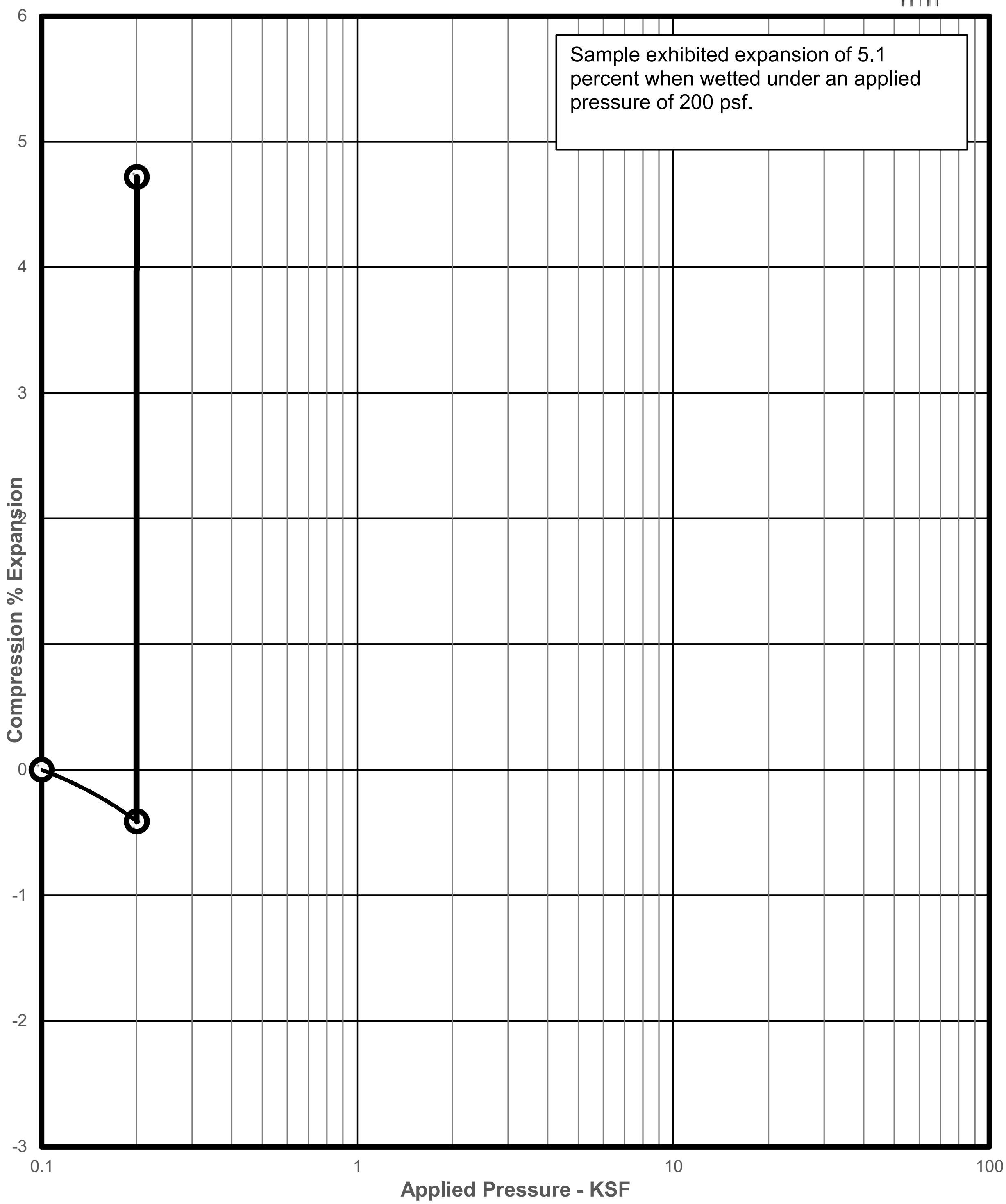
Swell Consolidation Test Results FIG. B- 56



SAMPLE OF:	FILL, CLAY, SANDY	DRY UNIT WEIGHT:	101	pcf
FROM:	S-4 AT 2 FEET	MOISTURE CONTENT:	11.9	%

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 57

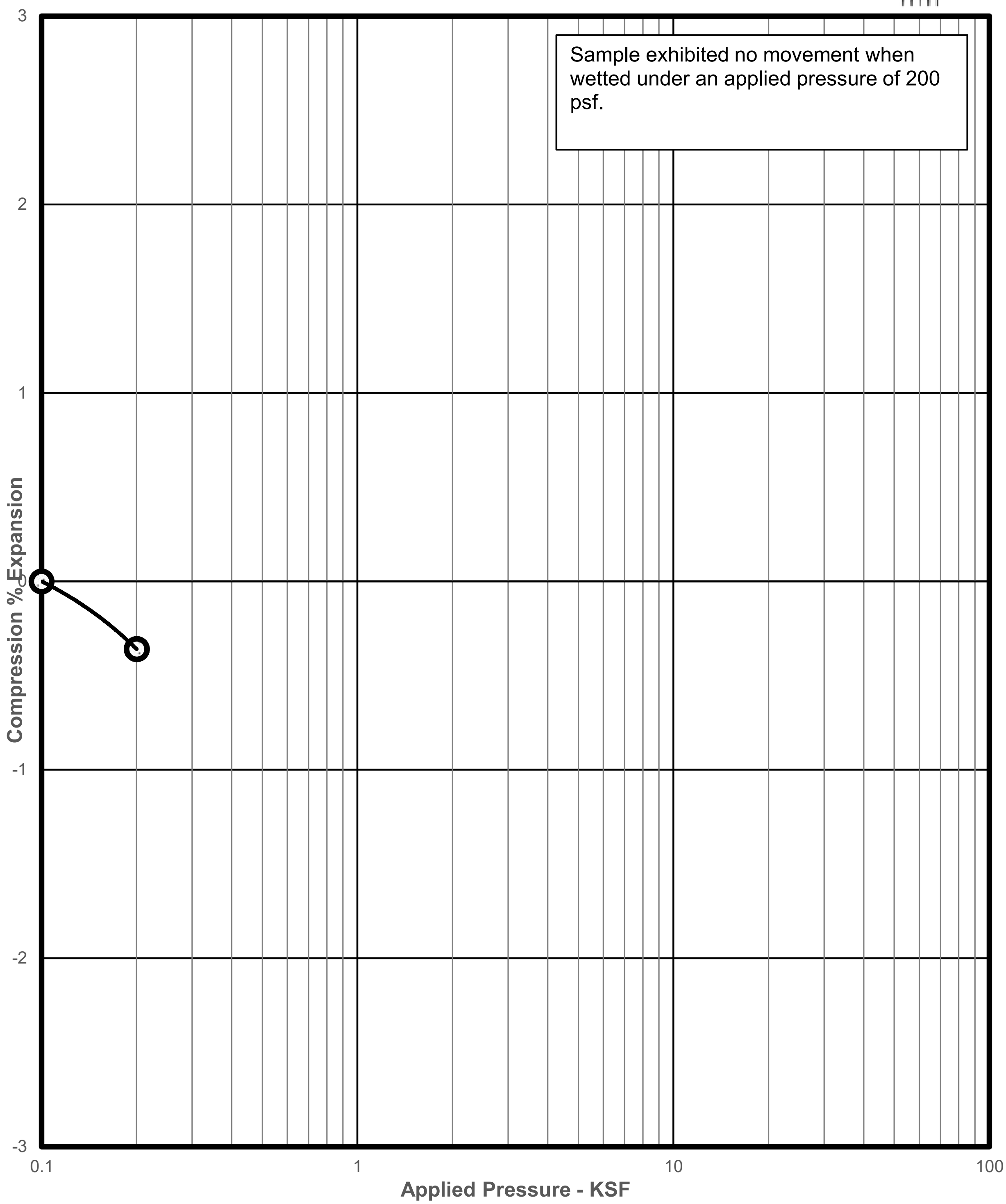


SAMPLE OF: CLAY, SANDY (CL)
FROM: S-5 AT 4 FEET

DRY UNIT WEIGHT: 109 pcf
MOISTURE CONTENT: 15.2 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 58

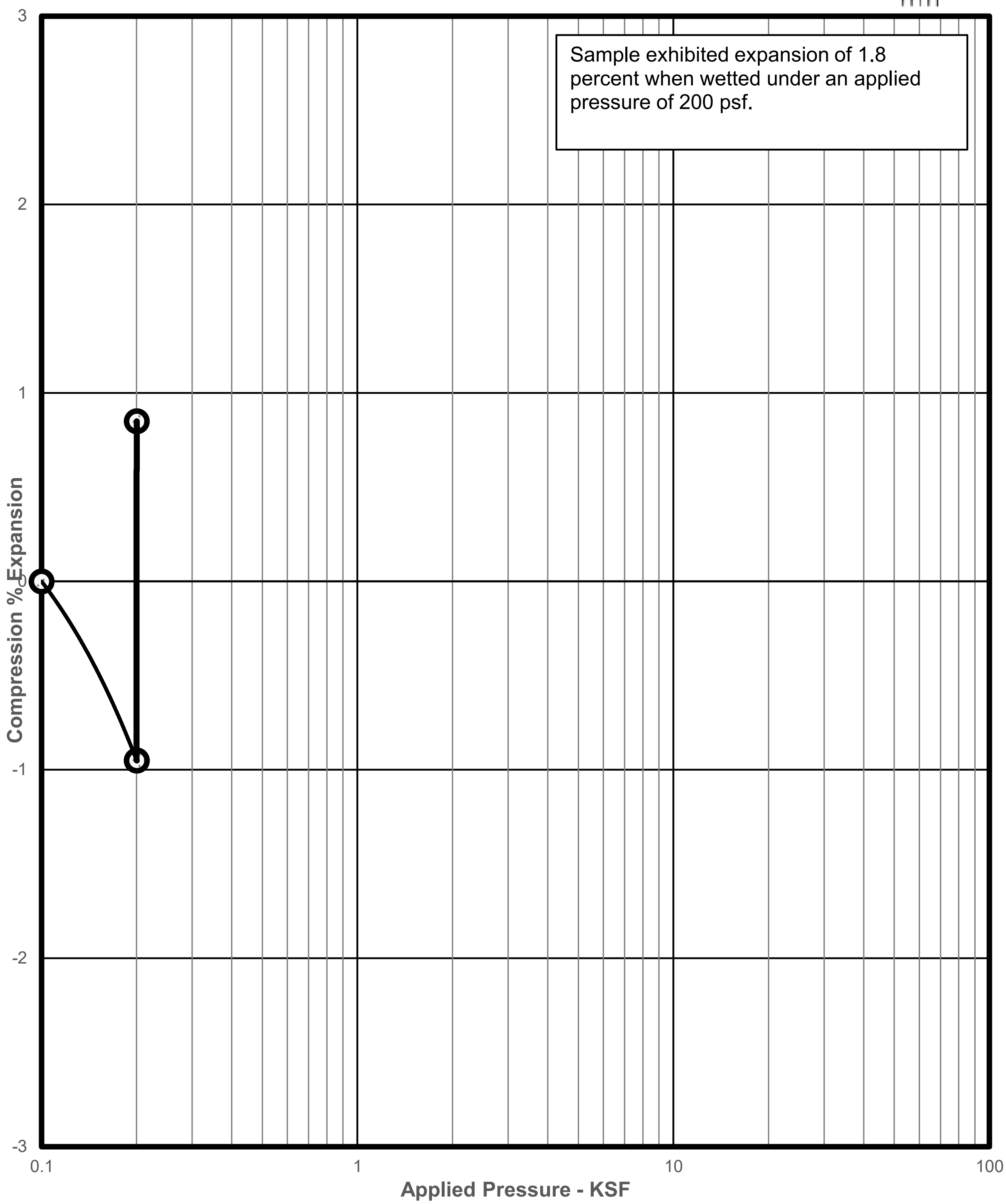


SAMPLE OF FILL, SAND, CLAYEY
FROM: S-6 AT 1 FEET

DRY UNIT WEIGHT: 108 pcf
MOISTURE CONTENT: 9.7 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 59

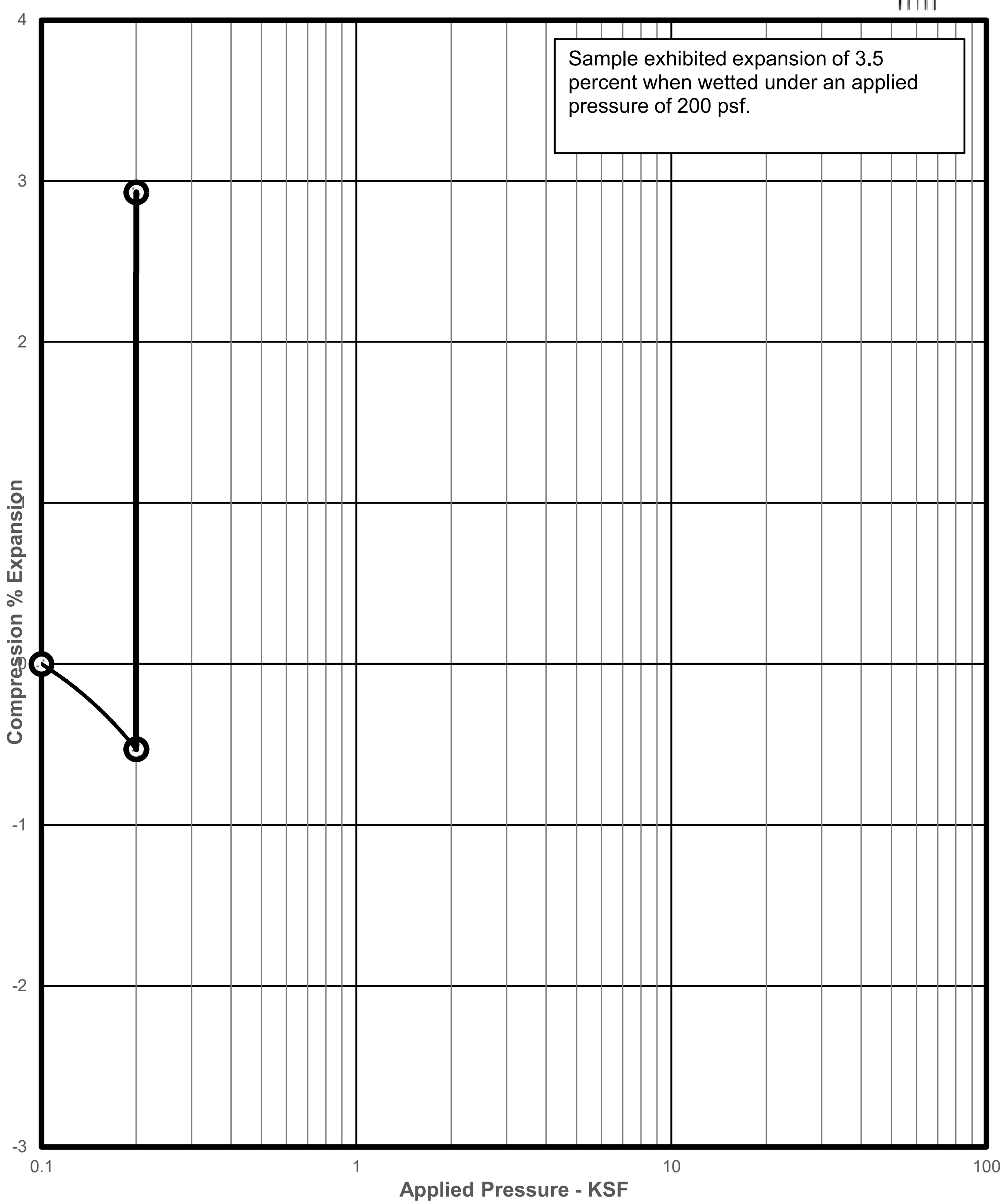


SAMPLE OF: CLAYSTONE
FROM: S-7 AT 1 FEET

DRY UNIT WEIGHT: 89 pcf
MOISTURE CONTENT: 18.7 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 60



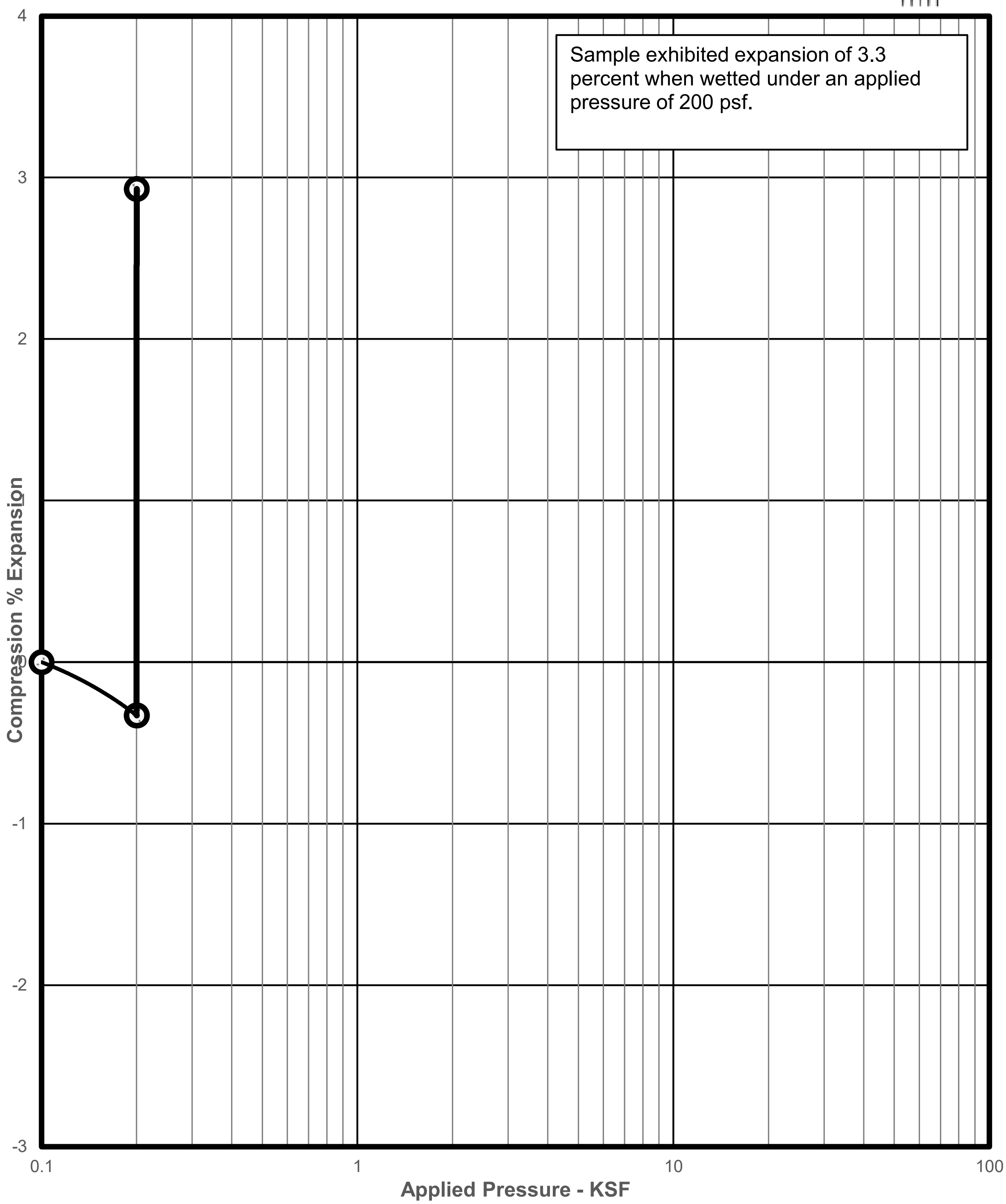
SAMPLE OF:	CLAYSTONE	DRY UNIT WEIGHT:	111	pcf
FROM:	S-7 AT 4 FEET	MOISTURE CONTENT:	17.8	%

SHEA PROPERTIES

HELIX INDUSTRIAL BUILDINGS C AND D

CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 61

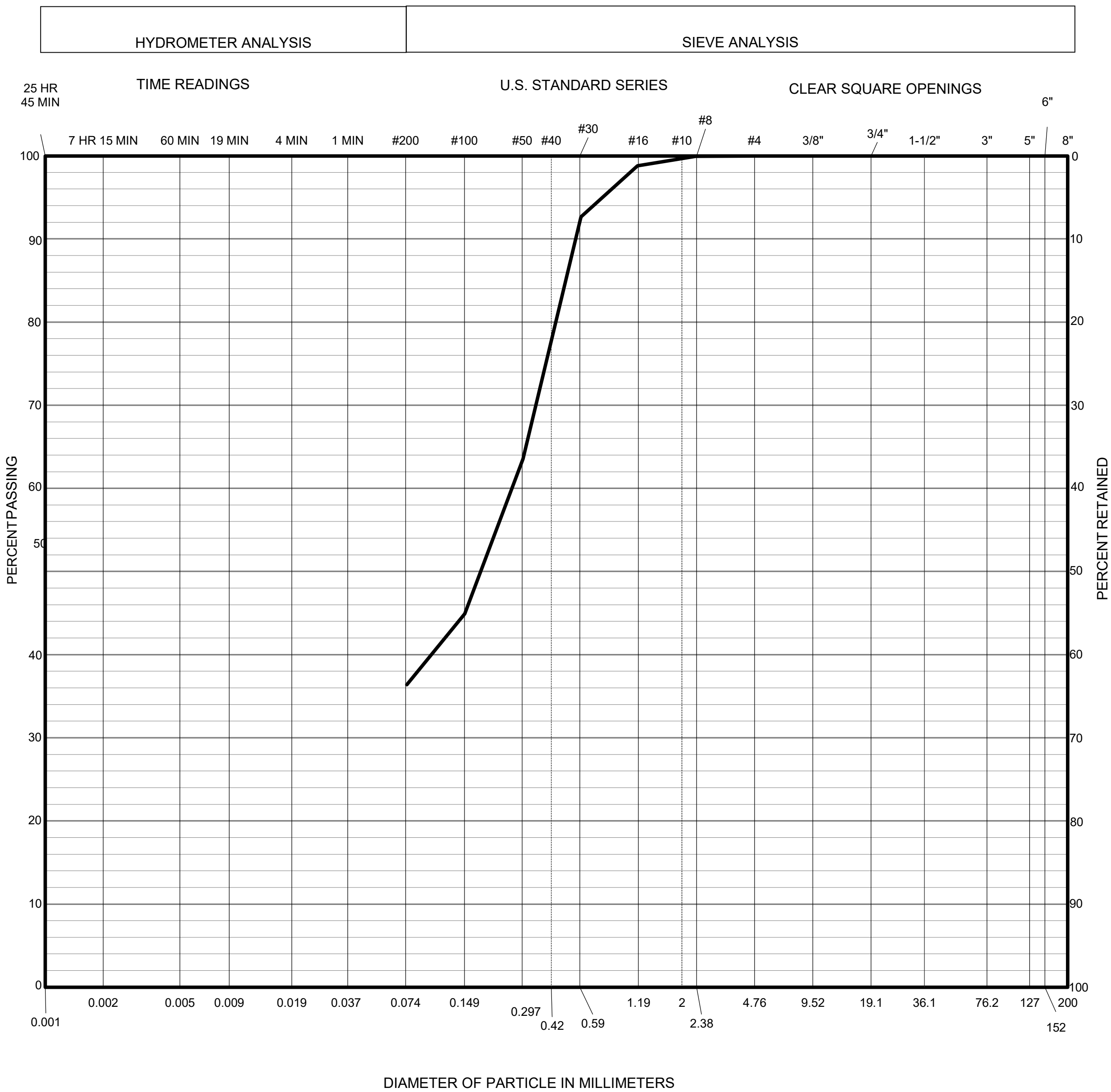


SAMPLE OF: INTERBEDDED CLAYSTONE/SANDSTONE
FROM: S-8 AT 1 FEET

DRY UNIT WEIGHT: 112 pcf
MOISTURE CONTENT: 7.3 %

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Swell Consolidation Test Results FIG. B- 62



CLAY (PLASTIC) TO SILT (NON-PLASTIC)	SANDS			GRAVEL		
	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLES

SAMPLE OF: SANDSTONE
FROM: S-4 AT 7 FEET

GRAVEL: 0.0% SAND: 63.6%
SILT & CLAY: 36.4% LIQUID LIMIT: 38
PLASTIC INDEX: 15

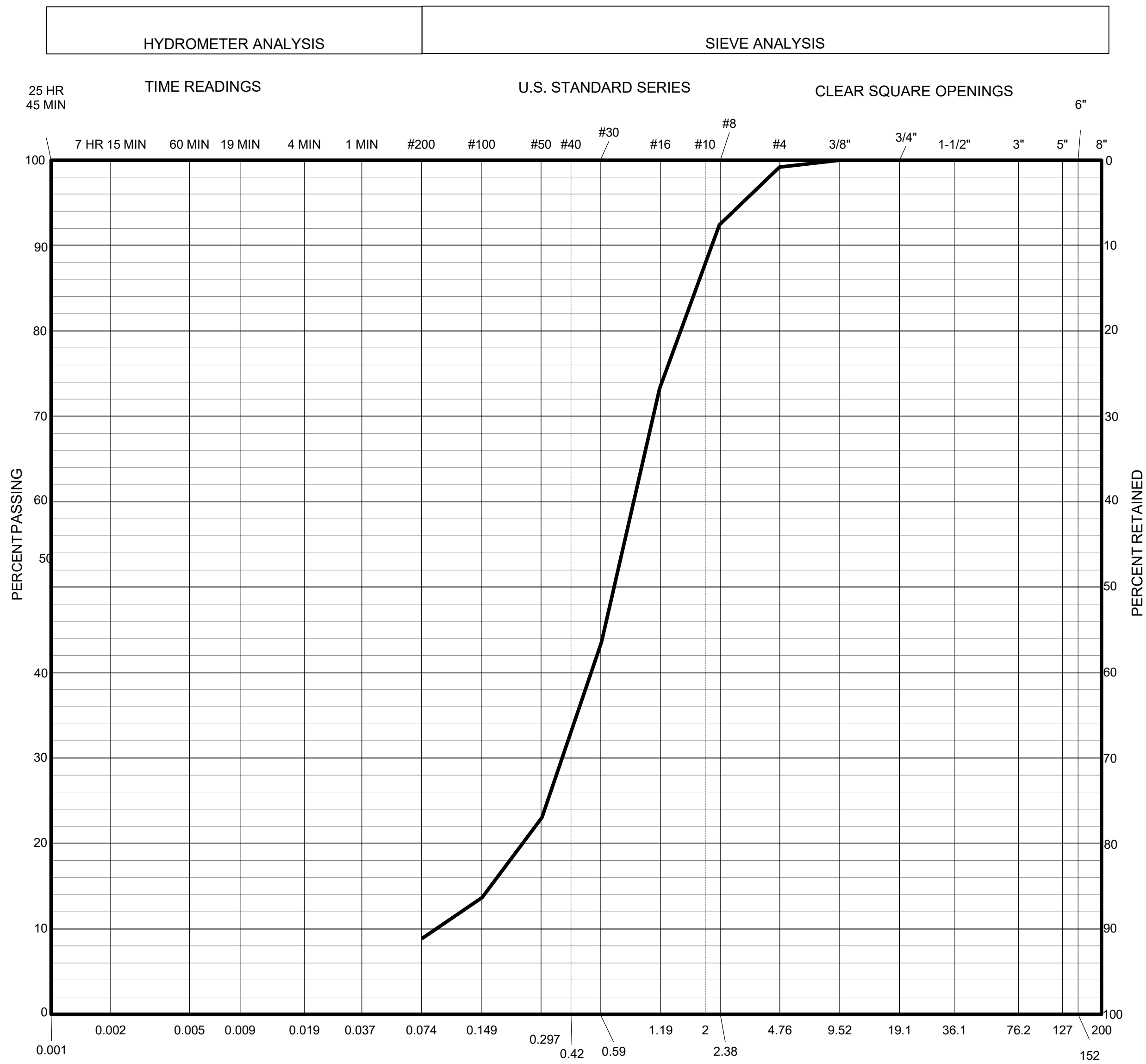
D10 -- D30 -- D60: 0.272 CU: -- CC: --

Size (mm)	-	-	-	-	-	-	4.8	2.4	1.2	0.6	0.3	0.2	0.1	-	-	-	-	-	-
% Passing	-	-	-	-	-	-	100.0	99.9	98.8	92.7	63.5	44.9	36.4	-	-	-	-	-	-

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Gradation Test Results

FIG. B- 63



DIAMETER OF PARTICLE IN MILLIMETERS

CLAY (PLASTIC) TO SILT (NON-PLASTIC)	SANDS			GRAVEL		
	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLES

SAMPLE OF: SANDSTONE
FROM: S-8 AT 4 FEET

GRAVEL: 0.8%
SILT & CLAY: 8.9%
PLASTIC INDEX: -

SAND: 90.2%
LIQUID LIMIT: -

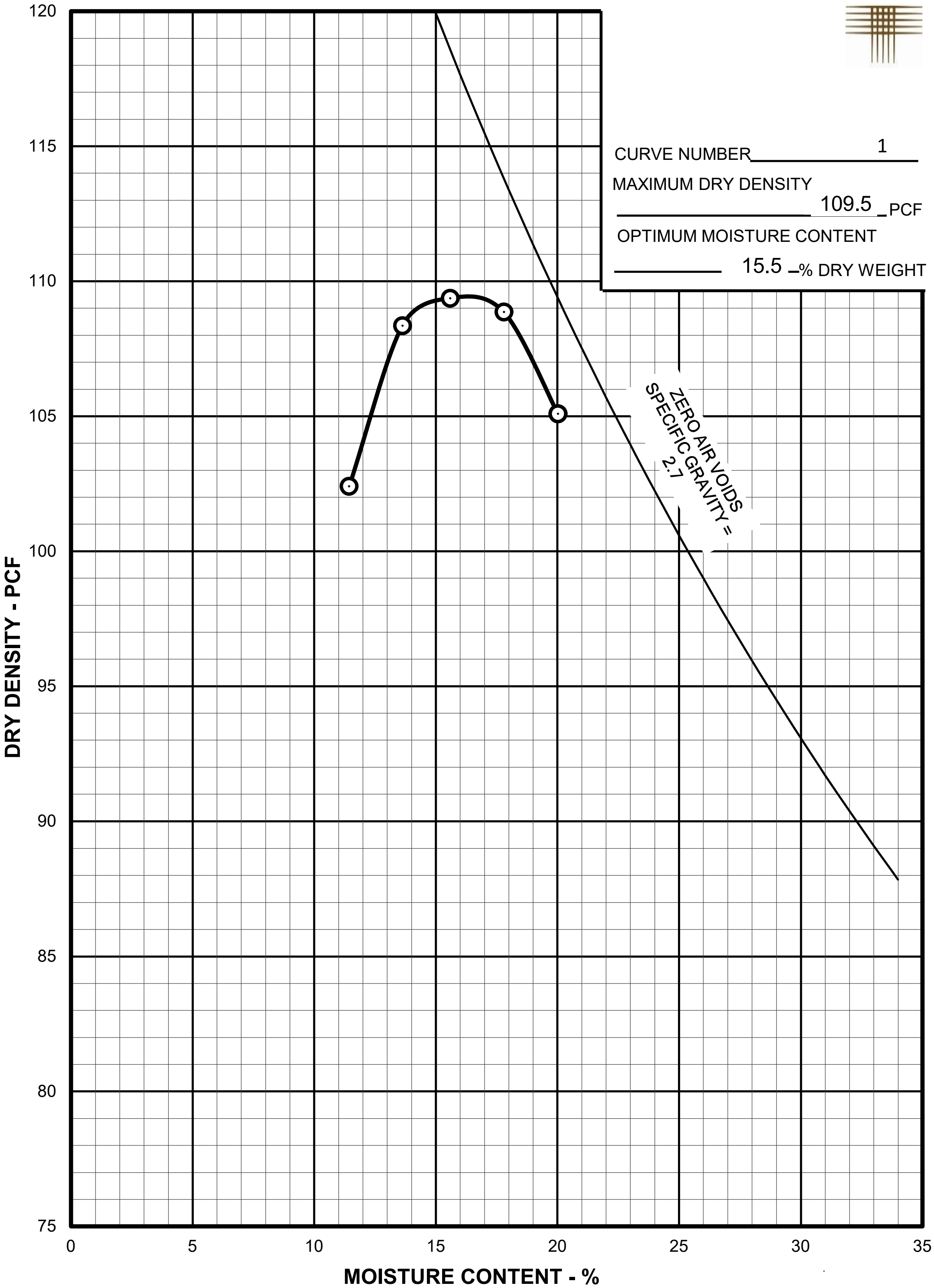
D1(0.092 D30 0.402 D60: 0.922 CU: 10.0 CC: 1.9

Size (mm)	-	-	-	-	-	9.5	4.8	2.4	1.2	0.6	0.3	0.2	0.1	-	-	-	-	-	-
% Passing	-	-	-	-	-	100.0	99.2	92.4	73.2	43.5	23.0	13.7	8.9	-	-	-	-	-	-

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

Gradation Test Results

FIG. B- 64



Sample Description Clay, Sandy (CL)

Location GROUP I SOILS (A-7-6)

Test Procedure ASTM D698

ASTM A

LIQUID LIMIT 42

PLASTICITY INDEX 26

GRAVEL %

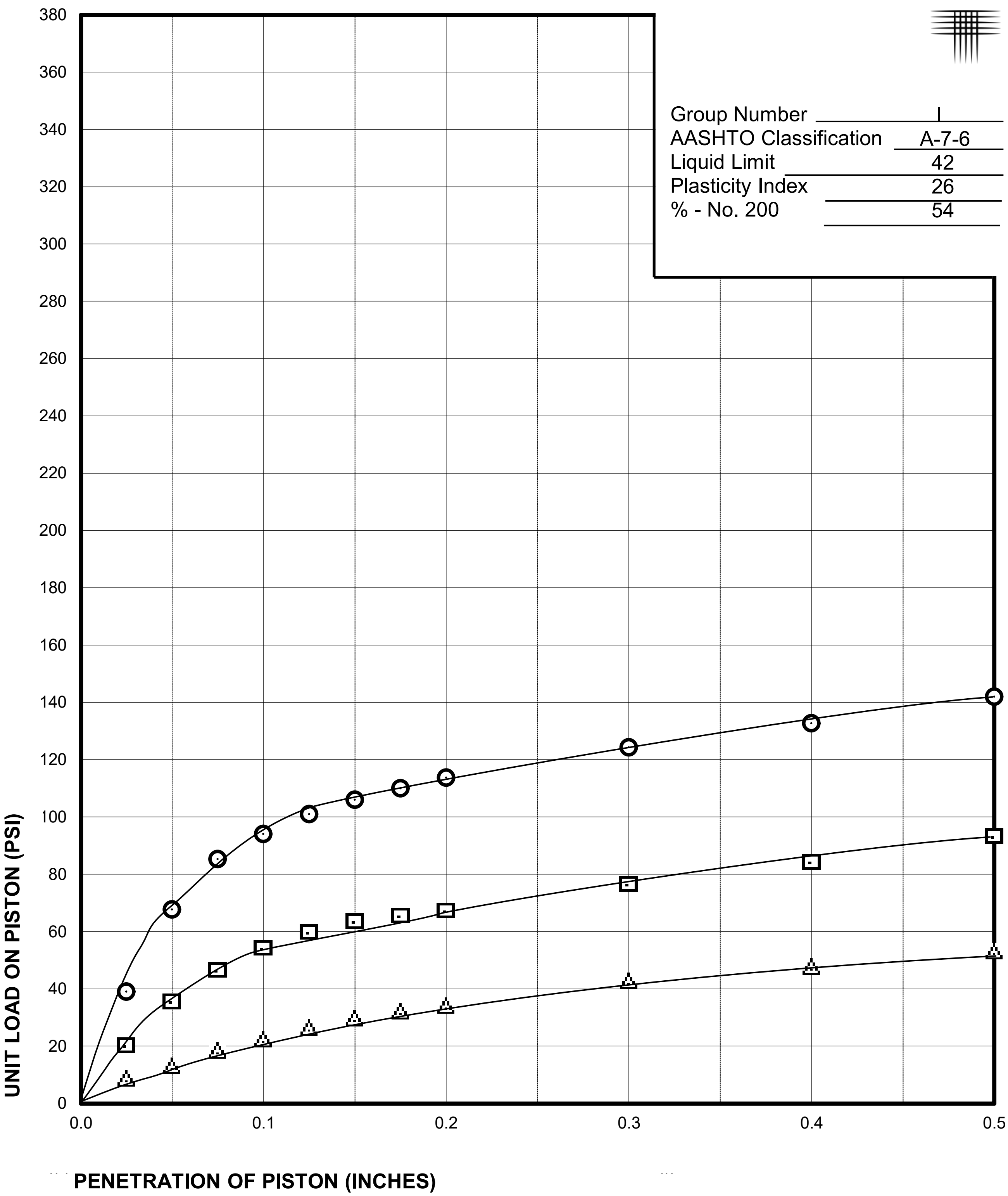
SAND %

SILT AND CLAY 54 %

Laboratory Proctor Compaction Test Results

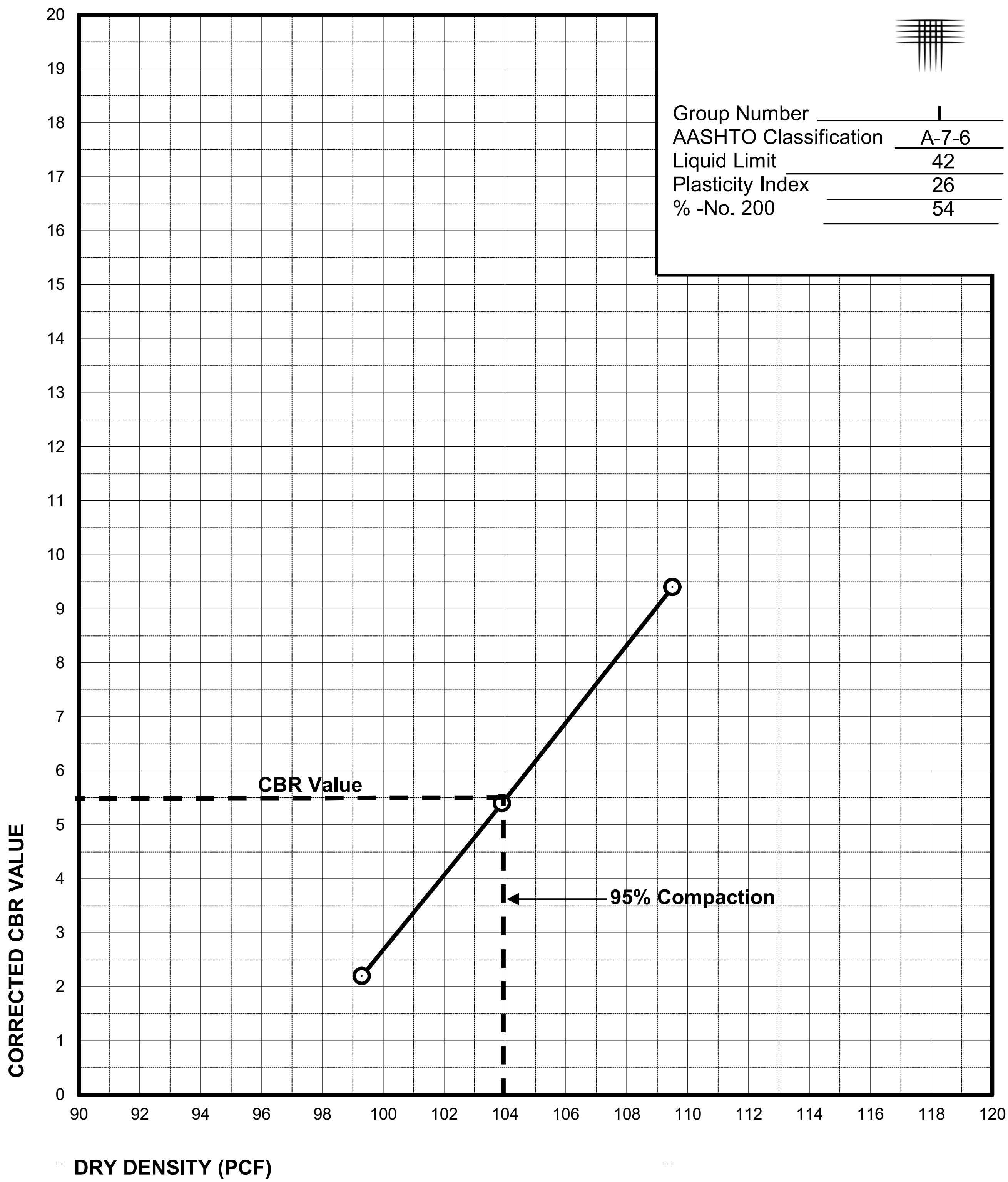
SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

FIG. B-65



Group No.	I		Description:	Clay, Sandy (A-7-6)		
Surcharge Load:	15 lbs.		Moisture Content:	15.5 %		
⊙ Compaction	100 %	Swell	0.8 %	CBR Value	9.4	
⊡ Compaction	94.9 %	Swell	0.9 %	CBR Value	5.4	
△ Compaction	90.7 %	Swell	1.5 %	CBR Value	2.2	

California Bearing Ratio



SYMBOL	GROUP NO.	95% COMPACTION	DESIGN CBR VALUE
⊙	I	104	5.5

California Bearing Ratio

SHEA PROPERTIES
HELIX INDUSTRIAL BUILDINGS C AND D
CTL|T PROJECT NO. DN51,883-125-R1

FIG. B-67

TABLE B - I

SUMMARY OF LABORATORY TEST RESULTS

BORING	DEPTH (ft)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	SWELL TEST DATA: SWELL (%)	SWELL TEST DATA: COMPRESSION (%)	SWELL TEST DATA: APPLIED PRESSURE (psf)	SWELL TEST DATA: SWELL PRESSURE (psf)	SOIL SUCTION VALUE (pF)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	SOLUBLE SULFATE CONTENT (%)	PASSING NO. 200 SIEVE (%)	SOIL TYPE
TH-1	4	19.0	100			500			58	27		64	CLAYSTONE
TH-1	9	18.0	90		0.9	1,100							SANDSTONE
TH-1	14	13.6	108	0.0		1,800							SANDSTONE
TH-2	4	5.8	114								0.07	14	SANDSTONE
TH-2	9	7.5	91									11	SANDSTONE
TH-2	14	17.2	107	0.0		1,800							SANDSTONE
TH-2	19	14.0	109		0.1	2,400							SANDSTONE
TH-3	4	3.1	88									6	SANDSTONE
TH-3	9	4.4	96		0.3	1,100		4.26					SANDSTONE
TH-3	14	20.0	95	0.5		1,800	3,200	4.00					SANDSTONE
TH-3	19	13.4	111	0.0		2,400		3.40					SANDSTONE
TH-3	24	15.2						4.69				91	CLAYSTONE
TH-3	29	16.7	112	0.5		3,600	6,700	4.26					SANDSTONE
TH-3	34	14.9	105		0.1	4,200		3.23					SANDSTONE
TH-3	39	16.2	102		0.3	4,900		3.27					SANDSTONE
TH-3	44	15.2	106		0.1	5,500		2.66					SANDSTONE
TH-4	4	25.9	90	1.8		500							CLAYSTONE
TH-4	9	14.3	99		0.6	1,100						57	CLAYSTONE
TH-4	14	11.7	104	0.0		1,800							SANDSTONE
TH-5	4	18.6	103	4.4		500							CLAYSTONE
TH-5	9	18.4	110	1.0		1,100							CLAYSTONE
TH-5	14	9.4	107		1.9	1,800							SANDSTONE
TH-6	4	13.1	108	8.3		500							CLAYSTONE
TH-6	9	13.7	116	0.4		1,100							CLAYSTONE
TH-6	14	10.4	103		1.7	1,800							SANDSTONE
TH-7	4	12.3	96	6.1		500							CLAY, SANDY (CL)
TH-7	9	17.3	94		1.6	1,100							SANDSTONE
TH-7	14	13.7	95		1.4	1,800							SANDSTONE
TH-7	19	18.8	108	1.1		2,400							CLAYSTONE
TH-8	4	13.3	106	1.9		500	2,100	4.58				79	CLAY, SANDY (CL)
TH-8	9	15.5	105	0.1		1,100	1,300	3.96					SANDSTONE
TH-8	14	7.6	104		1.3	1,800		4.65					SANDSTONE
TH-8	19	9.1	100		0.2	2,400		3.68					SANDSTONE
TH-8	24	22.5	100	0.9		3,000	6,600	3.86					CLAYSTONE
TH-8	29	20.8	102	0.3		3,600	4,600	4.57					CLAYSTONE
TH-8	34	19.3	102		0.3	4,200		3.89					CLAYSTONE
TH-8	39	18.8	109	0.9		4,900	11,800	4.85					CLAYSTONE
TH-8	44	16.8	116	0.3		5,500	8,000	4.06					CLAYSTONE
TH-9	4	17.5	101	0.7		500	1,400		44	28		74	CLAY, SANDY (CL)
TH-9	9	12.3	103		0.1	1,100							CLAY, SANDY (CL)
TH-9	14	7.5	110		0.6	1,800							SANDSTONE
TH-10	4	11.7	112	0.0		500							FILL, CLAY, SANDY
TH-10	9	14.1	104	0.1		1,100	1,200						FILL, CLAY, SANDY
TH-10	14	16.7	110	0.2		1,800	3,200						CLAY, SANDY (CL)
TH-10	19	13.3	111									25	SAND, CLAYEY (SC)
TH-11	4	19.1	100	0.7		500	1,200	3.36					FILL, CLAY, SANDY
TH-11	9	16.6	108	0.3		1,100	1,800	3.46					CLAY, SANDY (CL)
TH-11	14	16.5	113	0.5		1,800	3,900	3.91					CLAYSTONE
TH-11	19	16.7	112	0.1		2,400	2,900	3.66					CLAYSTONE
TH-11	24	15.1	117	0.0		3,000		3.10					CLAYSTONE

TABLE B - I

SUMMARY OF LABORATORY TEST RESULTS

BORING	DEPTH (ft)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	SWELL TEST DATA SWELL (%)	SWELL TEST DATA COMPRESSION (%)	SWELL TEST DATA APPLIED PRESSURE (psf)	SWELL TEST DATA SWELL PRESSURE (psf)	SOIL SUCTION VALUE (pF)	ATTERBERG LIMITS LIQUID LIMIT	ATTERBERG LIMITS PLASTICITY INDEX	SOLUBLE SULFATE CONTENT (%)	PASSING NO. 200 SIEVE (%)	SOIL TYPE
TH-11	29	19.3	97			3,600		2.75					SANDSTONE
TH-11	34	19.1	105		0.1	4,200		2.73					SANDSTONE
TH-11	39	16.4	101	0.0		4,900		2.84					SANDSTONE
TH-11	44	20.6	100		0.2	5,500		2.34					SANDSTONE
TH-12	4	21.1	105	1.6		500							FILL, CLAY, SANDY
TH-12	9	8.4	116									36	SAND, CLAYEY (SC)
TH-12	14	17.8	112	0.5		1,800							CLAYSTONE
TH-13	4	8.7	118									45	FILL, SAND, CLAYEY
TH-13	9	15.4	112	0.0		1,100							CLAY, SANDY (CL)
TH-13	14	11.6	112		0.3	1,800							SANDSTONE
S-1	1	5.9	111									6	SANDSTONE
S-1	9	12.5	117	0.9		200							SANDSTONE
S-2	2	13.0	107	7.3		200							CLAYSTONE
S-2	4	10.1	114									46	INTERBEDDED CLAYSTONE/SANDSTONE
S-2	7	11.8	112										SANDSTONE
S-3	1	21.2	105						50	33	<0.01	81	FILL, CLAY, SANDY
S-3	4	15.2	113	1.3		200							FILL, CLAY, SANDY
S-4	2	11.9	101	5.8		200							FILL, CLAY, SANDY
S-4	4	11.6	100						46	29		72	CLAY, SANDY (CL)
S-4	7	9.4	117						38	15		36	SANDSTONE
S-5	1	13.3	108						43	28		58	FILL, CLAY, SANDY
S-5	4	15.2	109	5.1		200							CLAY, SANDY (CL)
S-5	9	16.6	109						41	23		46	SANDSTONE
S-6	1	9.7	108	0.0		200					0.01	19	FILL, SAND, CLAYEY
S-6	4	19.2	106						47	29		63	FILL, CLAY, SANDY
S-7	1	18.7	89	1.8		200						50	CLAYSTONE
S-7	4	17.8	111	3.5		200							CLAYSTONE
S-8	1	7.3	112	3.3		200			38	22		42	INTERBEDDED CLAYSTONE/SANDSTONE
S-8	4	5.7	96									9	SANDSTONE
S-3,4,5	0-5								42	26		54	FILL, CLAY, SANDY

APPENDIX C

FLEXIBLE AND RIGID PAVEMENT MATERIALS,
CONSTRUCTION AND MAINTENANCE GUIDELINES

MATERIAL GUIDELINES FOR FLEXIBLE AND RIGID PAVEMENTS

Aggregate Base Course (ABC)

1. A Class 5 or 6 Colorado Department of Transportation (CDOT) specified aggregate base course should be used. A recycled concrete alternative which meets the Class 5 or 6 designation is also acceptable.

2. Aggregate base course should have a minimum Hveem stabilometer value of 78. Aggregate base course or recycled concrete material must be moisture stable. The change in R-value from 300 psi to 100 psi exudation pressure should be 12 points or less.

3. Aggregate base course or recycled concrete should be laid in thin lifts not to exceed 6 inches, moisture treated to within 2 percent of optimum moisture content, and compacted to at least 95 percent of maximum modified Proctor dry density (ASTM D 1557, AASHTO T 180). The material should be placed without segregation.

4. Placement and compaction of aggregate base course or recycled concrete should be observed and tested by a representative of our firm. Placement should not commence until the underlying subgrade is properly prepared and tested.

Hot-Mix Asphalt (HMA)

1. HMA should be composed of a mixture of aggregate, filler, hydrated lime and asphalt cement. Mixes shall be designed with 1 percent lime. Some mixes may require polymer modified asphalt cement, or make use of up to 20 percent reclaimed asphalt pavement (RAP). A project mix design is recommended and periodic checks on the project site should be made to verify compliance with specifications.

2. HMA should be relatively impermeable to moisture and should be designed with crushed aggregates that have a minimum of 80 percent of the aggregate retained on the No. 4 sieve with two mechanically fractured faces.

3. Gradations that approach the maximum density line (within 5 percent between the No. 4 and 50 sieves) should be avoided. A gradation with a nominal maximum size of 1 or 2 inches developed on the fine side of the maximum density line should be used.

4. Total void content, voids in the mineral aggregate (VMA) and voids filled should be considered in the selection of the optimum asphalt cement content. The optimum asphalt content should be selected at a total air void content of about 4 percent. The mixture should have a minimum VMA of 14 percent and between 65 percent and 80 percent of voids filled.

5. Asphalt cement should be PG 58-28 for local streets and PG 64-22 for collectors and arterials.

6. Hydrated lime should be added at the rate of 1 percent by dry weight of the aggregate and should be included in the amount passing the No. 200 sieve. Hydrated lime for aggregate pretreatment should conform to the requirements of ASTM C 207, Type N.

7. Paving should only be performed when subgrade temperatures are above 40˚F and air temperature is at least 40˚F and rising.

8. HMA should not be placed at a temperature lower than 245˚F for mixes containing PG 58-28 and PG 64-22 asphalt, and 290˚F for mixes containing polymer modified asphalt. The breakdown compaction should be completed before the mixture temperature drops 20˚F.

9. The maximum compacted lift should be 3 inches and joints should be staggered. No joints should be placed within wheel paths.

10. HMA should be compacted to between 92 and 96 percent of Maximum Theoretical Density. The surface shall be sealed with a finish roller before the mix cools to 185˚F.

11. Placement and compaction of HMA should be observed and tested by a representative of our firm. Placement should not commence until the subgrade is properly prepared, tested and proof-rolled.

<u>Portland Cement Concrete (PCC)</u>

1. Portland cement concrete should meet CDOT Class P concrete and have a minimum compressive strength of 4,500 psi at 28 days and a minimum modulus of rupture (flexural strength) of 600 psi. <u>A job mix design is recommended and periodic checks on the job site should be made to verify compliance with specifications</u>.

2. Portland cement should be Type II “low alkali” and should conform to ASTM C 150. Portland cement should conform to ASTM C 150.

3. Portland cement concrete should not be placed when the subgrade or air temperature is below 40ºF.

4. Free water should not be finished into the concrete surface. Atomizing nozzle pressure sprayers for applying finishing compounds are recommended whenever the concrete surface becomes difficult to finish.

5. Curing of the portland cement concrete should be accomplished by the use of a curing compound. The curing compound should be applied in accordance with manufacturer recommendations.

6. Curing procedures should be implemented, as necessary, to protect the pavement against moisture loss, rapid temperature change, freezing, and mechanical injury.

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

8. All joints should be properly sealed using a rod back-up and approved epoxy sealant.

9. Traffic should not be allowed on the pavement until it has properly cured and achieved at least 80 percent of the design strength, with saw joints already cut.

10. Placement of portland cement concrete should be observed and tested by a representative of our firm. Placement should not commence until the subgrade is properly prepared and tested.

FLEXIBLE PAVEMENT CONSTRUCTION GUIDELINES

Experience has shown that construction methods can significantly affect the life and serviceability of a pavement system. A site-specific mix design is recommended and periodic checks during the project should be made to verify compliance with specifications. We recommend the proposed pavement be constructed in the following manner:

1. The subgrade should be stripped of organic matter, scarified, moisture conditioned and compacted. Subgrade soils should be moisture conditioned to within 2 percent of optimum moisture content, and compacted to at least 95 percent of maximum modified Proctor dry density (ASTM D 1557).

2. Utility trenches and all subsequently placed fill should be moisture conditioned, compacted, and tested prior to paving. As a minimum, fill should be compacted to 95 percent of maximum standard Proctor dry density.

3. After final subgrade elevation has been reached and the subgrade compacted, the resulting subgrade should be checked for uniformity and all soft or yielding materials should be replaced prior to paving. Concrete should not be placed on soft, spongy, frozen, or otherwise unsuitable subgrade.

4. If areas of soft or wet subgrade are encountered, the material should be sub-excavated and replaced with properly compacted structural backfill. 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 thin, loose lifts no more than 6 inches, moisture treated to within 2 percent of optimum moisture content, and compacted to at least 95 percent of modified Proctor maximum dry density (ASTM D 1557).

6. Asphaltic concrete should be hot plant-mixed material compacted to between 92 and 96 percent of maximum Theoretical density. The temperature at laydown time should be at least 245°F. The surface shall be sealed with a finish roller prior to the mix cooling to 185°F.

7. The maximum compacted lift should be 3 inches and joints should be staggered. No joints should be within wheel paths.

8. Paving should only be performed when subgrade temperatures are above 40°F and air temperature is at least 40°F and rising.

9. Subgrade preparation and placement and compaction of all pavement material should be observed and tested. Compaction criteria should be met prior to the placement of the next paving lift. The additional requirements of the Douglas County should apply.

RIGID PAVEMENT CONSTRUCTION GUIDELINES

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. The subgrade should be stripped of organic matter, scarified, moisture conditioned and compacted. Subgrade soils should be moisture conditioned to within 2 percent of optimum moisture content and compacted to at least 95 percent of maximum modified Proctor dry density (ASTM D 1557).

2. After final subgrade elevation has been reached and the subgrade compacted, the resulting subgrade should be checked for uniformity and all soft or yielding materials should be replaced prior to paving. Concrete should not be placed on soft, spongy, frozen, or otherwise unsuitable subgrade.

3. The subgrade should be kept moist prior to paving.

4. 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.

5. Curing of the portland cement concrete should be accomplished by use of a curing compound in accordance with manufacturer recommendations.

6. 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.

7. Construction control and inspection should be performed during the subgrade preparation and paving procedures. Concrete should be carefully monitored for quality control. The additional requirements of the Douglas County should apply.

The design sections are based upon 10-year and 20-year periods. Experience in the Denver area indicates virtually no maintenance or overlays are necessary for a 20-year design period. We believe some maintenance and sealing of concrete joints will help pavement performance by helping to keep surface moisture from wetting and softening or heaving subgrade. To avoid problems associated with scaling and to continue the strength gain, we recommend deicing salts not be used for the first year after placement.

MAINTENANCE GUIDELINES FOR FLEXIBLE PAVEMENTS

A primary cause for deterioration of pavements is oxidative aging resulting in brittle pavements. Tire loads from traffic are necessary to "work" or knead the asphalt concrete to keep it flexible and rejuvenated. Preventive maintenance treatments will typically preserve the original or existing pavement by providing a protective seal or rejuvenating the asphalt binder to extend pavement life.

<u>Annual Preventive Maintenance</u>

- Visual pavement evaluations should be performed each year.
- Reports documenting the progress of distress should be kept current to provide information on effective times to apply preventive maintenance treatments.
- Crack sealing should be performed annually as new cracks appear.

<u>3 to 5-Year Preventive Maintenance</u>

- The owner should budget for a preventive treatment (e.g. chip seal, fog seal, slurry seal) at approximate intervals of 3 to 5 years to reduce oxidative embrittlement problems.

<u>5 to 10-Year Corrective Maintenance</u>

- Corrective maintenance (e.g. full-depth patching, milling and overlay) may be necessary, as dictated by the pavement condition, to correct rutting, cracking and structurally failed areas.

MAINTENANCE GUIDELINES FOR RIGID PAVEMENTS

High traffic volumes create pavement rutting and smooth, polished surfaces. Preventive maintenance treatments will typically preserve the original or existing pavement by providing a protective seal and improving skid resistance through a new wearing course.

Annual Preventive Maintenance

- Visual pavement evaluations should be performed each spring or fall.
- Reports documenting the progress of distress should be kept current to provide information of effective times to apply preventive maintenance.
- Crack sealing should be performed annually as new cracks appear.

4 to 8 Year Preventive Maintenance

- The owner should budget for a preventive treatment at approximate intervals of 4 to 8 years to reduce joint deterioration.
- Typical preventive maintenance for rigid pavements includes patching, crack sealing and joint cleaning and sealing.
- Where joint sealants are missing or distressed, resealing is mandatory.

15 to 20 Year Corrective Maintenance

- Corrective maintenance for rigid pavements includes patching and slab replacement to correct subgrade failures, edge damage and material failure.
- Asphalt concrete overlays may be required at 15 to 20-year intervals to improve the structural capacity of the pavement.