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GEOTECHNICAL INVESTIGATION

CANYONSIDE – SINGLE AND MULTI-FAMILY DEVELOPMENT
EAST OF CANYON FORGE DRIVE AND CANYONSIDE BOULEVARD
CASTLE PINES, COLORADO

Prepared for:

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SCOPE

This report presents the results of our Geotechnical Investigation for the Canyonside Single and Multi-Family Development located in Castle Pines, Colorado (Fig. 1). The purpose of our investigation was to evaluate the site geology and subsurface conditions to provide geotechnical design and construction recommendations and criteria for the project. The scope was described in a Service Agreement (DN 23-0010) dated January 16, 2023. A Subgrade Investigation and Pavement Design for Sweet Creek Lane is not included in this report.

This report is based on subsurface conditions found in our exploratory borings, results of field and laboratory tests, published geologic maps, previous investigation, engineering analysis of field and laboratory data, and our experience. The report contains our opinions and recommendations for design criteria and construction details for site development, foundations and floor systems, slabs-on-grade, surface and subsurface drainage precautions, concrete, and pavements. Other types of construction not described herein may require revision of this report and the recommended design criteria. A brief summary of our conclusions and recommendations follows, with more detailed discussion in the report.

SUMMARY

1. Strata encountered in our exploratory borings consisted of nil to about 23 feet of native clay and sand underlain in 86 borings by weathered and comparatively unweathered claystone and sandstone bedrock to the maximum explored depth of 50 feet. About 4½ and 7 feet of existing fill was encountered in two borings. The clay and claystone are expansive. The sand is low swelling or non-expansive and the sandstone is judged to be non-expansive. Soil and bedrock conditions were highly variable across the site.
2. Groundwater was encountered in 26 borings at depths of about 12 to 36 feet below proposed grade. Groundwater may be encountered during sub-excavation and deep utility installation in some areas of the site. Groundwater may develop and rise after construction in response to development, precipitation, landscaping irrigation, flow in Newlin Gulch, and changes in land use.
3. The primary geologic hazards identified at this site are expansive soil and bedrock, undocumented fill, and steep slopes and erosion. There is risk that slabs-on-grade and foundations will heave or settle and be damaged. To mitigate the risk, sub-excavation can be performed as described in this report. Fill depth below foundations should be uniform below each building foundation. We believe



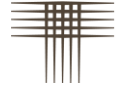
the recommendations presented in this report will help to reduce risk of damage; they will not eliminate that risk.

4. Due to the subsurface variability, a conservative approach to mitigating the expansive soil and bedrock is warranted. We recommend sub-excavation ranging from 5 to 10 feet below lowest foundation elements on a majority of the site (Fig. 2). Pavements will require mitigation to a depth of 3 feet.
5. The grading plans and data indicate post-tensioned slab-on-grade foundations (PTS) or footings designed to maintain minimum deadload can be used for all buildings provided uniform depth of fill and recommended sub-excavation is performed. Design criteria and recommendations are presented in this report. The soil and bedrock at Canyons are variable. If claystone is exposed in footing or PTS excavations where sub-excavation is not performed, further evaluation is recommended.
6. We recommend structurally supported floors for all finished areas unless PTS are used.
7. Access drives should be paved with at least 7 inches of asphalt, 5 inches of asphalt over 8 inches base course, or 6 inches of concrete. Thinner pavement sections can be considered for automobile parking areas.
8. Control of surface drainage will be critical to the performance of foundations. Overall surface drainage should be designed to provide rapid run-off of surface water away from the proposed structures and off pavements and flatwork. Water should not be allowed to pond near the crests of slopes, near structures, or on pavements and flatwork.

SITE CONDITIONS

The Canyonside Single and Multi-Family Development contains roughly 28 acres and is located east of Canyon Forge Drive and Canyonside Boulevard in Castle Pines, Colorado (Fig. 1 and Photo 1). Cross Canyon Trail bisects the site, separating the multi-family development on the south from the single and multi-family developments to the north. The surrounding area is under development. The ground cover consists of grass, weeds, and bushes.

Based on review of historical aerial images, the site has been vacant dating back to at least 1937 (Photo 2) and has remained largely unchanged until construction began on Hess Road in 2004. During the period of the Hess Road construction, aerial images indicate that cuts were made on the western portion of the single-family development. Cross Canyon Trail was constructed around 2018 and involved fill placement along the site boundary with the road. The southern portion of the site has been used as a staging area for the adjacent development.



Newlin Gulch runs along the southeast border of the site, for an approximate distance of 150 feet. The ground generally slopes to the southeast with elevations ranging from about 6300 to 6230.



Photo 1: Aerial Image, Google Earth©, September 2022.

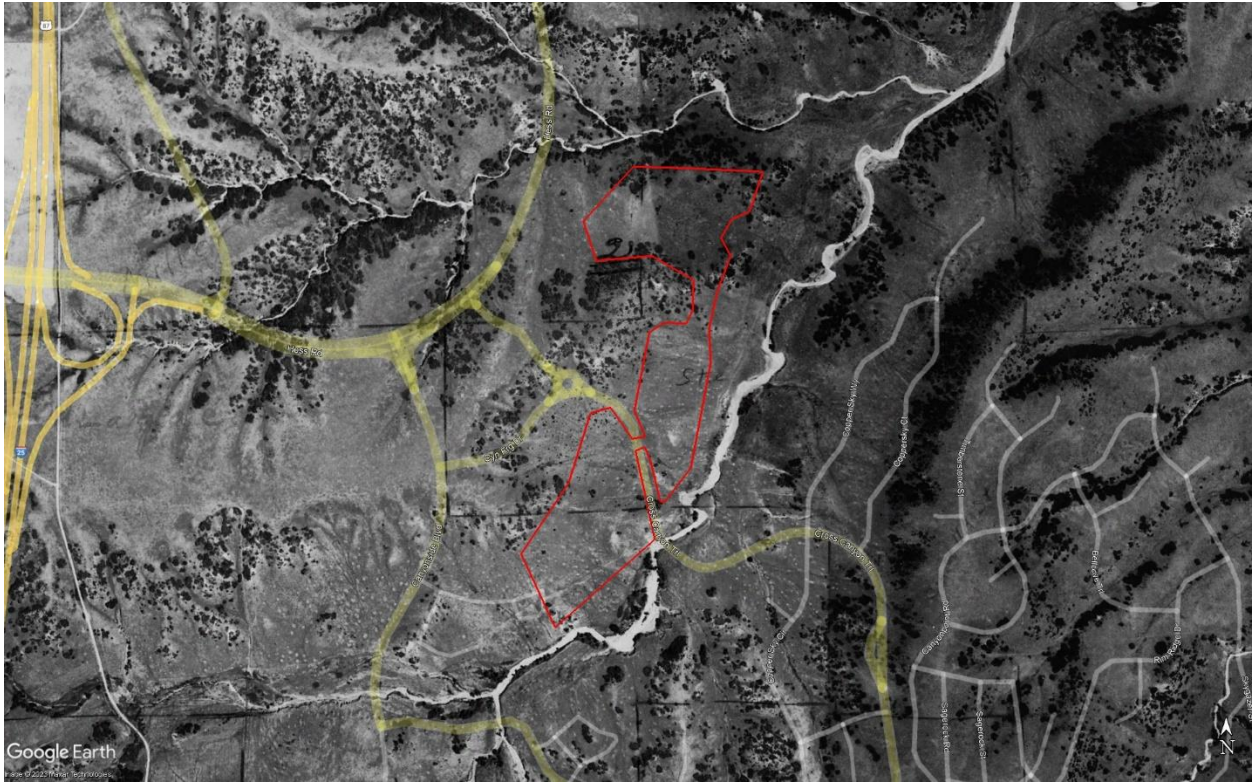


Photo 2: Aerial Image, Google Earth©, 1937.

PROPOSED DEVELOPMENT

Draft construction documents prepared by Kimley Horn (dated December 20, 2022) indicate the site is being developed for a mix of single-family and multi-family residences. We anticipate the single-family residences will be one to two-story, wood-framed structures and the multi-family buildings will be one to three-story, wood-framed structures. The single-family residences will have attached garages and some of the multi-family buildings will have tuck-under garages. Some of the structures will have lower level walk-outs. The residences will be a for-rent product. The homes/buildings will be served by paved access drives, buried utilities, and associated infrastructure. A clubhouse and pool are planned on the southern portion of the site. Three detention ponds will be located along the eastern edge of the development. Cuts up to about 27 feet and fills of up to about 21 feet will be required to achieve proposed grades.



PREVIOUS INVESTIGATION

We performed a Geologic and Preliminary Geotechnical Investigation for The Canyons, Planning Areas 1, 2, and 3 and presented results in a report dated August 16, 2018 (Project No. DN49,582-115-R1). About four of our widely-spaced borings were located within the subject site's boundary. We identified expansive soil and bedrock as the primary geologic and geotechnical concerns. Some of the bedrock was very hard.

Intertek PSI published a "Report of Geotechnical Engineering Evaluation" dated November 8, 2022, that included the subject site as well. They encountered variable subsurface conditions including overburden sand and clay underlain by claystone and sandstone bedrock. They performed limited swell testing and swell tests results suggest moderate to very high swelling soils are present on-site. Pertinent data from the previous investigations was considered during this investigation.

INVESTIGATION

Subsurface conditions were investigated between February 18 and March 8, 2023, by drilling and sampling 94 exploratory borings at the approximate locations shown on Fig. 1. Prior to drilling, we contacted the Utility Notification Center of Colorado and local sewer and water districts to clear boring locations for conflicts with buried utilities. Boring locations were staked and surveyed by R&R Engineers-Surveyors, Inc.

The borings were drilled to depths of 5 to 50 feet using 4-inch diameter, continuous-flight auger powered by buggy-, track-, or truck-mounted drill rigs. The depths were selected to extend about 25 feet below proposed grades. We obtained samples at approximate 2- to 5-foot intervals using 2.5-inch diameter (O.D.) modified California barrel samplers driven by blows of a 140-pound hammer falling 30 inches. Practical auger refusal occurred in TH-18 at a depth of 12 feet. Bulk specimens were collected from auger cuttings within specified-depth intervals from the shallow, pavement borings. Our field representatives observed drilling, logged the strata encountered in the borings, and obtained samples. Graphical logs of the borings, including results of field penetration resistance tests and some laboratory test data, are presented in Appendix A.



Samples obtained during drilling were returned to our laboratory where they were examined and assigned testing. Laboratory testing included moisture content and dry density, swell-consolidation, total suction, Atterberg limits, percent silt- and clay-sized particles (passing No. 200 sieve), standard Proctor compaction, and water-soluble sulfate concentrations. Swell tests were performed by wetting samples under approximate overburden pressures after proposed grading (i.e., the pressure exerted by the overlying soil and bedrock after site grading). Load-back analysis was performed on select samples to help estimate swelling pressures. Results of the laboratory tests are summarized in Appendix B.

SUBSURFACE CONDITIONS

Strata encountered in our exploratory borings consisted of nil to about 23 feet of native clay and sand, often interlayered, underlain by weathered and comparatively unweathered clay-stone and sandstone bedrock to the maximum explored depth of 50 feet. Bedrock was encountered in 86 borings. About 4½ to 7 feet of existing fill, considered to be undocumented, was encountered in TH-57 and TH-58. Soil and bedrock conditions were highly variable across the site. Pertinent engineering characteristics of the soil and bedrock are presented in the following paragraphs.

Undocumented/Existing Fill

We encountered about 4½ to 7 feet of existing fill in TH-57 and TH-58, respectively. This fill is likely related to grading for Cross Canyon Trail. Records of fill placement were not available for our review and thus the fill is considered undocumented. The fill was comprised of clayey sand and was very dense based on the result of a field penetration resistance test. The fill was difficult to discern from the native soils as they are similar in composition, and more or less fill may be present than our borings imply. Although not encountered, existing fill may also be present in the southern most portion of the site that was used as a staging area for the adjacent development. Our estimate of the extent of existing fill is shown on Fig. 1. One sample contained 29 percent silt- and clay-sized particles (passing the No. 200 sieve) and exhibited moderate plasticity. Unless documentation of fill placement can be provided, we judge the fill to be unsuitable to support proposed construction and should be removed and reworked to the criteria outlined in **Site Grading**.



Native Clay and Sand

Native soils were encountered in 93 borings and consisted of sandy to very sandy clay and silty to clayey sand, often interlayered, with occasional gravel. The clay was stiff to very stiff and the sand was medium dense to very dense based on results of field penetration resistance tests. Two clay samples compressed while 19 swelled 0.5 to 10.0 percent when wetted. The average swell of clay samples was 3.9 percent. Two clay samples had total suction values of 3.97 and 4.79 pF and developed load-back swell pressures of approximately 2,000 to 5,600 psf. Nine sand samples compressed and one swelled 0.5 percent when wetted. Three sand samples had total suction values of 4.10 to 4.93 pF. Five clay samples contained 52 to 68 percent silt- and clay-sized particles, with two exhibiting moderate plasticity. Twelve sand samples contained 12 to 49 percent silt- and clay-sized particles. Testing indicates the sand is generally non-expansive and the clay is expansive with high swell potential. The location, depth and expansion/compression characteristics of the native soils were found to be highly variable.

Bedrock

Weathered and comparatively unweathered claystone and sandstone bedrock was encountered below the native soils or fill in 86 borings at depths of 1 to 23 feet below existing grades. The unweathered bedrock was medium hard to very hard. Very hard sandstone in TH-18 resulted in practical auger refusal. Twelve weathered zones were identified, ranging in thickness from 3½ to 6 feet.

Seven claystone samples compressed slightly while 71 samples swelled 0.2 to 10.6 percent when wetted. The average swell of claystone samples was 2.4 percent. One sandstone sample compressed slightly while two samples swelled 0.1 and 0.4 percent. The comparatively high measured compression of bedrock samples is likely due to sample disturbance during sampling and/or laboratory preparation and not indicative of compressible soils. Three claystone samples had total suction values of 4.21 to 4.44 pF, with two developing load-back swell pressures of approximately 8,900 and 10,200 psf. One sandstone sample had a total suction value of 3.98 pF. Eight claystone samples contained 50 to 92 percent silt- and clay-sized particles, six of which exhibited moderate to high plasticity. Ninety sandstone samples contained 10 to 48 percent silt- and clay-sized particles, with seven exhibiting low to high plasticity.



The location and depth of the claystone and sandstone bedrock were highly variable. Claystone layers were often located within sandstone with no obvious pattern or consistency. Testing indicates the claystone is expansive with high swell potential and we judge the sandstone to be generally non-expansive with lenses of low swell potential.

Groundwater

Groundwater was encountered at a depth of about 8½ and 44 feet in TH-69 and TH-79, respectively, during drilling. When holes were checked after drilling on April 6, 2023, groundwater was measured in 26 borings at depths ranging from about 13 to 45 below existing grade, or about 12 to 37½ feet below proposed grade. TH-69 did not contain groundwater when checked after drilling, indicating the groundwater level recorded during drilling was the result of a confined pocket of groundwater and is not representative of actual groundwater conditions. The remaining borings were dry or were buried/destroyed and delayed groundwater measurements could not be obtained.

Groundwater may be encountered during deep utility installation and sub-excavation. A typical sub-excavation interceptor drain detail is presented on Fig. 20. We do not anticipate groundwater will impact proposed construction otherwise. Groundwater may develop and rise after construction in response to development, precipitation, landscaping irrigation, flow in Newlin Gulch, 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 and southern 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. According to the International Building Code (Section 16.13.2.2) and based upon the results



of our investigation and experience in the region, we judge the site classifies as Seismic Site Class C.

GEOLOGY

To evaluate the geology of the site, we used information provided by the borings drilled during this and previous investigation and review of both the Geologic Map of the Castle Rock North Quadrangle, Douglas County, Colorado by CGS (Open-File Report OF05-02 by Thorson, J.P., dated 2005) and the Surficial Geologic Map of the Denver 1 degree X 2 degree Quadrangle, Colorado by USGS (Miscellaneous Field Studies Map MF-2347 by Moore, D.W., Straub, A.W., Berry, M.E., Baker, M.L., and Brandt, T.R., dated 2001). An excerpt from the 2005 geologic map with the site outline is reproduced below, Photo 3. The limits of the 2001 geologic map are too wide to accurately display site geology in an excerpt.

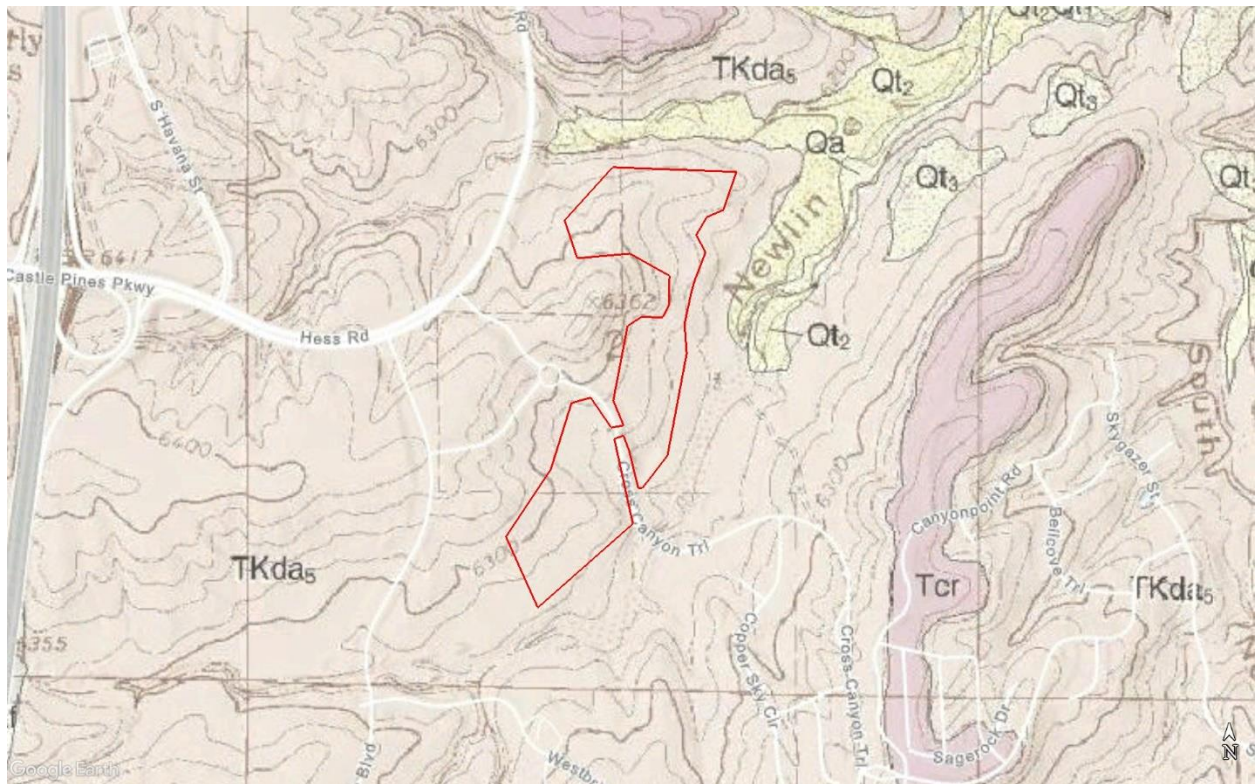


Photo 3: Geologic Map of the Castle Rock North Quadrangle, Douglas County, Colorado by CGS, 2005



As described in the geologic maps, and supported by material encountered in our exploratory borings, the site is covered by surficial soils of loamy colluvium and sheetwash alluvium deposits (map symbol cac, 2001 geologic map) of Holocene age. The underlying bedrock outcrops portions of the site and is sandstone and claystone interbedding of facies unit five of the Dawson formation (map symbol TKda₅) of early-middle Eocene age. The sandstone is arkosic and can be cemented in areas. The claystone is expansive, silty at times, and plastic when wet.

GEOLOGIC HAZARDS

Geologic hazards and geotechnical concerns can affect development risks and costs. Geologic hazards and geotechnical concerns identified through exploration, document resources, and experience include:

1. Undocumented/existing fill;
2. Expansive soil and bedrock; and
3. Steep slopes and erosion.

The hazards and concerns identified will not preclude construction and development as currently planned.

Expansive Soil and Bedrock

We conducted swell/consolidation tests to provide a basis to calculate potential heave of the on-site materials. The analysis involves dividing the soil profile into layers and modeling the heave of each layer from representative swell tests. A depth of wetting of 24 feet below the proposed ground surface was used for the heave evaluation. Research by Walsh, Colby, Houston and Houston¹ indicates there is a 90 percent probability that the wetting depth will not exceed 24 feet for this region, suggesting the risk of ground heave exceeding the estimated values is low. This depth of wetting is typically used for irrigated residential sites with basements. If no basements are incorporated in the structure, the likelihood of heave exceeding these estimates is low.

¹"Method for Evaluation of Depth of Wetting in Residential Areas" by Walsh, Colby, Houston and Houston, Journal of Geotechnical and Geoenvironmental Engineering, ASCE February 2009.



We chose two methods to evaluate potential ground heave, the Thompson Method and the partial-wetting technique by Houston et al.² The latter study theorizes that the highest degree of wetting occurs near-surface with a gradually decreasing degree of wetting with depth. The Thompson method does not account for partial wetting and assigns a constant 30 percent reduced wetting factor to each layer. This typically results in higher heave estimates at greater depths. Houston, Stauffer, West, Bradford, and Houston's 2017 publication indicates that about 80 to 90 percent of the laboratory measured swell actually occurs in the field in the upper 30 to 40 percent of the depth of wetting, decreasing parabolically from that point to the maximum depth of wetting, i.e., about 50 percent of the laboratory measured swell occurs in the field at 70 percent of the depth of wetting and 10 percent occurs at 90 percent of the depth of wetting. We estimated potential heave by averaging the Thompson method and partial-wetting technique, along with using engineering judgement, to tabulate Table B-II.

We estimate potential ground heave at proposed grades may range from less than ½-inch to 6½ inches. Calculations assumed a swell of 0.5 percent for site grading fill. These estimates do not consider potential compression of the soil and represent a range of potential heave. Excessive wetting could lead to more heave or settlement. Engineered design of grading, pavements, foundations, slabs-on-grade, and surface drainage can mitigate, but not eliminate, the effects of expansive soil and bedrock. Sub-excavation, as recommended in this report, is an alternative to reduce potential heave and allow use of shallow foundations. Additional borings are recommended after the completion of sub-excavation to confirm the expansive material has been properly mitigated and the resulting fill is low swelling.

Colorado is a challenging location to practice geotechnical engineering. The climate is relatively dry, and the near-surface soils are typically dry and relatively stiff. These soils and related sedimentary bedrock formations react to changes in moisture conditions. Some of the soils and bedrock swell as they increase in moisture and are referred to as expansive soils. Other soils can compress significantly upon wetting and are called compressible or collapsible soils. Much of the land available for development east of the Front Range is underlain by expansive clay or claystone bedrock near the surface. The soils that exhibit compressible behavior are

²"Use of the Net Partial Wetting Factor (NPWF Method of Computation of Remaining Heave: A Forensic Study" by Houston, Stauffer, West, Bradford, and Houston, 2017.



more likely west of the Continental Divide; however, both types of soils occur throughout the state.

Covering the ground with buildings, streets, driveways, patios, etc., coupled with irrigation and changing drainage patterns, leads to an increase in subsurface moisture conditions. As a result, 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. After construction, the homeowner's association (HOA) or property manager must assume responsibility for maintaining the structures and use appropriate practices regarding drainage, landscaping, and irrigation.

Bedrock encountered in this investigation and in other investigations in the Canyons has been highly variable. Expansive claystone, non-expansive sandstone and variably expansive interbedded claystone and sandstone bedrock have been encountered and can occur within the same building footprint. Excavations may expose expansive bedrock that may not be anticipated and merit additional remediation.

We believe potential hazards can be mitigated with proper engineering, design, and construction practices as discussed in this report. We believe the recommendations in this report will help reduce risk of damage; they will not eliminate the risk. The builder and owner should understand that slabs-on-grade, pavements and, (in some instances) foundations may be affected by movement of the subsoils. Maintenance will be required to reduce risk. We recommend you provide property managers a booklet that describes swelling soils and includes recommendations for care and maintenance. Colorado Geological Survey Special Publication 43³ was designed to provide this information.

Undocumented/Existing Fill

During our investigation we encountered existing, undocumented fill in TH-57 and TH-58. Based on review of historical aerial imagery we believe this fill to be related to the construction of Cross Canyon Trail. Existing, undocumented fill may also be present in the southern por-

³"A Guide to Swelling Soils for Colorado Homebuyers and Homeowners," Second Edition Revised and Updated by David C. Noe, Colorado Geological Survey, Department of Natural Resources, Denver, Colorado, 2007.



tion of the site, where the adjacent development encroached onto the subject site (Fig. 1). Unless documentation of fill placement can be provided, we judge the fill to be unsuitable to support proposed construction. All undocumented fill should be removed and reworked to the criteria outlined in **Site Grading**. The fill can be reused, provided it is free of deleterious material.

Steep Slopes and Erosion

Existing slopes appear to be stable. Slopes will require erosion control during and after construction. The granular soils are considered highly erodible. Soil cut and fill slopes no steeper than 3H:1V (horizontal:vertical) should be stable. Slopes of 4H:1V are preferable. Re-vegetation or other erosion control measures should be employed to control erosion. Slope stability analysis was not part of our scope.

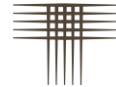
Water is expected to flow onto the site from the northwest. During peak precipitation events, some accumulation of surface sheet flow in gullies and drainages is expected. Development will increase the amount of impervious surfaces, which can lead to drainage problems and erosion if surface water flow is not adequately designed. Surface drainage design and evaluation of flood potential should be performed by a Civil Engineer as part of the project design.

SITE DEVELOPMENT

The primary geotechnical concerns that we believe will influence development is the presence of undocumented/existing fill, expansive soil and bedrock, steep slopes, and erosion. These concerns can be mitigated, but not eliminated, with proper planning, engineering, design and construction. We believe there are no geologic or geotechnical constraints that would preclude development. The following sections provide site development recommendations.

Excavation

We believe most of the soils penetrated by our exploratory borings can be excavated with typical heavy-duty equipment. Very hard bedrock was encountered across our borings; practical drill auger refusal was encountered at a depth of 12 feet in TH-18. There may be areas



with very hard or cemented bedrock that could require use of rock excavation techniques such as heavy ripping, rock saws, blasting, and/or pneumatic hammers.

We recommend the owner and contractor become familiar with applicable local, state and federal safety regulations, including the current Occupational Safety and Health Administration (OSHA) Excavation and Trench Safety Standards. Based on our investigation and OSHA standards, we anticipate the clay and bedrock may classify as Type B soil and the sand as Type C. Based on OSHA regulations, Type B soils require maximum slope inclinations of 1H:1V (horizontal: vertical) and 1½H:1V for Type C soils for temporary excavations in dry conditions. Seepage and saturated soils will necessitate flatter conditions. The contractor's "competent person" is required to identify the soils encountered in the excavations and refer to OSHA standards to determine appropriate slopes. 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.

Fill and Backfill

Prior to fill placement, the ground surface should be stripped of vegetation, scarified, and moisture conditioned to between 1 and 4 percent above optimum moisture content for clay and within 2 percent of optimum for sand, and compacted to at least 95 percent of standard Proctor maximum dry density (ASTM D 698). Fill should be moisture conditioned and compacted in accordance with criteria shown in Table I.

TABLE I
SUMMARY OF COMPACTION AND MOISTURE CONTENT SPECIFICATIONS

Soil Type	Depth of Site Grading Fill	
	≤20 Feet	>20 Feet
Clay	95% STD, 1 to 3 percent above optimum	98% STD, 2 percent below to 1 percent above optimum
Sand and Gravel	95% STD, -2 to +2 percent from optimum	98% STD, 2 percent below to 1 percent above optimum

*Compaction and moisture content percentage specifications based on standard Proctor maximum dry density (STD, ASTM D 698) and optimum moisture content (optimum).



The properties of fill will affect the performance of foundations, slabs-on-grade, utilities, pavements, flatwork, and other improvements. The on-site soils are suitable for use as site grading fill, provided they are substantially free of debris, organics, and other deleterious materials. Rock particles and boulders larger than 6 inches and soil clods or claystone pieces larger than 3 inches should be removed from the fill or be broken down. Fill should be placed in thin loose lifts, moisture conditioned and compacted prior to placement of the next lift using the criteria presented in Table I. The placement and compaction of site grading fill should be observed, and density tested by our representative during construction. Guideline grading specifications are presented in Appendix C. If imported soil is necessary, it should ideally consist of soil having maximum particle sizes of 2 inches, between 20 and 40 percent passing the No. 200 sieve, a liquid limit less than 30, and a plasticity index less than 15. Potential import fill materials should be submitted to our office for approval prior to importing to the site.

Our experience indicates fill will settle under its own weight. We estimate potential settlement of about 1 to 2 percent of the fill thickness even if the fill is compacted to the specified criteria. Most of this settlement usually occurs during and soon after construction; for clayey fill, it may continue for longer. Heave or additional settlement may occur after development in response to wetting.

If fill will be placed on slopes of 20 percent or steeper the slopes should be benched prior to placing fill (Fig. 4). There may be some areas where proposed grading will create non-uniform depths of fill below residence sites. Where the depth varies more than about 5 feet, sub-excavation or benching of existing slopes should be considered to create more uniform fill depth. Non-uniform fill depths below structures increases risk of differential heave or settlement.

Sub-Excavation

Deep foundations and structurally supported floor systems are commonly used on sites where moderate to high swelling clay or claystone bedrock are found at depths likely to influence performance. Many local builder-developers choose to perform sub-excavation to reduce potential heave and provide a relatively uniform fill layer that is likely suitable for shallow foundations and slab-on-grade floors. Sub-excavation has been employed in the Denver area with satisfactory performance for the large majority of the sites where this ground modification method



has been completed. We have seen isolated instances where differential settlement of sub-excavation fill has led to damage to buildings supported on footings. In most cases, the settlement was caused by wetting associated with poor surface drainage or seepage, and/or poorly compacted fill placed at the horizontal limits of excavation. Wetting of the fill may cause softening and settlement. Interceptor drains (Fig. 20) may be needed if sub-excavation is attempted below measured groundwater levels.

The variability of soil and bedrock materials implies that a conservative sub-excavation approach is merited. We recommend sub-excavation to a depth of 5 to 10 feet below lowest foundation element for portions of the site (Fig. 2). The bottom of sub-excavation areas should extend laterally at least 5 feet outside the largest possible foundation footprint to ensure foundations are constructed over moisture-conditioned fill. The sub-excavation areas should be checked by a surveyor. An “as-built” sub-excavation plan should be prepared and provided to construction and sales staff to ensure foundations and decks are located within the sub-excavated area. Conceptual sub-excavation profiles are shown on Figs. 5 through 7.

The excavation contractor should be chosen carefully to assure they have experience with fill placement at over-optimum moisture and have the necessary compaction equipment. In order for the sub-excavation procedure to be performed properly, close control of fill placement to specifications is required. Special precautions should be taken for compaction of fill at corners, access ramps, and edges of the sub-excavation cuts due to equipment access constraints. The contractor should have the appropriate equipment to reach and compact these areas. Fill should be moisture treated between 1 and 4 percent above optimum moisture content and be compacted to at least 95 percent of maximum standard proctor dry density (ASTM D 698). Guideline specifications are provided in Appendix D. Our representative should observe and test compaction of the fill. The fill should be tested after the fill placement.

If the fill dries excessively prior to construction, it may be necessary to rework the upper drier materials prior to constructing foundations. We judge the fill should retain adequate moisture for about two years. Moisture conditions can be assessed in excavations as construction progresses. Additional borings are recommended after the completion of sub-excavation to confirm the expansive material has been properly mitigated and the resulting fill is low swelling.



Utility Construction

Water and sewer lines are often constructed beneath slabs and pavements. Compaction of trench backfill can have a significant effect on the life and serviceability of slabs and pavements. Compaction should be performed to the appropriate jurisdictional specifications. Trench backfill should be placed in thin (8 inches or less) loose lifts. The placement and compaction of utility trench backfill should be observed and tested by a representative of our firm during construction.

Utility trench backfill compaction by a sheepfoot wheel attachment on a backhoe or trackhoe has been found to be generally less successful than using self-propelled roller compactors. The trenches should be widened to allow the use of a self-propelled compactor. Special attention should be paid to backfill placed adjacent to manholes as we have seen instances where settlement in excess of 2 percent has occurred. Any improvements placed over backfill should be designed to accommodate movement.

Slopes

We recommend permanent cut and fill slopes be designed with a maximum slope of 3H:1V (horizontal to vertical); use of 4H:1V would be better to control erosion. If site constraints (property boundaries and streets) do not permit construction with recommended slopes, we should be contacted to evaluate the subsurface soils and steeper slopes. Slopes higher than 20 feet should be evaluated by our office on a case-by-case basis. Concentrated surface drainage should not be allowed to sheet flow across slopes or pond near the crest of slopes. All slopes should be re-vegetated soon after grading to reduce erosion.

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.



Interceptor Drains

Sub-excavation interceptor drains may be required if groundwater is encountered during sub-excavation. Interceptor drains should consist of Marafi G200N drain board with two-sided drainage capacity connected to 4-inch diameter perforated PVC pipe or approved equivalent. The drain board should extend vertically at least 3 feet above groundwater. A typical detail is provided on Fig. 20. Drains should slope to a gravity outlet or a wet well where water can be removed with a pump.

Underdrain

With long term development and subsequent irrigation, groundwater may develop and rise. Our firm typically advocates an underdrain system below sanitary sewer mains and services to control groundwater that may accumulate in response to development and provide a gravity outlet for foundation drains. If a gravity outfall for the underdrain system is not possible, an alternative would be to outfall underdrains to a wet well where water can be removed with a pump; maintenance should be expected with this option. It may not be practical to install underdrains at this site if a gravity discharge is not available. If an underdrain system is not installed, individual house/building foundation drains would discharge into sumps with pumps. Sump discharge can result in ponding and recycling if slopes between lots/buildings are not adequately graded and well-drained. Problems with chronic ice or algae formation on sidewalks have also developed from sump discharge.

The underdrain should consist of $\frac{3}{4}$ to 1½-inch clean, free-draining gravel surrounding rigid PVC pipe (Fig. 8). The pipe should be sized for anticipated flow by the civil engineer and may consist of 4 or 6-inch lines. The PVC pipe should be placed at a grade of at least 0.5 percent. A concrete cutoff should be constructed around the sewer pipe and underdrain pipe immediately downstream of the point where the underdrain pipe exits the sewer trench and transitions to the outlet (Fig. 9). The underdrains should be designed to discharge to a gravity outfall and be provided with a permanent concrete headwall and trash rack. If the underdrain discharges into a detention pond, the risk of flood water backflow through the underdrain into basements or below-grade areas should be evaluated. A check valve or backflow preventer can be consid-



ered. Where feasible, the underdrain services should be installed deep enough so that the lowest point of the foundation drain can be connected to the underdrain service as a gravity outlet (Fig. 10). Underdrain services can be 3-inch to avoid confusion with the 4 or 6-inch main line.

FOUNDATIONS

Our investigation indicates expansive clay and claystone are present at depths likely to influence the performance of shallow foundations and slabs-on-grade. The expansive clay and claystone materials should be mitigated as discussed in **Sub-Excavation**. Alternatively, a deep foundation system such as drilled piers can be considered. Footing or post-tensioned slab-on-grade foundation may be used for buildings where non-expansive and low swelling site-grading fill, native soil and bedrock are present, or after sub-excavation is performed (Fig. 2). The soil and bedrock at Canyons are variable. If claystone is exposed in footing or PTS excavations where sub-excavation is not performed, further evaluation is recommended. Design criteria for footings designed to maintain minimum deadload and PTS foundations are presented below. The criteria presented were developed from analysis of field and laboratory data and our experience.

Footings With Minimum Deadload

1. Footings should be constructed on moisture conditioned, compacted fill or native, undisturbed sandy soil or sandstone bedrock. Where soils are loosened during excavation or in the forming process, or if any loose, soft, or dry soils are exposed in excavations, the soils should be removed and re-compacted 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 minimum deadload pressure of 1,000 psf. The maximum pressure can be calculated using dead load plus $\frac{1}{2}$ -live load.
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.
4. Footings should have a minimum width of 16 inches. Foundations for isolated columns should have minimum dimensions of 18 inches by 18 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 large openings and 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.
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 buildings is critical to control wetting.

Post-Tensioned Slab-On-Grade (PTS) Foundation

A post-tensioned slab foundation incorporates the floor and will likely perform better than a conventional floor slab. Design and construction criteria for PTS foundations are presented below.

PTS design is based on methods developed by the Post-Tensioning Institute (PTI) and outlined in PTI's third edition of *Design of Post-Tensioned Slabs-on-Ground* (2004). Various climate and relevant soil factors are required to evaluate the PTI design criteria. These include the Thornthwaite Moisture Index (I_m), suction compression index (γ_h), unsaturated diffusion coefficient (α), depth of probable moisture variation, Atterberg limits, initial and final soil suction profiles, and percent clay fraction and predominant clay mineral. In the Denver Metro area, I_m is about -25.

In the PTI design method, two mechanisms of soil movement (edge lift and center lift) are evaluated based on assumptions that wetting and drying of the foundation soils are primarily affected by the climate and are the prominent movement mechanisms that will influence foundation performance. Our experience indicates the foundation soils will normally undergo an increase in moisture due to covering the ground surface with buildings and flatwork, coupled with the introduction of landscape irrigation. Considering the limitations of the current PTI design method, we believe a conservative approach with reasonable engineering judgment is merited in PTS foundation design.



Design criteria for post-tensioned slab-on-grade foundations are presented below. Criteria were developed from analysis of field and laboratory data, the PTS design method outlined in PTI's third edition of *Design and Construction of Post-Tensioned Slabs-on-Ground* (2004), and our experience.

1. PTS foundations should be constructed on moisture conditioned, compacted fill or native, undisturbed soil or sandstone bedrock. Where soils are loosened during excavation or in the forming process, or if any loose, soft, or dry soils are exposed in excavations, the soils should be removed and re-compacted as recommended in **Fill and Backfill**, prior to placing concrete.
2. PTS foundations should be designed for a maximum allowable soil pressure of 3,000.
3. Using the PTI design method, we recommend a differential soil movement (y_m) of 2 inches for edge and center lift conditions.
4. The PTI 3rd Edition includes two methods for determining the edge moisture variation distance (e_m) and states that the higher values should be used. Based on the method using the Thornthwaite Index, the moisture variation distances for center lift and edge lift (e_m) of 5.5 feet and 2.5 feet are indicated, respectively. Based on the unsaturated diffusion coefficient, center lift and edge lift e_m values of 9 feet and 4.5 feet are indicated, respectively. Based on the subsurface conditions and experience with PTS foundation use in Denver, we believe it is probable that use of the average values (center lift of 7.25 feet and edge lift of 3.5 feet) may yield acceptable performance. We can consult with the structural engineer further as design progresses, if desired.
5. We understand the PTI design method assumes the slab is somewhat flexible. The above-grade construction, such as framing, drywall, brick, and stucco are not as flexible. We are aware of situations where minor differential slab movement has caused distress to finish materials. One way to enhance performance would be to place reinforcing steel in the bottom of stiffening beams. The structural engineer should evaluate the merits of this approach, as well as other potential alternatives to reduce damage to finish materials.
6. Stiffening beams maybe poured "neat" into excavated trenches. Soil may cave or slough during trench excavation for the stiffening beams. Disturbed soil should be removed from trench bottoms prior to placement of concrete. Formwork or other methods may be required for proper stiffening beam installation.
7. Exterior stiffening beams should be protected from frost action. Three feet of frost cover is normally assumed in the area. If exterior patios are incorporated into PTS, we believe the stiffening beams around the patios should be as deep as those around the building exterior to increase the likelihood they will perform similarly to the rest of the PTS.



8. For slab tensioning design, a coefficient of friction value of 0.75 or 1.0 can be assumed for slabs on polyethylene sheeting or a sand layer, respectively. A coefficient of friction of 2.0 should be used for slabs on clay fill. We believe use of polyethylene is preferable because it serves as a vapor retarder which helps to control moisture migration up through the slabs.
9. A representative of our firm should observe the completed excavations. A representative of the structural engineer or our firm should observe the placement of the reinforcing tendons and any mild reinforcement prior to placing the slabs and beams.

FLOOR SYSTEMS

Slab Performance Risk

There is risk of distress due to expansive soil and bedrock and settlement of fill at this site. We estimate total potential ground heave of up to about 6½ inches considering a 24-foot depth of wetting below anticipated finished grade, based on existing conditions. We judge there is low to high risk of poor basement/ground-supported slab performance across the site (Fig. 3). Risk should be reduced to low after sub-excavation is performed and results in low swelling fill. The only method to significantly reduce the risk of slab movement is to use a structural floor supported by the foundation system or a slab constructed with a void between the slab and soils. Structurally supported floors should be used for all finished living areas unless PTS are used. For PTS, the foundations are structurally integral with the floor and should perform better than a conventional slab-on-grade floor.

Basement Slabs-On-Grade

Basements are planned for some of the structures. Our experience indicates basement slab performance has generally been satisfactory on low risk lots. Slab movement of 1 to 2 inches is considered “normal” for these sites. In unusual cases, movement can exceed this range. Slab-on-grade basement floors may be used for basements on low risk lots provided the estimated slab movements and associated damages are acceptable. We recommend structurally supported basement floors on high risk lots and moderate risk lots with basements. If movement cannot be tolerated, a structurally supported floor should be installed. Where slabs-on-



grade are provided in basements, we recommend the precautions for slab-on-grade construction which are included in Exhibit A. We recommend that conventional slab-on-grade construction be limited to garages and exterior flatwork.

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 or sub-excavation fill), 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 such as slabs cast-on-void-form, 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.

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. This detail is often accomplished by placing void form below utility lines.



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 minimum, 10 mil for better durability) placed on the soils below accessible subfloor areas. The vapor retarder/barrier should be sealed at joints and attached to concrete foundation elements.

Garage Floors and Exterior Flatwork

Garage floors, driveways and sidewalks are normally constructed as slabs-on-grade. Performance of conventional slabs on expansive soils is erratic. Various properties of the soils and environmental conditions influence magnitude of movement and other performance. Increases in the moisture content in these soils will cause heaving and may result in cracking of slabs-on-grade. Backfill below slabs should be moisture conditioned and compacted to reduce settlement, as discussed in **Fill and Backfill**. Driveways and exterior slabs founded on the backfill may settle and crack if the backfill is not properly moisture treated and compacted.

Concrete driveways and other surface improvements installed outside the sub-excavated area (where applicable) may be affected by the expansive clay and claystone. These improvements may exhibit significant heave, cracking or other distress. This distress can impinge on finishes and framing. Sub-excavation can be considered below these improvements to enhance performance.

We recommend exterior flatwork and sidewalks around the buildings be isolated to reduce the risk of transferring slab movement to the structures. 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 buildings, rather than tying the slabs directly into the buildings' 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.



Porches, Decks, and Patios

Porches and decks with roofs that are integral with a building should be constructed with the same foundation type as the building when porch or deck foundation movement would damage the house. Deck foundations should be designed by a structural engineer. For simple decks that are not integral with the building and can tolerate some movement, the use of short pier or footing foundations bottomed at least 3 feet below grade can be considered, as long as, the foundations are located outside foundation wall backfill. Deck foundations should be bottomed below foundation wall backfill to reduce risk of settlement; longer (8 to 10 feet or more) deck piers may be necessary to provide adequate support. The inner edge of a deck may be constructed on haunches or steel angles bolted to the foundation walls and detailed such that movement of the deck foundation will not cause distress to the residence. We suggest use of adjustable bracket-type connections or other details between foundations and deck posts so the posts can be adjusted if movement occurs.

Patio slabs, and other exterior flatwork which are not part of a PTS should be isolated from the structures. Porch slabs can be constructed to reduce the likelihood that settlement or heave will affect the slabs. One approach (for smaller porches located over backfill zones) is to place loose backfill under a structurally supported slab. A lower risk approach is to construct the structural porch slab over void-forming materials. Conditions should allow the void-forming materials to soften quickly after construction to reduce the risk of transmitting ground heave to the porch slab. Wax or plastic-coated void boxes should not be used unless provisions are made to allow water vapor to penetrate the boxes, resulting in softening.

POOL AND DECK

A pool is planned on the north side of the clubhouse within the southern multi-family development. We anticipate the pool will be about 4 to 8 feet deep and be cast-in-place concrete or gunite. Rigid concrete or gunite pools are brittle and can crack from shrinkage or movement. Use of a well-reinforced gunite or concrete pool can reduce potential cracking.

The pool deck is normally constructed as a slab-on-grade. The most common problem with swimming pool performance in this area is heave or settlement and cracking of the deck



slabs surrounding the pool. Our testing indicates potential heave could be up to 2½ inches. Infiltration of surface water from precipitation or pool splash can cause moisture seepage through cracks in the pool deck and the joints between the deck and pool. This wetting causes the underlying soils to swell or compress and results in cracking and distress of the deck, and potentially the nearby building foundations. Sub-excavation to a depth of 3 feet can be performed below the pool deck to improve performance. In our opinion, the swelling hazard can be reduced, but not prevented, by construction considerations, maintenance, and providing rapid surface water runoff to drains and other stormwater features outside the pool area.

The deck and surrounding areas should be sloped to provide positive drainage. We recommend slabs be isolated from the pool shell, and well-reinforced to function as independent units. Cracking of the pool deck may occur and will require maintenance. Cracks and joints in the deck should be assessed and sealed seasonally, at a minimum. Cracking of the pool and pool deck can allow water to infiltrate the subgrade soils which leads to soil movement problems and possibly hydrostatic uplift of the pool shell. A drain board installed around the perimeter of the pool walls is recommended to facilitate drainage of free water within the subsoil. This free water should be captured in a gravel drain. The drain should be sloped to a sump where the water can be removed by pumping. Impermeable PVC sheeting (20 mil or thicker) should be placed between the gravel drain and the subgrade. The PVC sheeting should be adequately sealed at the joints. A conceptual detail for the pool drainage is shown on Fig. 11.

DETENTION POND CONSTRUCTION

Based on strata encountered in our borings, the ponds will be predominantly underlain by interlayered sandstone and claystone bedrock. Groundwater was encountered at the elevations shown on the table below. The Colorado Department of Public Health and Environment (CDPHE) may require treatment of storm water if it mixes with groundwater. Plans did not show locations of micro-basins or outlet structures or indicate at what elevations they may be constructed. We should review these plans once they are available.

Permeability of claystone is negligible. The permeability of sandstone will vary depending on how well consolidated the layer is, the percent fines present in the matrix, and/or the extent of cementation (if present). For non-cemented zones, estimated permeability on the order



of 10 inches per hour can be considered for design. We can conduct infiltration testing near the planned detention pond bottoms if desired to refine this estimate. Fill composed of on-site clay will be essentially impervious.

TABLE II
GROUNDWATER DEPTHS AND ELEVATIONS
OF DETENTION POND BORINGS

Detention Boring	Depth to Groundwater Surface (feet)		Elevation of Groundwater Surface (feet)
	From Existing Grade	From Proposed Grade	
TH-79	39	16½	6235½
TH-80	18	20	6211½
TH-81	28	12	6217

PAVEMENTS

Pavement areas will be used for automobile parking, access drives, and truck/fire lanes. We drilled 13 shallow borings in proposed pavement areas. Our investigation indicates pavement subgrade soils are variable and may consist of clay, sand, bedrock, or fill of similar composition. The clayey soils will provide relatively poor support. Pavements can experience heave due to expansive clay or claystone or settlement due to compression of clay or utility trench backfill. Testing indicates clayey subgrade soils could swell up to 10 percent.

We recommend sub-excavating pavement areas to a depth of at least 3 feet to enhance pavement performance. Sub-excavation should extend horizontally to the 1-foot beyond the back of curb or the back of the attached sidewalk, if any. Subgrade should be prepped by scarifying the pavement subgrade to a depth of at least 12 inches prior to moisture conditioning, compacting, and proof-rolling 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. Pavement alternatives are presented below.



TABLE III
SUMMARY OF RECOMMENDED MINIMUM PAVEMENT SECTIONS

Roadway Classification	Full-Depth Hot Mix Asphalt (HMA)	Hot Mix Asphalt + Aggregate Base Course (HMA + ABC)	Portland Cement Concrete (PCC)
Automobile Parking	6" HMA	4" HMA + 6" ABC	6" PCC
Access Drives and Fire/Truck Lanes	7" HMA	5" HMA + 8" ABC	6" PCC

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 and trash collection areas, we recommend a 6-inch or thicker Portland cement concrete pavement placed directly on prepared subgrade. The concrete pads should be of sufficient size to accommodate truck turning, trash pickup and delivery/loading areas.

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. Recommended material properties and construction criteria for pavements are provided in Appendix E. 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 reevaluated based upon available materials.

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.



Flexible and rigid pavement materials, construction and maintenance guidelines are presented in Appendix E.

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.

BELOW-GRADE WALLS

Basement and/or foundation walls and grade beams that extend below grade should be designed to resist lateral earth pressures where backfill is not present to about the same extent on both sides of the wall. Many factors affect the value of the design lateral earth pressure. These factors include, but are not limited to the type, compaction, slope and drainage of the backfill, and the rigidity of the wall against rotation and deflection. 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 can deflect or rotate 0.5 to 1 percent of the wall height (depending upon the backfill types), lower “active” lateral earth pressures are appropriate. Our experience indicates foundation walls can deflect or rotate slightly under normal design loads, and that this deflection typically does not affect the structural integrity of the walls. Thus, the earth pressure on the walls will likely be between the “active” and “at-rest” conditions.

If on-site soils are used as backfill and the backfill is not saturated, we recommend design of foundation walls at this site using an equivalent fluid density of at least 60 pcf. This value assumes deflection; some minor cracking of walls may occur. If very little wall deflection is desired, a higher design value is appropriate. The structural engineer should also consider site-specific grade restrictions on the behavior of the walls.



BACKFILL COMPACTION

Settlement of foundation wall and utility trench backfill can cause damage to concrete flatwork and/or result in poor drainage conditions. Compaction of backfill can reduce settlement. Attempts to compact backfill near foundations to a high degree can cause damage to foundation walls and may increase lateral pressures on the foundation walls. The potential for cracking of a foundation wall can vary widely based on many factors including the degree of compaction achieved, the weight and type of compaction equipment utilized, the structural design of the wall, the strength of the concrete at the time of backfill compaction, and the presence of any bracing.

Proper moisture-conditioning of backfill is just as important, because settlement commonly occurs in response to wetting. The addition of water complicates the backfill process, especially during cold weather. Frozen soils are not considered suitable for use as backfill because excessive settlement can result when the frozen materials thaw. Exhibit D describes four alternative methods to place, moisture-condition, and compact backfill along with a range of possible settlements, and advantages and disadvantages of each approach, all based upon our experience. These are just a few of the possible techniques and represent a range for your evaluation. We recommend Alternatives C or D if you wish to control potential settlement.

Precautions should be taken when backfilling against a foundation wall. Temporary bracing of comparatively long, straight sections of foundation walls should be used to limit damage to walls during the compaction process. Waiting at least seven days after the walls are placed to allow the concrete to gain strength can also reduce the risk of damage. Compaction of fill placed beneath and next to counterforts (if any) and grade beams may be difficult to achieve without damaging these building elements. Proper moisture-conditioning of the fill prior to placement in these areas will help reduce potential settlement.

SURFACE AND SUBSURFACE DRAINAGE

Surface water can penetrate relatively permeable loose backfill soils located adjacent to structures and collect at the bottom of relatively impermeable basement or crawl space excavations, causing wet or moist conditions. Foundation walls which retain earth should be designed



for lateral earth pressures. Foundation drains should be constructed around the lowest excavation levels and ideally should be connected to an underdrain system to provide a gravity outlet. A foundation drain should also be built where walls extend below grade. The drains should be connected to a sump pit where water can be removed by pumping if an underdrain is not provided. The drain should consist of a 4-inch diameter, perforated or slotted pipe encased in free-draining gravel. Elevator/stairwell cores (if any) will likely extend about 5 feet deeper than the finished floor elevation and will require their own perimeter drain system. Alternatively, they can be designed and constructed to be watertight. Buoyancy effects should be considered if constructed watertight. Typical foundation wall drain details are presented on Figs. 12 through 17. Window well drain details are presented on Figs. 18 and 19.

Our experience indicates moist conditions can develop in crawl spaces or basements (if constructed), resulting in isolated instances of damp soils, musty smells, and, in rare cases, standing water. Crawl spaces should be well ventilated, depending on the use of a vapor retarder/barrier and the floor material selected.

Proper design, construction, and maintenance of surface drainage are critical to the satisfactory performance of foundations, slabs-on-grade, and other improvements. Landscaping and irrigation practices will also affect performance. Exhibit B contains our recommendations for surface drainage, irrigation, and maintenance. Variation from these criteria is acceptable in some areas. Roof downspouts and other water collection systems should discharge beyond the limits of backfill around structures.

Proper control of surface runoff is also important to control the erosion of surface soils. Sheet flow should not be directed over unprotected slopes. Water should not be allowed to pond at the crest of slopes. Permanent slopes should be prepared in such a way to reduce erosion.

Attention should be paid to compaction of the soils behind curbs and gutters adjacent to streets and in utility trenches during the construction and development. If surface drainage between preliminary development and construction phases is neglected, performance of the roadways, flatwork and foundations may be poor.



CONCRETE

Concrete in contact with soil can be subject to sulfate attack. We measured water-soluble sulfate concentrations in 7 samples during this investigation. Concentrations were measured at less than 0.01 to 0.06 percent, averaging 0.02 percent. As indicated in our tests and ACI 332-20, the sulfate exposure class is *Not Applicable* or *RS0*.

SULFATE EXPOSURE CLASSES PER ACI 332-20

Exposure Classes		Water-Soluble Sulfate (SO ₄) in Soil ^A (%)
Not Applicable	RS0	< 0.10
Moderate	RS1	0.10 to 0.20
Severe	RS2	0.20 to 2.00
Very Severe	RS3	> 2.00

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

For this level of sulfate concentration, ACI 332-20 *Code Requirements for Residential Concrete* 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 332-20

Exposure Class	Maximum Water/Cement Ratio	Minimum Compressive Strength ^A (psi)	Cementitious Material Types ^B			Calcium Chloride Admixtures
			ASTM C150/C150M	ASTM C595/C595M	ASTM C1157/C1157M	
RS0	N/A	2500	No Type Restrictions	No Type Restrictions	No Type Restrictions	No Restrictions
RS1	0.50	2500	II	Type with (MS) Designation	MS	No Restrictions
RS2	0.45	3000	V ^C	Type with (HS) Designation	HS	Not Permitted
RS3	0.45	3000	V + Pozzolan or Slag Cement ^D	Type with (HS) Designation plus Pozzolan or Slag Cement ^E	HS + Pozzolan or Slag Cement ^E	Not Permitted

- A) Concrete compressive strength specified shall be based on 28-day tests per ASTM C39/C39M
- B) Alternate combinations of cementitious materials of those listed in ACI 332-20 Table 5.4.2 shall be permitted when tested for sulfate resistance meeting the criteria in section 5.5.
- C) Other available types of cement such as Type III or Type I are permitted in Exposure Classes RS1 or RS2 if the C3A contents are less than 8 or 5 percent, respectively.
- D) 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 slag to be used shall not be less than the amount tested in accordance with ASTM C1012/C1012M and meeting the criteria in section 5.5.1 of ACI 332-20.
- E) Water-soluble chloride ion content that is contributed from the ingredients including water aggregates, cementitious materials, and admixtures shall be determined on the concrete mixture ASTM C1218/C1218M between 29 and 42 days.



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 $\pm$ 1.5 percent. We advocate all foundation walls and grade beams in contact with the subsoils (including the inside and outside faces of garage and crawl space grade beams) be damp-proofed.

CONSTRUCTION OBSERVATIONS

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

GEOTECHNICAL RISK

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

LIMITATIONS

This report has been prepared for the exclusive use of BMC Investments and your design team to provide geotechnical design and construction criteria for the proposed development. The information, conclusions, and recommendations presented herein are based upon consideration of many factors including, but not limited to, the type of structures proposed, the



geologic setting, and the subsurface conditions encountered. The conclusions and recommendations contained in the report are not valid for use by others. Standards of practice evolve in geotechnical engineering. The recommendations provided are appropriate for about three years. If the site is not developed within about three years, we should be contacted to determine if we should update this report.

Our borings were spaced to provide a general picture of subsurface conditions below the proposed Canyonside Single and Multi-Family Development project. Variations from our borings should be anticipated. We believe this investigation was conducted in a manner consistent with that level of care and skill ordinarily used by geotechnical engineers practicing in this area at this time. No warranty, express or implied, is made. If we can be of further service in discussing the contents of this report or analysis of the influence of subsurface conditions on the project, please call.

CTL | THOMPSON, INC.

Robert J. Brown
Staff Geologist



Reviewed by:

Erin Bouchet
Apr 18 2023

Erin C. Bouchet, P.E., P.G.
Associate Engineer | Denver Engineering Manager

Via e-mail: tim.odonnell@bmcinv.com



EXHIBIT A

SLAB PERFORMANCE RISK EVALUATION, INSTALLATION AND MAINTENANCE

As part of our evaluation of the subsoils and bedrock, samples were tested in the laboratory using a swell test. In the test procedure, a relatively undisturbed sample obtained during drilling is first loaded and then flooded with water and allowed to swell. The pressure applied prior to wetting can approximate the weight of soil above the sample depth or be some standard load. The measured percent swell is not the sole criteria in assessing potential movement of slabs-on-grade and the risk of poor slab performance. The results of a swell test on an individual boring are tempered with data from surrounding borings, depth of tests, depth of excavation, soil profile, and other tests. This judgment has been described by the Colorado Association of Geotechnical Engineers⁴ (CAGE, 1996) as it relates to basement slab-on-grade floors. It can also be used to help judge performance risk for other slabs-on-grade such as garage floors, driveways, and sidewalks. CTL Thompson also performs potential heave calculations to aid in our judgment. The risk evaluation is considered when we evaluate appropriate foundation systems for a given site. In general, more conservative foundation designs are used for higher risk sites to control the likelihood of excessive foundation movement.

As a result of the Slab Performance Risk Evaluation, sites are categorized as low, moderate, high, or very high risk. This is a judgment of the swelling characteristics of the soils and bedrock likely to influence slab performance.

REPRESENTATIVE MEASURED SWELL AND CORRESPONDING SLAB PERFORMANCE RISK CATEGORIES

Slab Performance Risk Category	Representative Percent Swell* (500 psf Surcharge)	Representative Percent Swell* (1000 psf Surcharge)
Low	0 to <3	0 to <2
Moderate	3 to <5	2 to <4
High	5 to <8	4 to <6
Very High	≥ 8	≥ 6

*Note: The representative percent swell values presented are not necessarily measured values; rather, they are a judgment of the swelling characteristics of the soil and bedrock likely to influence slab performance.

The rating of slab performance risk on a site as low or high is not absolute. Rather, this rating represents a judgment. Movement of slabs may occur with time in low, moderate, high, and very high risk areas as the expansive soils respond to increases in moisture content. Overall, the severity and frequency of slab damage usually is greater in high and very high rated areas. Heave of slabs-on-grade of 3 to 5 inches is not uncommon in areas rated as high or very high risk. On low and moderate risk sites, slab heave of 1 to 2 inches is considered normal and we believe in the majority of instances, movements of this magnitude constitute reasonable slab performance; more heave can occur. Slabs can be affected on all sites. On lots rated as high or very high risk, there is more likelihood of need to repair, maintain or replace garage floors and exterior flatwork.

⁴"Guideline for Slab Performance Risk Evaluation and Residential Basement Floor System Recommendations", Colorado Association of Geotechnical Engineers, December 1996.



The property manager and renter should be advised slabs-on-grade may move and crack due to heave or settlement and that there may be maintenance costs associated during and after the builder warranty period. Heave or settlement may require maintenance of finish details to control damage. Our experience suggests that soil moisture increases below residence sites due to covering the ground with the residence and exterior flatwork, coupled with the introduction of landscape irrigation. In most cases, slab movements (if any) resulting from this change occur within three to five years.

For portions of the residences where conventional slabs-on-grade are used, we recommend the following precautions. These measures will not keep slabs-on-grade from heaving; they tend to mitigate damages due to slab heave.

1. Slab-on-grade floor construction should be limited to areas where slab movement and cracking are acceptable to the builder and property manager.
2. The International Residential Code (IRC R506) states that a 4-inch base course layer consisting of clean graded sand, gravel, crushed stone or crushed blast furnace slag shall be placed beneath below slab-on-grade floors (unless the underlying soils are free-draining), along with a vapor retarder. Installation of the base course and vapor retarder is not common in this area. Historically, there has been some concern that installation of clean base course could allow wetting of expansive soils to spread from an isolated source.
3. IRC states that the vapor retarder can be omitted where approved by the building official. 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 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. 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 American Concrete Institute (ACI) Committee 302, "Guide for Concrete Floor and Slab Construction (ACI 302.1R-15)".
4. Conventional slabs should be separated from exterior walls and interior bearing members with a slip joint that allows free vertical movement of the slabs. These joints must be maintained by the homebuyer to avoid transfer of movement.
5. Underslab plumbing (if any) should be thoroughly pressure tested during construction for leaks and be provided with flexible couplings. Gas and waterlines leading to slab-supported appliances should be constructed with flexibility. The homebuyer must maintain these connections.



6. Use of slab bearing partitions (if any) should be minimized. Where such partitions are necessary, a slip joint (or float) allowing at least 2 inches of free vertical slab movement should be used. Doorways should also be designed to allow vertical movement of slabs. To limit damage in the event of movement, sheetrock or dry-wall should not extend to the floor. The homebuyer should monitor partition voids and other connections and re-establish the voids before they close to less than ½-inch.
7. Plumbing and utilities that pass through slabs (if any) should be isolated from the slabs. Heating and air conditioning systems constructed on slabs should be provided with flexible connections capable of at least 2 inches of vertical movement so slab movement is not transmitted to the ductwork. These connections must be maintained by the property manager and/or renter.
8. Roofs that overhang a patio or porch should be constructed on the same foundation as the residence. Isolated piers or pads may be installed beneath a roof overhang provided the slab is independent of the foundation elements. Patio or porch roof columns may be positioned on the slab, directly above the foundation system, provided the slab is structural and supported by the foundation system. Structural porch or patio slabs should be constructed to reduce the likelihood that settlement or heave will affect the slab by placing loose backfill under the structurally supported slab or constructing the slab over void-forming materials.
9. Patio and porch slabs without roofs and other exterior flatwork should be isolated from the foundation. Movements of slabs should not be transmitted to the foundation. Decks are more flexible and more easily adjusted in the event of movement.
10. Frequent control joints should be provided in conventional slabs-on-grade to reduce problems associated with shrinkage cracking and curling. Panels that are approximately square generally perform better than rectangular areas. We suggest an additional joint about 3 feet away from and parallel to foundation walls.



EXHIBIT B

SURFACE DRAINAGE, IRRIGATION AND MAINTENANCE

Performance of foundations and concrete flatwork is influenced by the moisture conditions existing within the foundation soils. Surface drainage should be designed to provide rapid runoff of surface water away from proposed residences and townhome buildings. Proper surface drainage and irrigation practices can help control the amount of surface water that penetrates to foundation levels and contributes to settlement or heave of soils and bedrock that support foundations and slabs-on-grade. Positive drainage away from the foundation and avoidance of irrigation near the foundation also help to avoid excessive wetting of backfill soils, which can lead to increased backfill settlement and possibly to higher lateral earth pressures, due to increased weight and reduced strength of the backfill. CTL|Thompson, Inc. recommends the following precautions. The homebuyer should maintain surface drainage and, if an irrigation system is installed, it should substantially conform to these recommendations.

1. Wetting or drying of the open foundation, utility and earthwork excavations should be avoided.
2. Excessive wetting of foundation soils before, during and after construction can cause heave or softening of foundation soils and result in foundation and slab movements. Proper surface drainage around and between the buildings is critical to control wetting.
3. The ground surface surrounding the exterior of each building should be sloped to drain away from the structure in all directions. We recommend a minimum constructed slope of at least 6 inches in the first 10 feet (5 percent) in landscaped areas around each building without basements, and 12 inches in the first 10 feet (10 percent) in landscaped areas around each residence with a basement, where practical. The recommended slope is for the soil surface slope, not surface of landscaping rock.
4. We do not view the recommendation to provide a 5 percent slope away from the foundation as an absolute. It is desirable to create this slope where practical, because we know that backfill will likely settle to some degree. By starting with sufficient slope, positive drainage can be maintained for most settlement conditions. There are many situations around a building where a 5 percent slope cannot be achieved practically, such as around patios, at inside foundation corners, and between a house and nearby sidewalk. In these areas, we believe it is desirable to establish as much slope as practical and to avoid irrigation. We believe it is acceptable to use a slope on the order of 5 percent perpendicular to the foundation in these limited areas.
5. Between buildings that are separated by a distance of less than 20 feet, the constructed slope should generally be at least 5 percent to the swale used to convey water out of this area. For lots that are graded to drain to the front and back, we believe it is acceptable to install a slope of 2 to 3 percent at the high point (aka "break point") between houses.



6. Construction of retaining walls and decks adjacent to buildings should not alter the recommended slopes and surface drainage around the foundation. Ground surface under the deck should be compacted and slope away from the residence. A 10-mil plastic sheeting and landscaping rock is recommended above the ground under the decks to reduce water dripping from the deck causing soil erosion and/or forming depressions under the deck. The plastic sheeting should direct water away from the residence. Retaining walls should not flatten the surface drainage around the residence and block or impede the surface run-off.
7. Swales used to convey between building should be sloped so that water moves quickly and does not pond for extended periods of time. We suggest minimum slopes of about 2 to 2.5 percent in grassed areas and about 2 percent where landscaping rock or other materials are present. If slopes less than about 2 percent are necessary, concrete-lined channels or plastic pipe should be used. Fence posts, trees, and retaining walls should not impede the runoff in the swale.
8. Backfill around the foundations (if any) should be moistened and compacted.
9. Roof downspouts and drains should discharge well beyond the limits of backfill. Splash blocks and/or extensions should be provided at all downspouts so water discharges onto the ground beyond the backfill. We generally recommend against burial of downspout discharge. Where it is necessary to bury downspout discharge, solid, rigid pipe should be used, and it should slope to an open gravity outlet. Downspout extensions, splash blocks and buried outlets must be maintained by property managers.
10. The importance of proper irrigation practices cannot be over-emphasized. Irrigation should be limited to the minimum amount sufficient to maintain vegetation; application of more water will increase likelihood of slab and foundation movements. Landscaping should be carefully designed and maintained to minimize irrigation. Plants placed close to foundations, particularly within 5 feet of the foundation, should be limited to those with low moisture requirements and utilize only sub-surface irrigation such as standard low volume drip emitters or in-line drip irrigation. Irrigated grass, irrigation mainlines, above-surface spray heads, rotors, or other above-surface irrigation spray devices should not be located or discharge above the ground surface within 5 feet of the foundation.
11. Plastic sheeting should not be placed beneath landscaped areas adjacent to foundation walls or grade beams. Geotextile fabric can inhibit weed growth yet still allow natural evaporation to occur.
12. The design and construction criteria for foundations and floor system alternatives were compiled with the expectation that all other recommendations presented in this report related to surface and subsurface drainage, landscaping irrigation, backfill compaction, etc. will be incorporated into the project. It is critical that all recommendations in this letter are followed.



EXHIBIT C

MITIGATION OF EXPANSIVE SOILS AND BEDROCK CANYONSIDE – SINGLE AND MULTI-FAMILY DEVELOPMENT CASTLE PINES, COLORADO

Much of the soils and underlying bedrock found beneath the Front Range area of Colorado are expansive; they swell when wetted. Deep sub-excavation of these clayey soils and bedrock is considered an appropriate mitigation method in areas where expansive materials are present. Excavation destroys the fractures and non-uniform structure of the near-surface soils and bedrock (Fig. 1). At BMC Canyons, variable depths of sub-excavation are recommended.

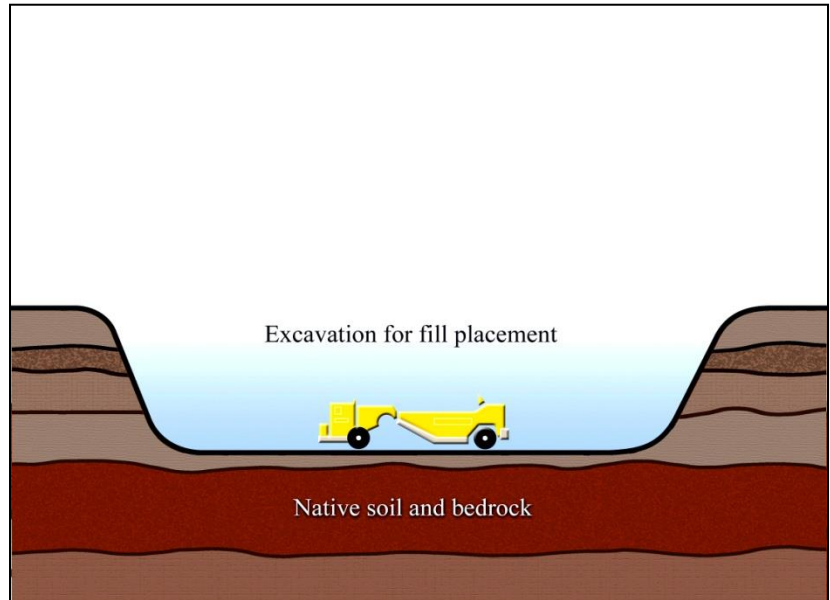


Figure 1

The excavated soils and bedrock are compacted in thin, nearly horizontal layers (Fig. 2). Water is mixed with the materials as they are placed to "pre-swell" the expansive soils and reduce potential future swell. On projects where deep sub-excavation has been employed, testing of the soils, bedrock, and fill has demonstrated that significant reduction in swelling characteristics can be achieved.

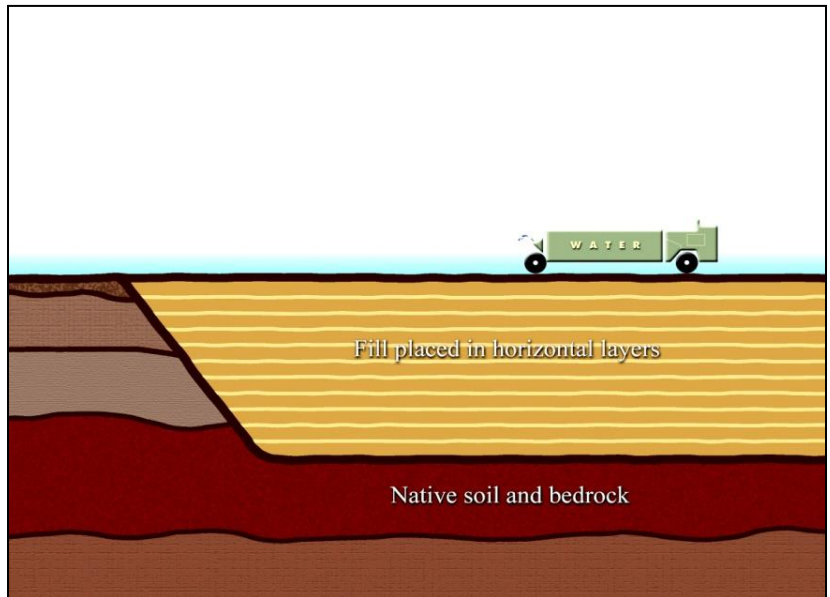
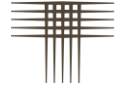


Figure 2



The sub-excavation approach mitigates the influence of expansive materials on man-made improvements and structures in several ways. The weight of the fill helps to resist potential heave of the underlying natural materials. The fill acts like a cushion to spread movement over a larger area, reducing damaging movements near the ground surface (Fig. 3). The fill also covers the site with a thick layer of compacted material that limits penetration of water to the underlying natural soils and/or bedrock.

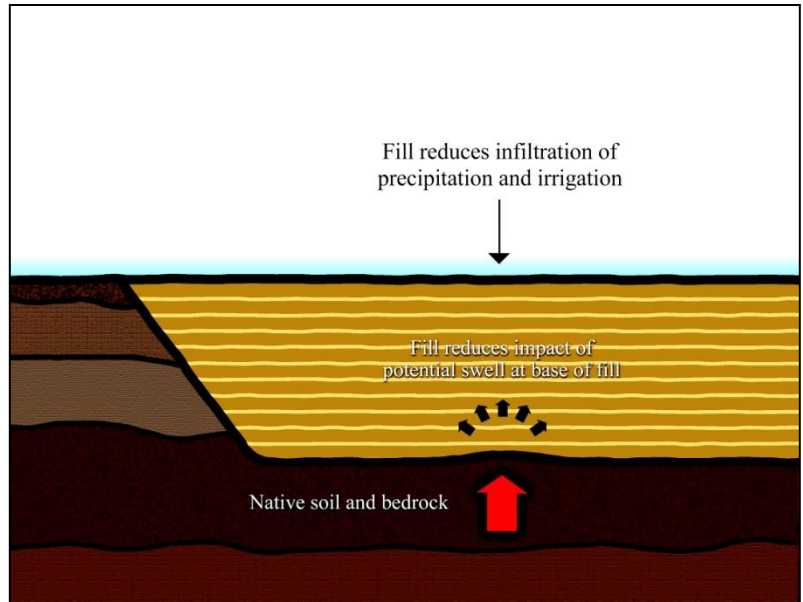


Figure 3

The sub-excavation process can provide greater stability for streets, sidewalks, driveways, and buried utilities (Fig. 4), if the sub-excavated layer extends beneath these improvements.

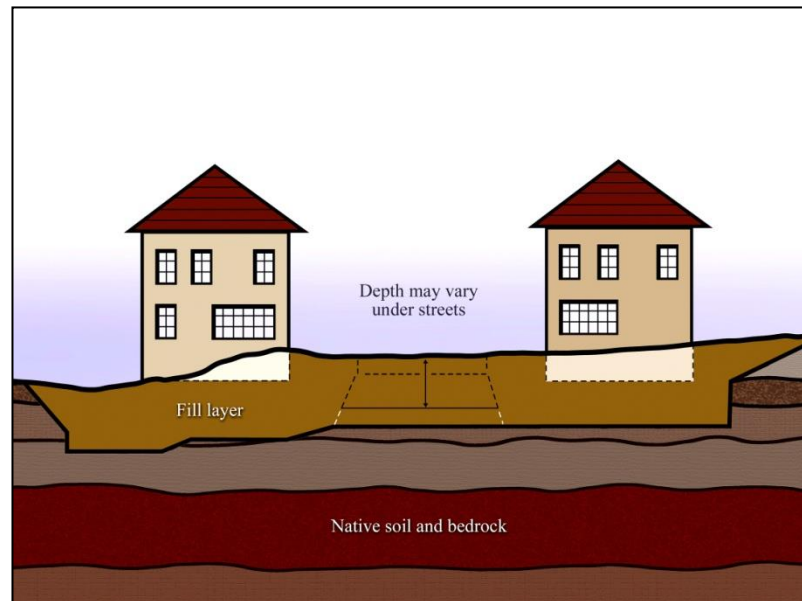
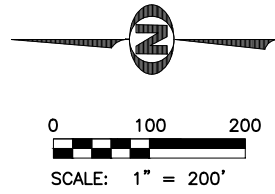


Figure 4



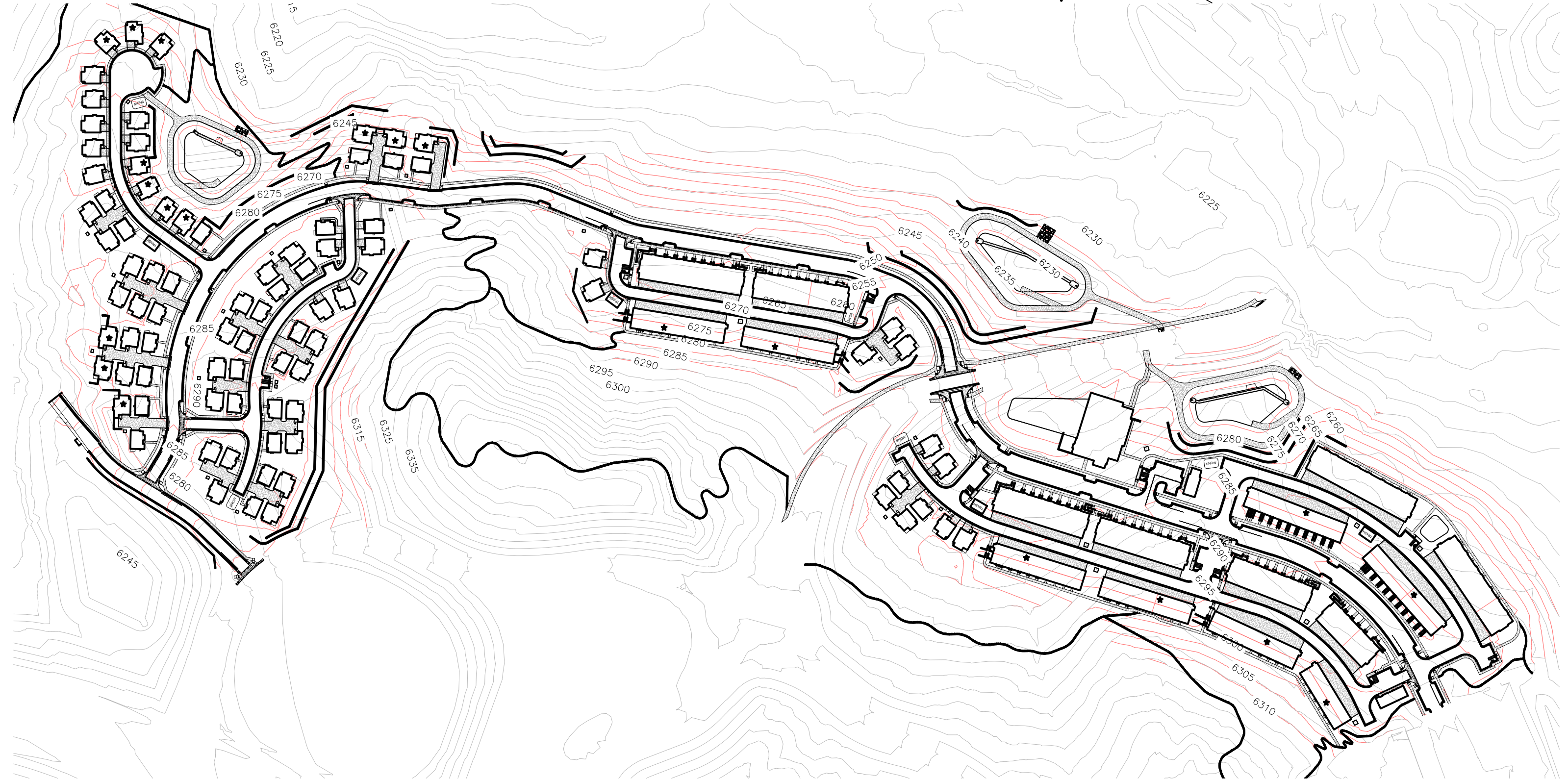
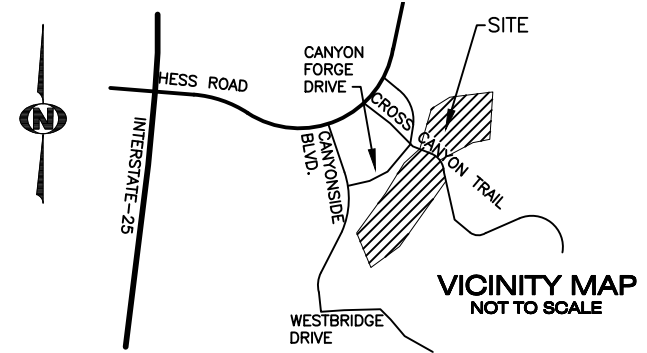
EXHIBIT D
EXAMPLE BACKFILL COMPACTION ALTERNATIVES

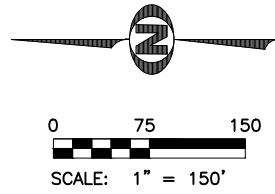
Alt.	Description	Possible Settlement	Pros (+) / Cons (-)
A	Place in 18 to 24-inch lifts, without moisture conditioning. Compact lift surface to about 85 percent of maximum standard Proctor (ASTM D698) dry density.	5 to 15% of depth (for 3 feet of backfill, 2 to 5 inches) (for 8 feet, 5 to 15 inches)	+ Fast + Water not required - Excessive Settlement - Highest water penetration - Highest probability of warranty repair
B	Moisture condition to 1 to 4 percent above optimum for clay and within 2 percent of optimum for sand and place in 12 to 18-inch lifts. Compact lift surface to about 85 to 90 percent.	5 to 10 percent of depth	+ Relatively Fast - Moderate water penetration - Excessive settlement - Need for water - Warranty repairs probable
C	Moisture condition to 1 to 4 percent above optimum for clay and within 2 percent of optimum for sand and place in 8 to 12-inch lifts. Compact lift surface to 90 to 95 percent.	2 to 5 percent of depth	+ Reduced warranty + Reduced water infiltration + Reduced settlement - Possible higher lateral pressure - Slower - Need for water - Potential damage to walls
D	Moisture condition and place as in C. Compact lift surface to at least 95 percent.	1 to 2 percent of depth	+ Reduced warranty + Reduced water infiltration + Lowest comparative settlement - Possible higher lateral pressure - Slower - Need for water - Potential damage to walls



LEGEND:

- INDICATES EXISTING GRADE
- INDICATES PROPOSED GRADE
- ★ INDICATES LOT/BUILDING WITH PLANNED BASEMENT





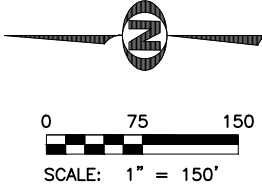
LEGEND:

- TH-1 APPROXIMATE LOCATION OF EXPLORATORY BORING
-
- S-1 APPROXIMATE LOCATION OF PAVEMENT BORING
- ⊙

- INDICATES EXISTING GRADE
- INDICATES PROPOSED GRADE

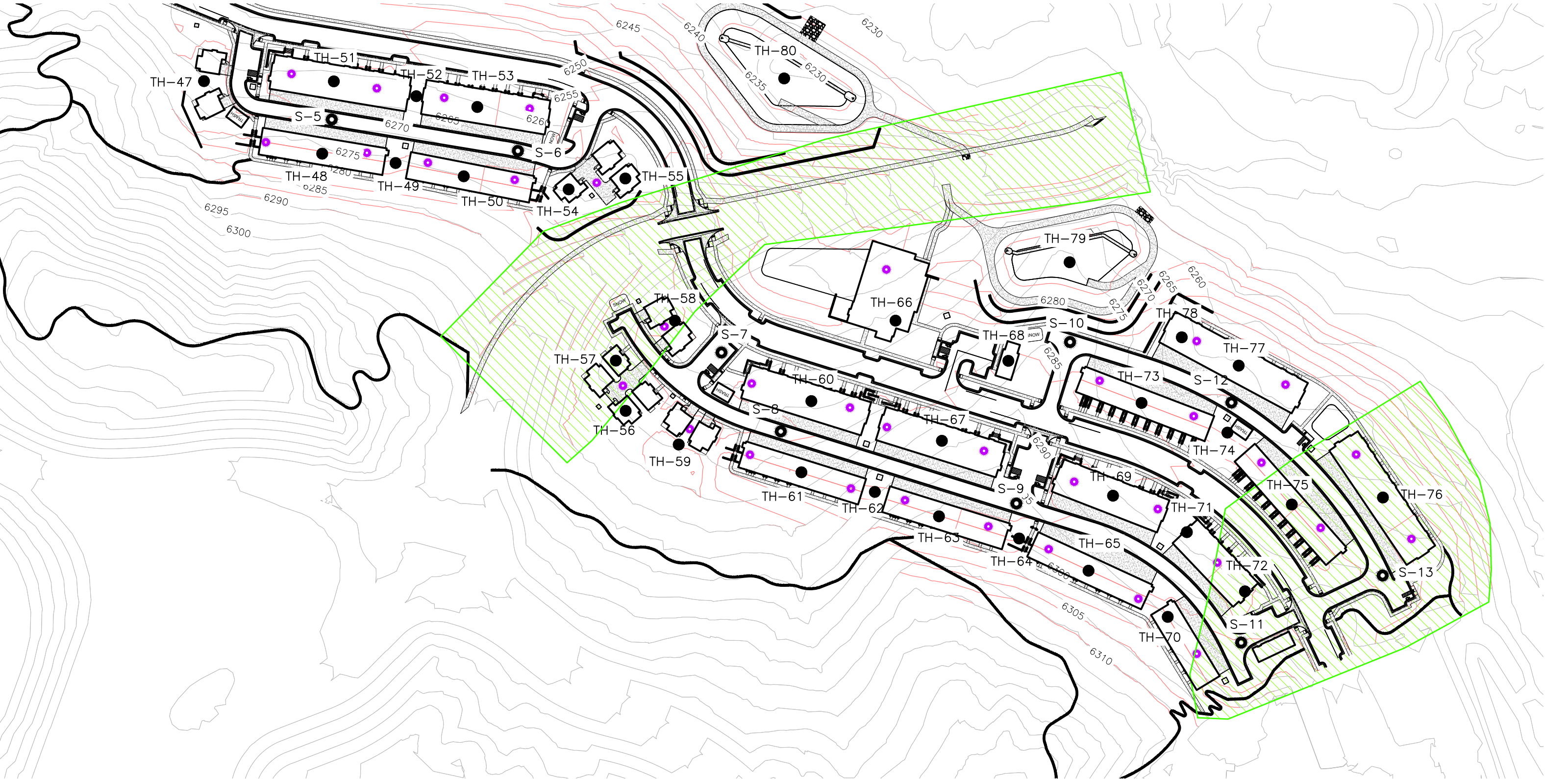


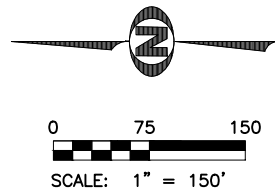
LEGEND:



- TH-47 APPROXIMATE LOCATION OF EXPLORATORY BORING
- S-5 APPROXIMATE LOCATION OF PAVEMENT BORING
- INDICATES LOCATION OF PSI BORINGS

- INDICATES EXISTING GRADE
- INDICATES PROPOSED GRADE
- ESTIMATED EXTENT OF EXISTING (UNDOCUMENTED) FILL FROM PREVIOUS ROAD DEVELOPMENT/EQUIPMENT STAGING





LEGEND:

— INDICATES EXISTING GRADE

— INDICATES PROPOSED GRADE



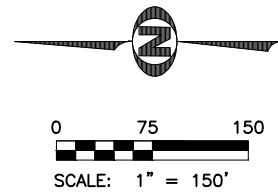
INDICATES SUB-EXCAVATION TO A DEPTH OF 5 FEET BELOW LOWEST FOUNDATION ELEMENT IS RECOMMENDED TO ALLOW THE USE OF FOOTINGS WITH MINIMUM DEADLOAD OR PTS FOUNDATIONS

INDICATES SUB-EXCAVATION TO A DEPTH OF 10 FEET BELOW LOWEST FOUNDATION ELEMENT IS RECOMMENDED TO ALLOW THE USE OF FOOTINGS WITH MINIMUM DEADLOAD OR PTS FOUNDATIONS

△ INDICATES EXISTING/UNDOCUMENTED FILL MAY BE PRESENT THAT WOULD REQUIRE MITIGATION

★ INDICATES LOT/BUILDING WITH PLANNED BASEMENT





LEGEND:

- INDICATES EXISTING GRADE
- INDICATES PROPOSED GRADE



INDICATES SUB-EXCAVATION TO A DEPTH OF 5 FEET BELOW LOWEST FOUNDATION ELEMENT IS RECOMMENDED TO ALLOW THE USE OF FOOTINGS WITH MINIMUM DEADLOAD OR PTS FOUNDATIONS



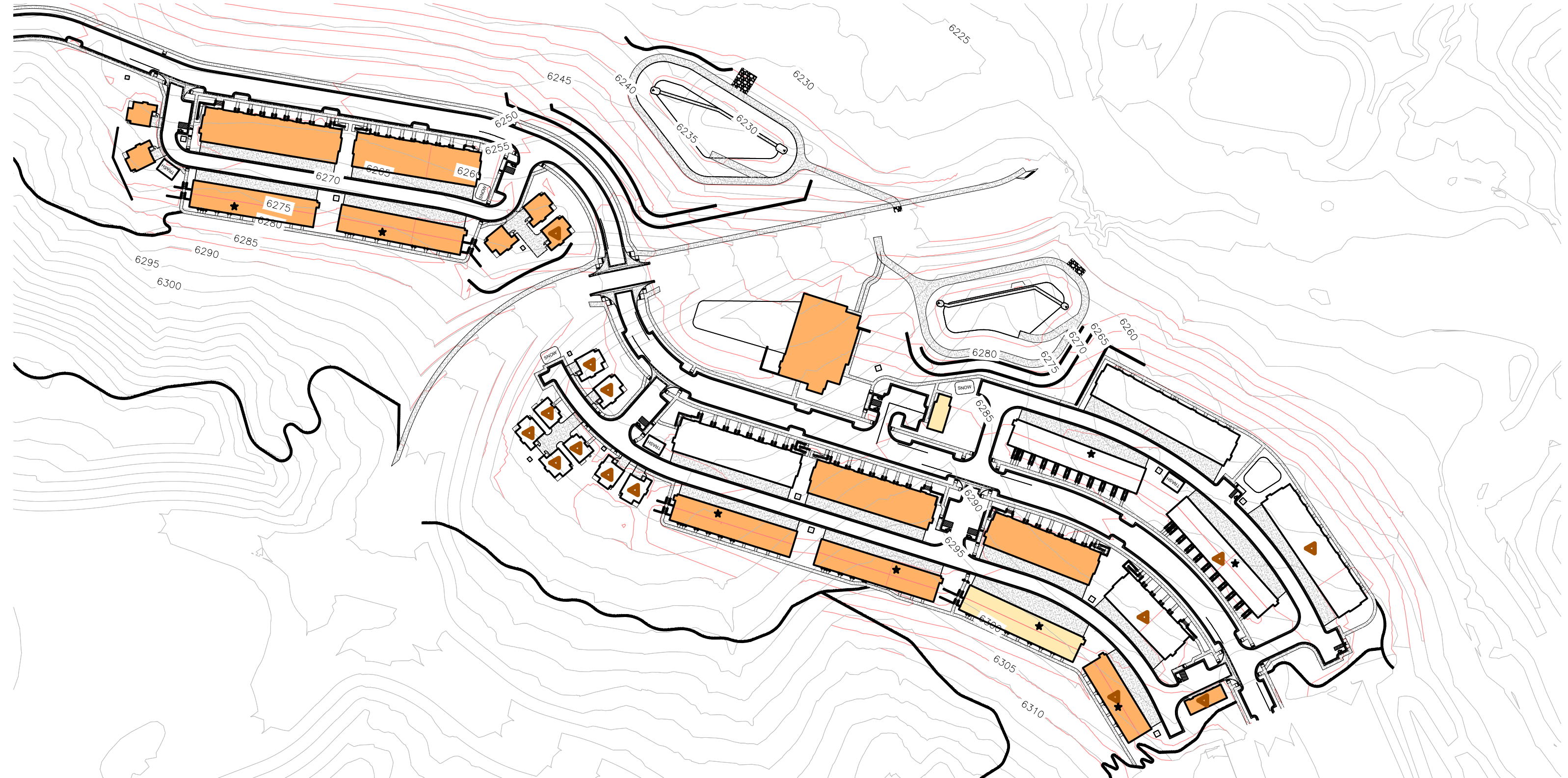
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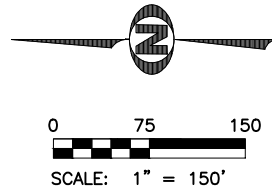


INDICATES EXISTING/UNDOCUMENTED FILL MAY BE PRESENT THAT WOULD REQUIRE MITIGATION



INDICATES LOT/BUILDING WITH PLANNED BASEMENT





LEGEND:

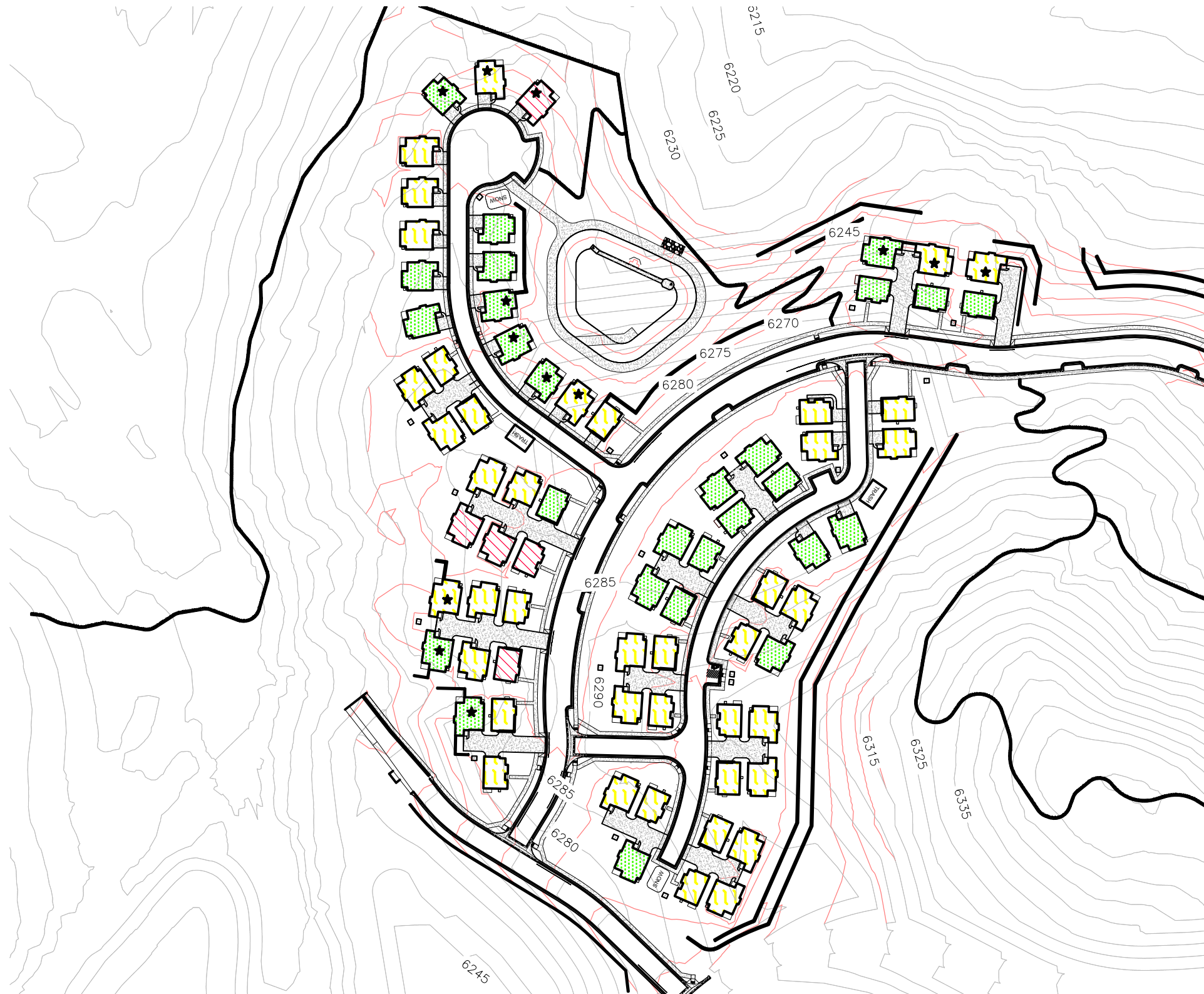
- | | |
|--|---|
| — INDICATES EXISTING GRADE |  LOW RISK |
| — INDICATES PROPOSED GRADE |  MODERATE RISK |
| ★ INDICATES LOT/BUILDING WITH PLANNED BASEMENT |  HIGH RISK |

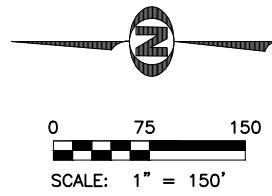
NOTES: 1) THIS ESTIMATE WAS BASED UPON A SUBJECTIVE ANALYSIS OF LABORATORY TEST RESULTS AND DRILL HOLE DATA.

2) A STRUCTURAL FLOOR SHOULD BE USED IN ALL FINISHED LIVING AREAS, INCLUDING THE GROUND FLOOR.

3) SLAB-ON-GRADE BASEMENTS FLOOR CAN BE CONSIDERED ON LOW RISK LOTS (WHERE BASEMENT ARE PLANNED) PROVIDED RISK OF MOVEMENT CAN BE TOLERATED.

4) SUB-EXCAVATION AS RECOMMENDED IN OUR REPORT AND FIGURES 2A AND 2B SHOULD REDUCE RISK TO LOW PROVIDED IT RESULTS IN LOW SWELLING FILL





LEGEND:

— INDICATES EXISTING GRADE

— INDICATES PROPOSED GRADE

★ INDICATES LOT/BUILDING WITH PLANNED BASEMENT

LOW RISK

MODERATE RISK

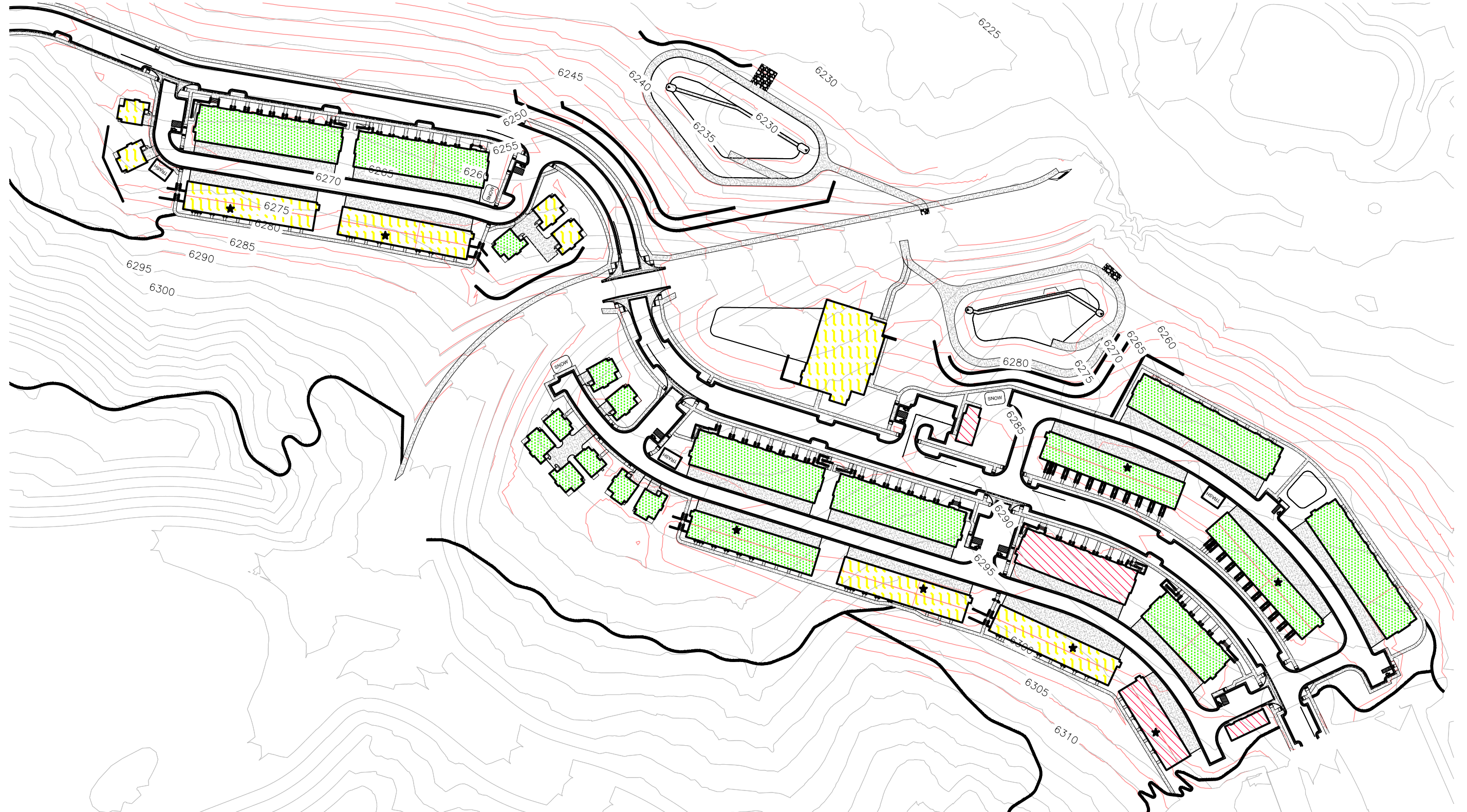
HIGH RISK

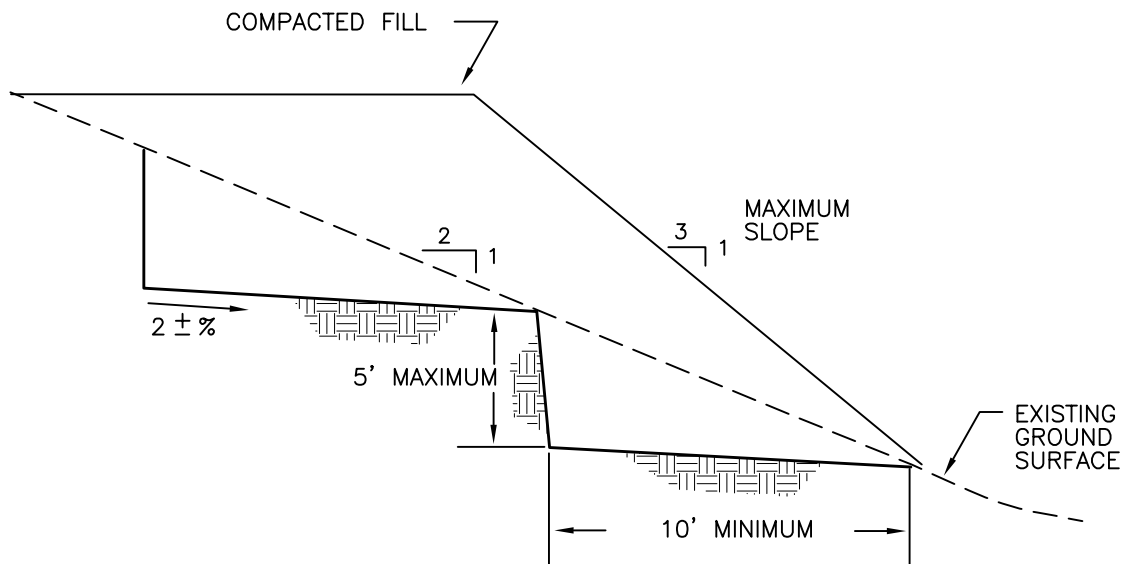
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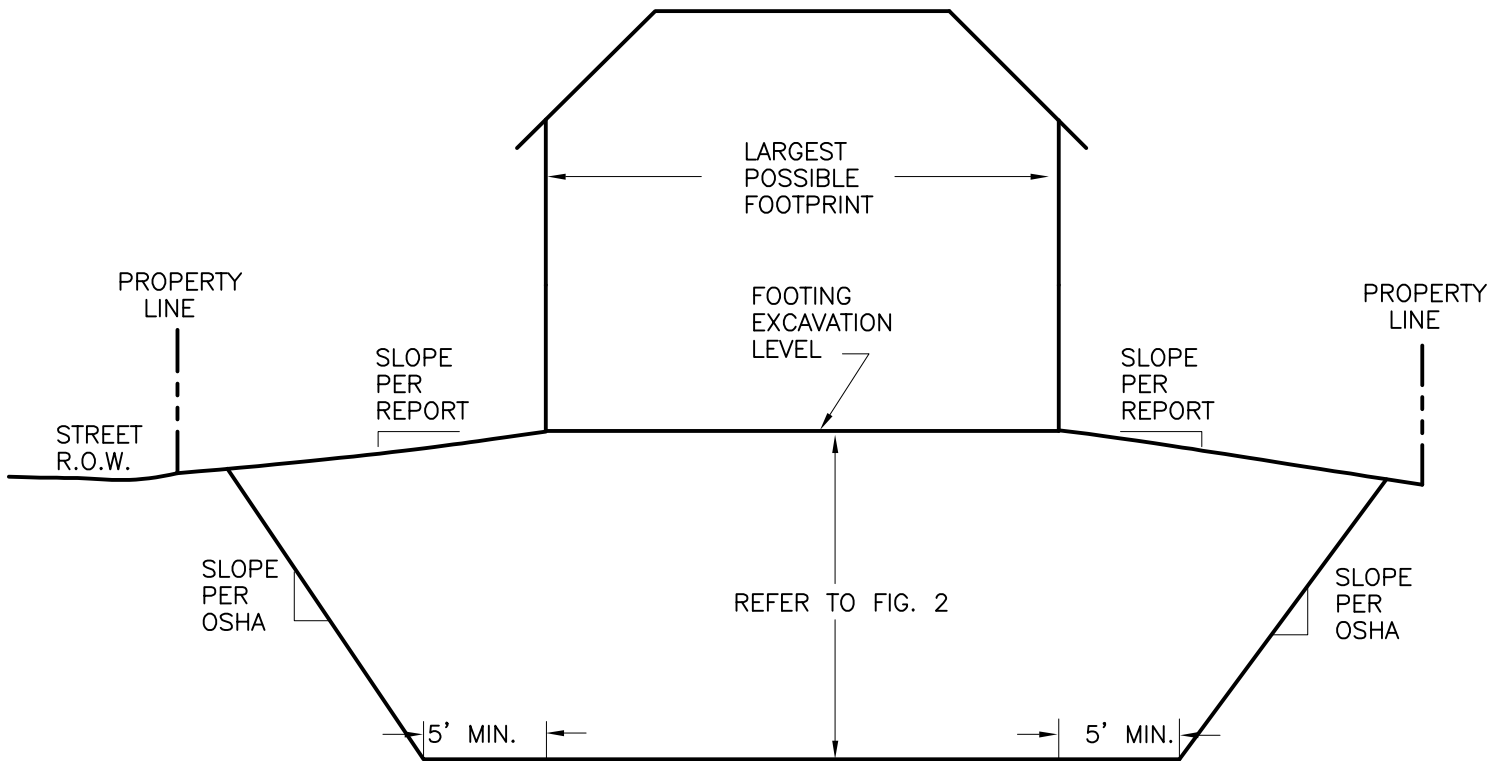
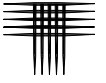




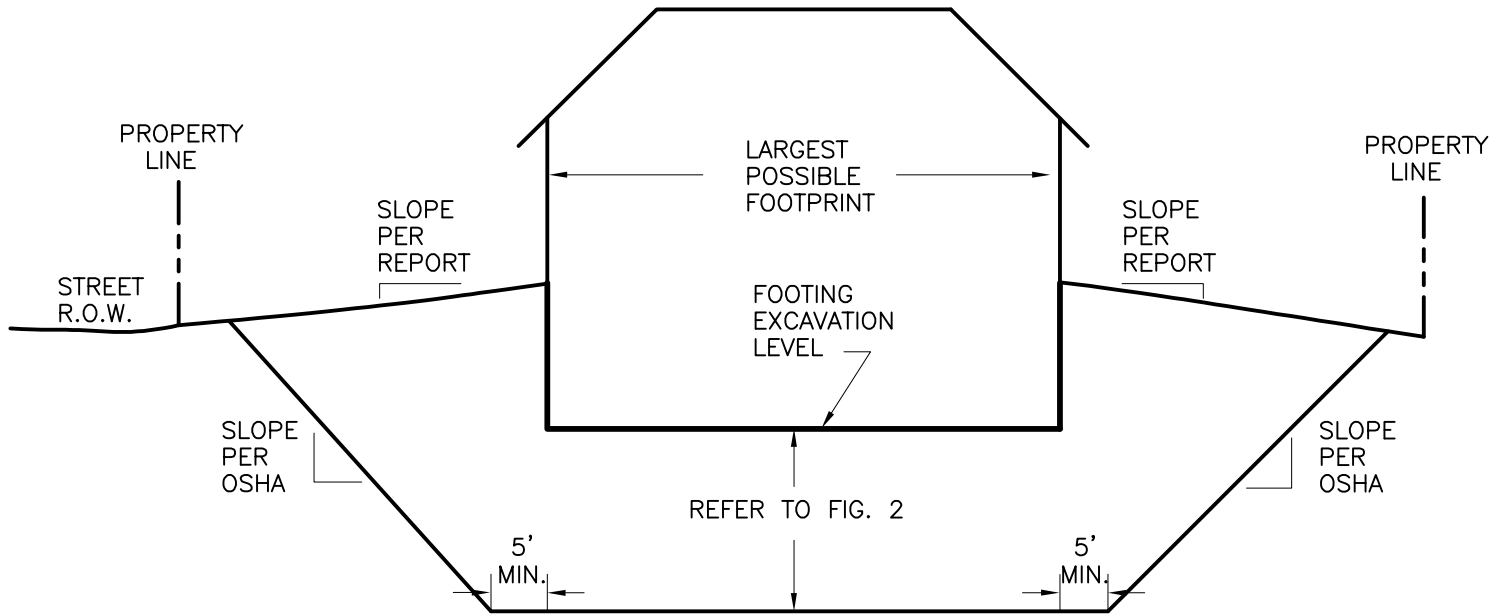
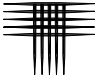
NOTES:

- 1) NATURAL SLOPES OF 20 PERCENT OR STEEPER ARE TO BE BENCHED PRIOR TO FILL PLACEMENT.
- 2) SLOPE BENCHES TO OUTSLOPE AT $2 \pm$ PERCENT.

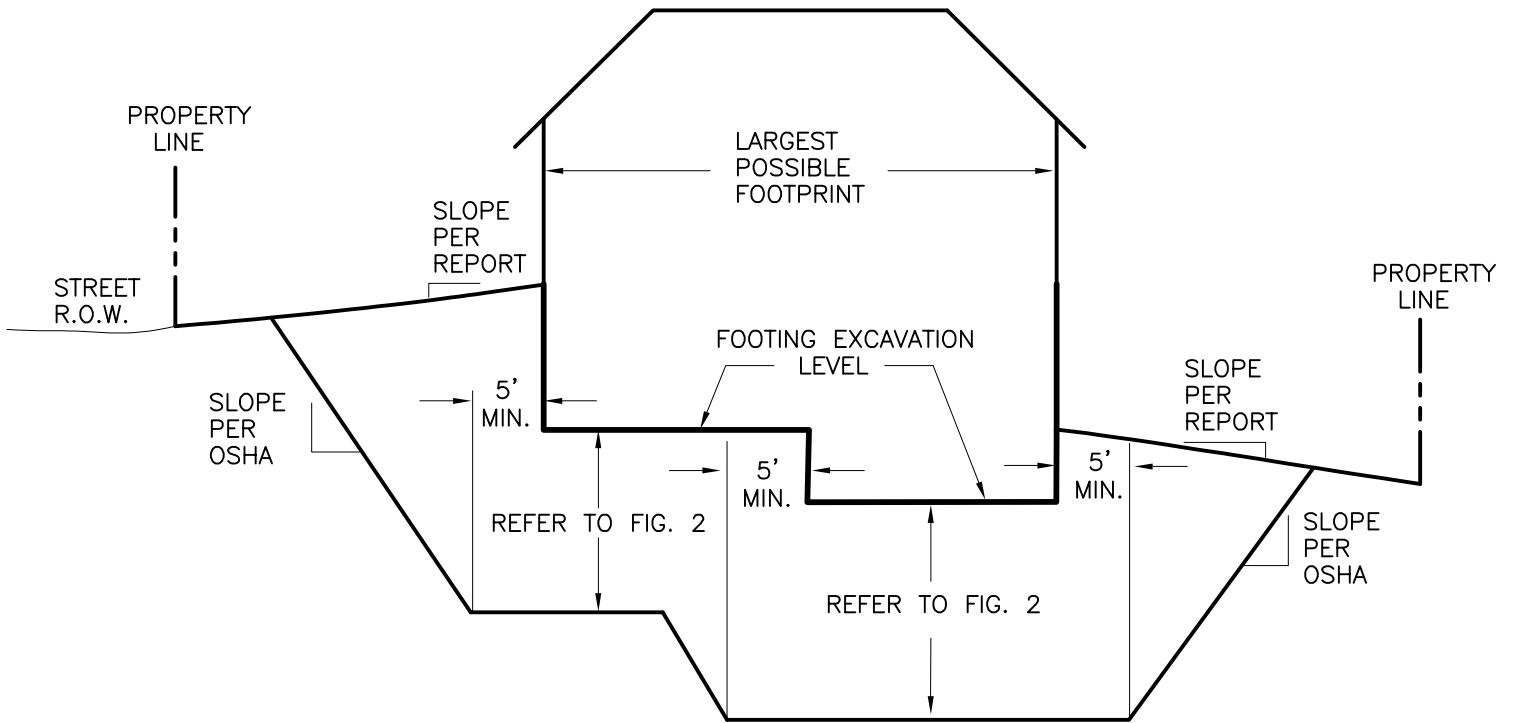
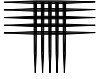
NOT TO SCALE



NOT TO SCALE



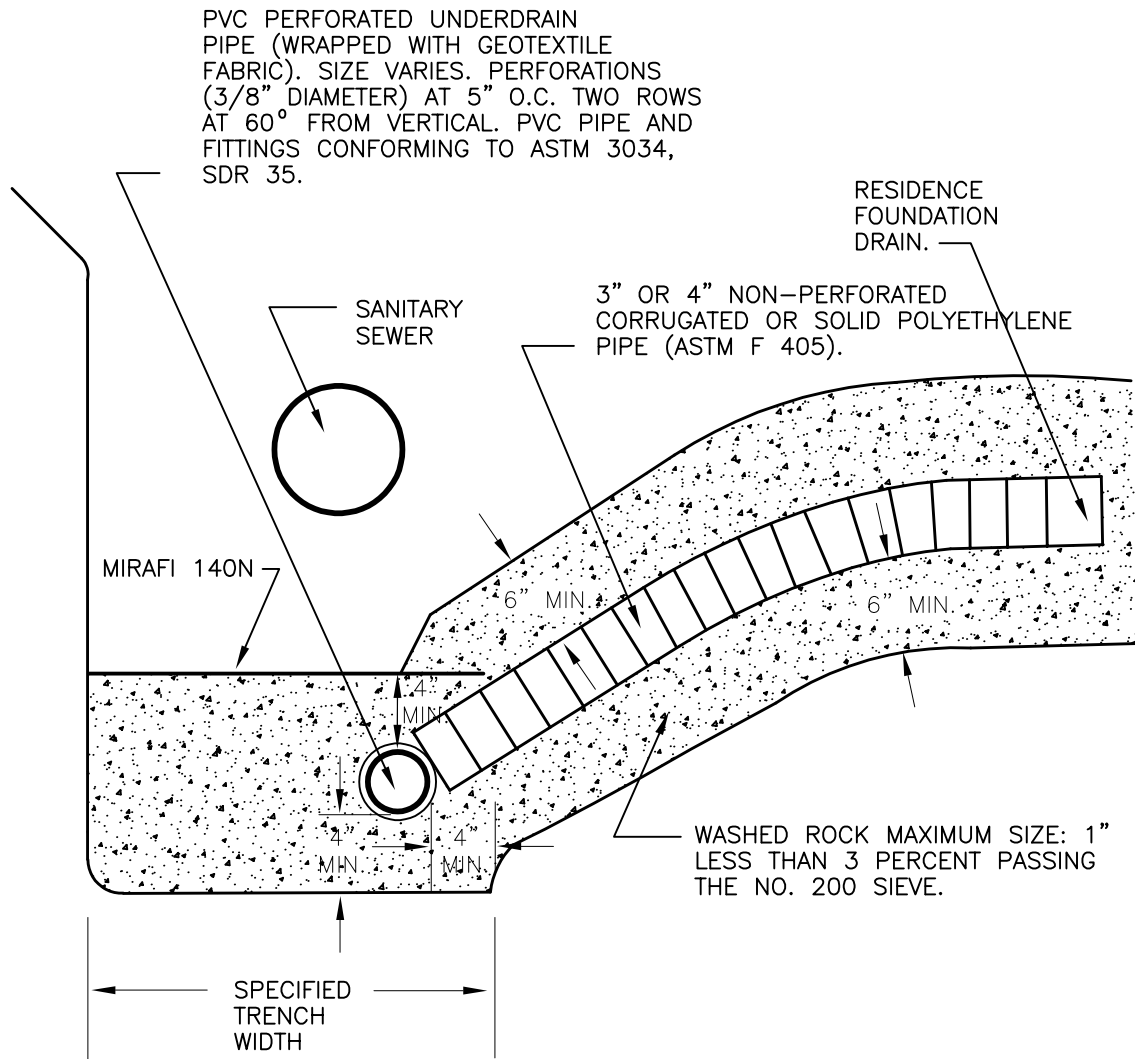
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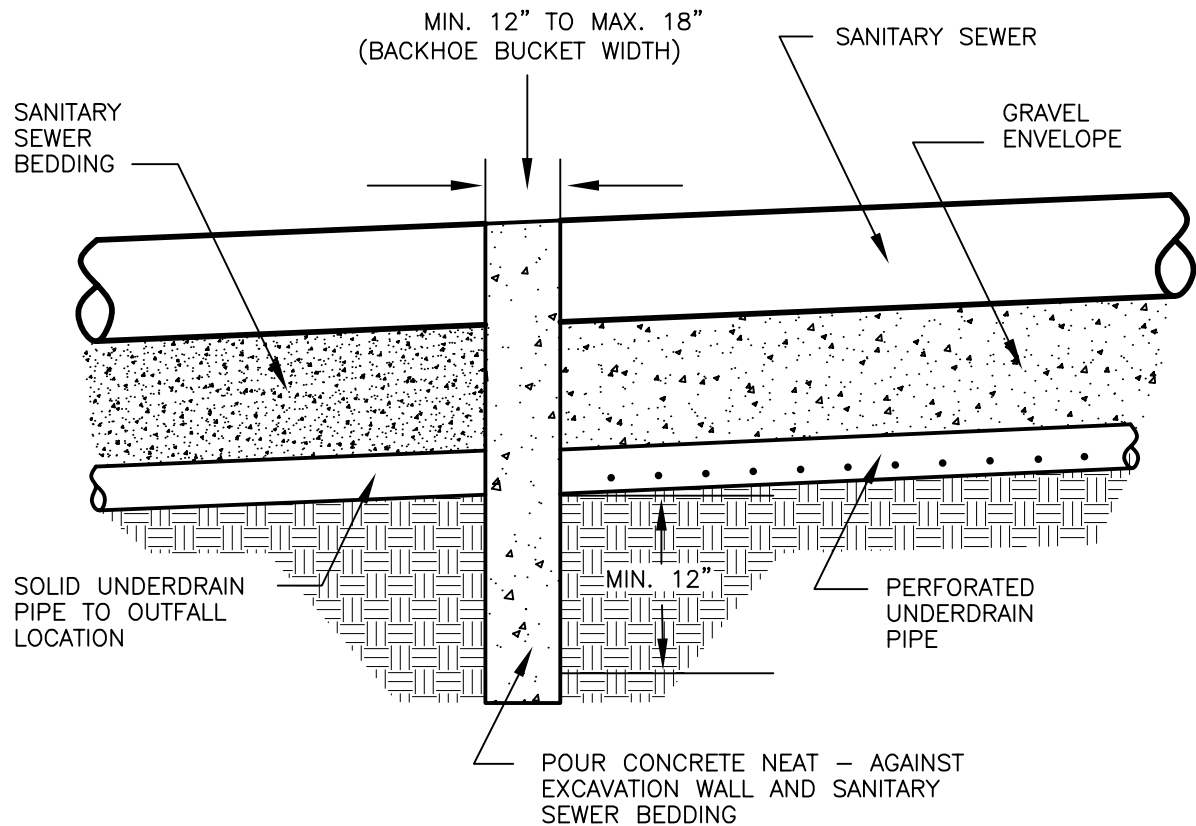
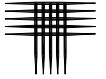
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Conceptual Sub-excavation Profile

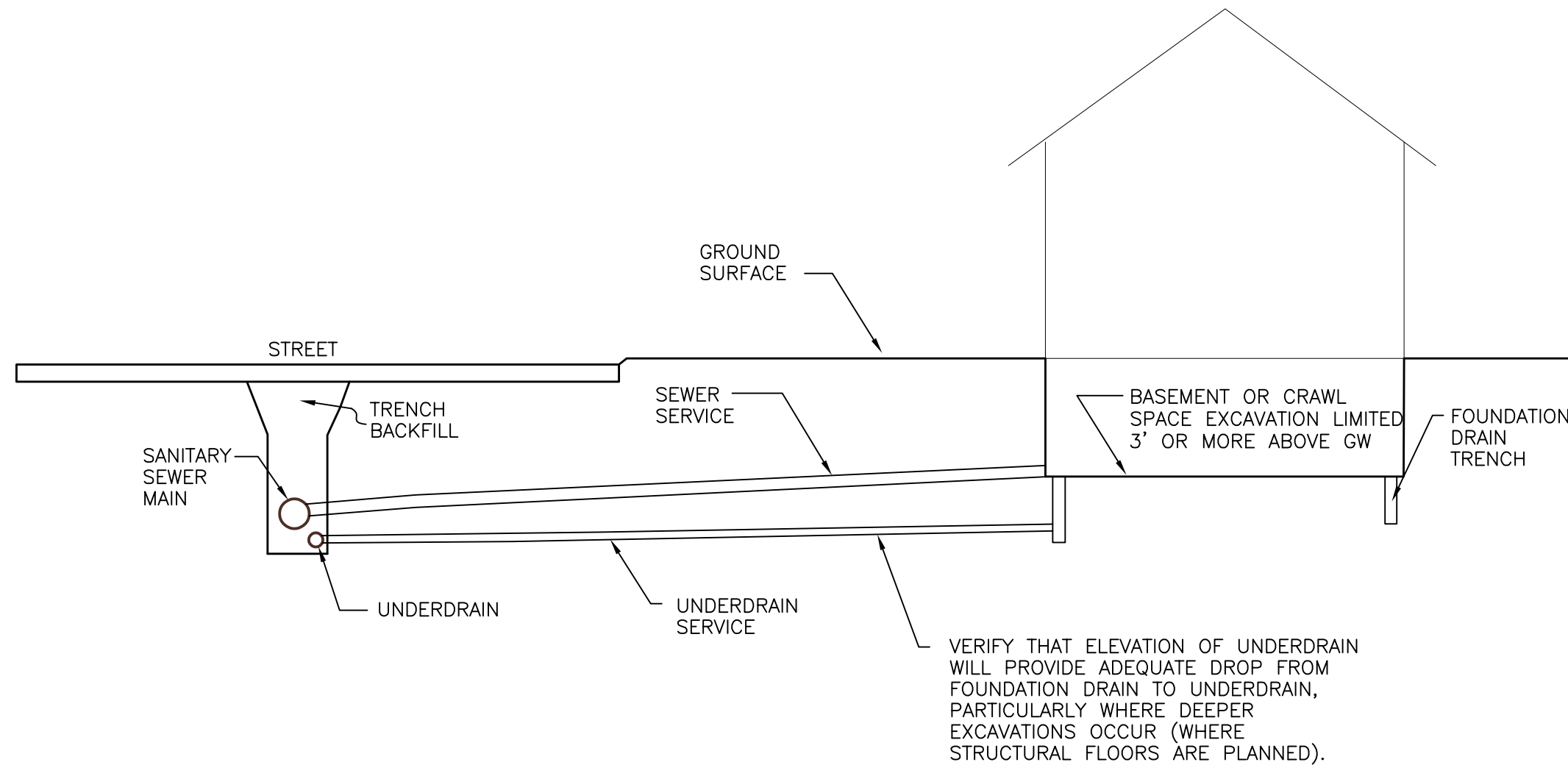
(Walk-out Basement) Fig. 7



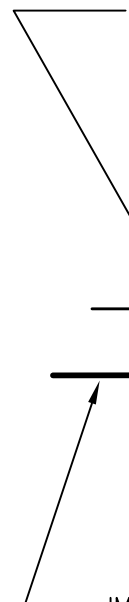
NOTE: NOT TO SCALE.

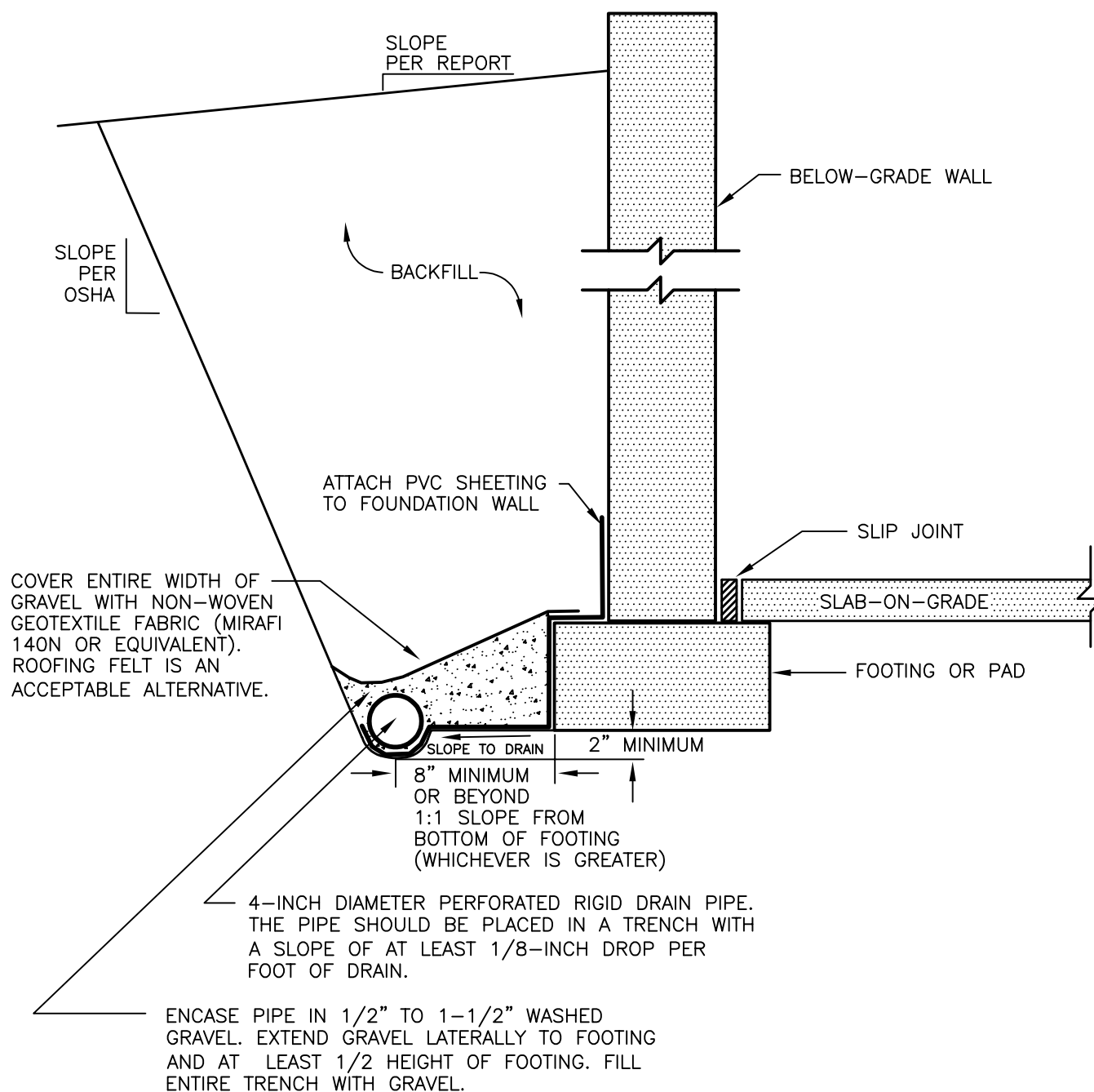
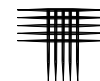


NOTE:
THE CONCRETE CUTOFF WALL SHOULD EXTEND INTO THE UNDISTURBED
SOILS OUTSIDE THE UNDERDRAIN AND SANITARY SEWER TRENCH A
MINIMUM DISTANCE OF 12 INCHES.



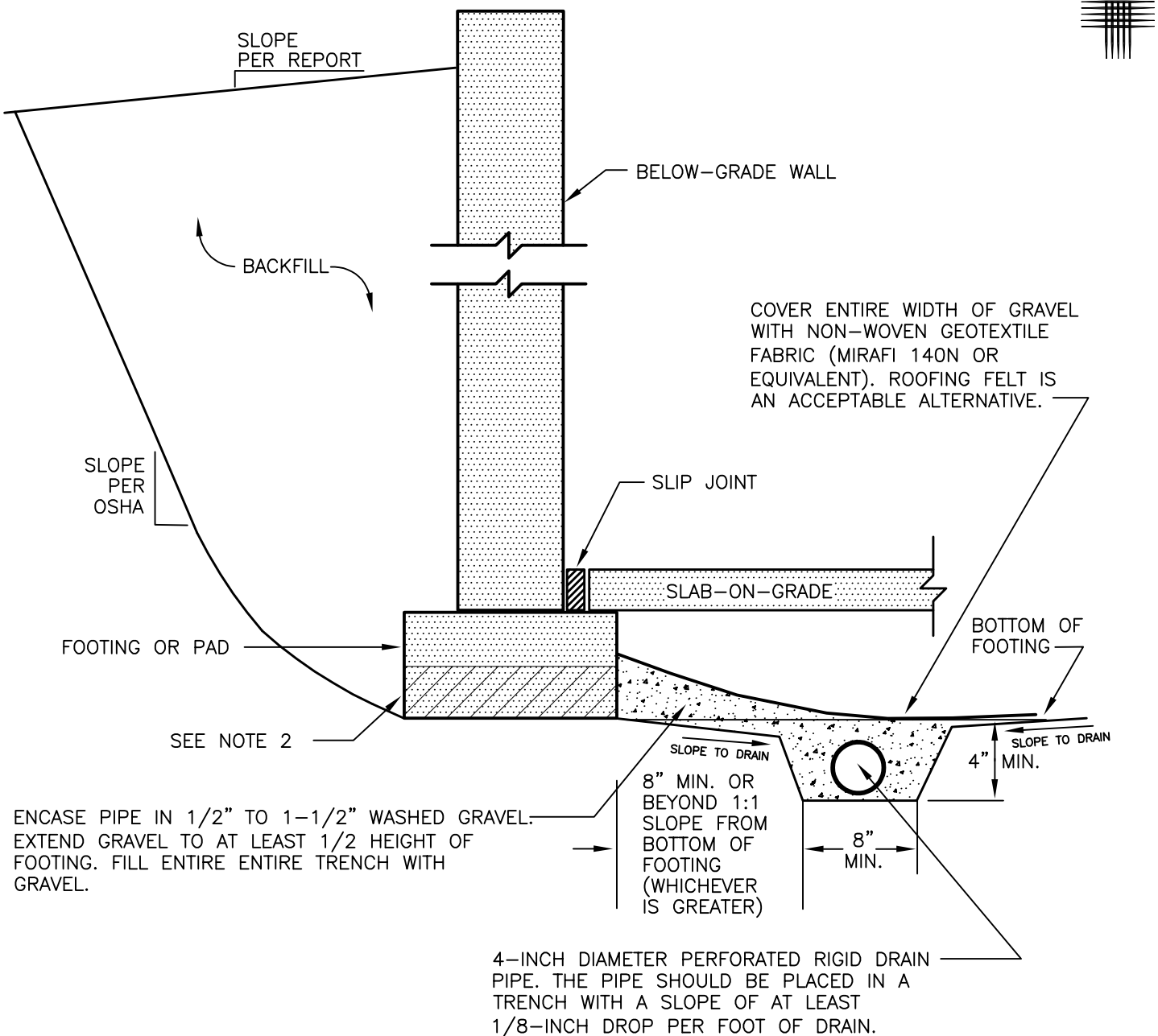
NOT TO SCALE





NOTE:

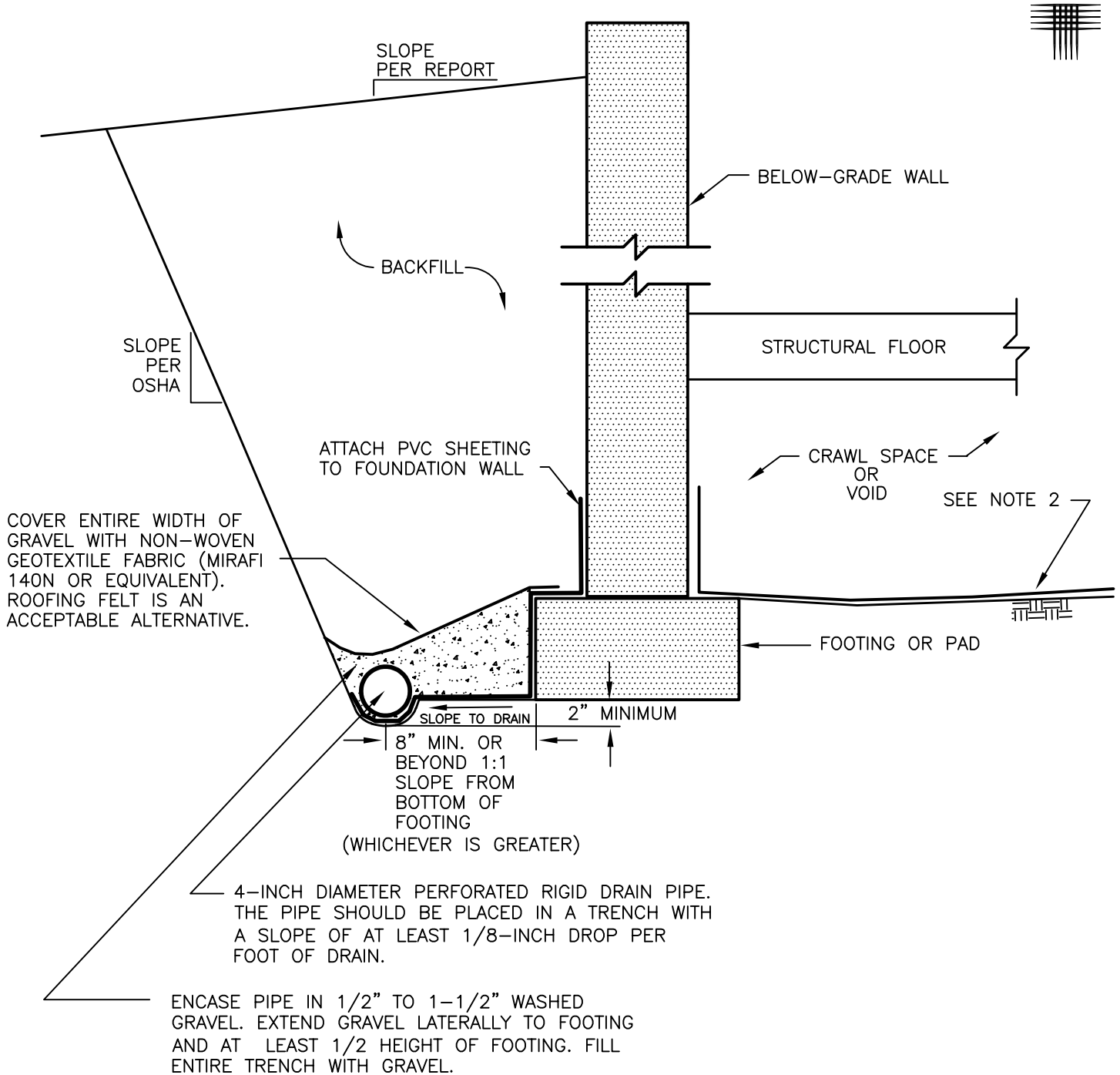
THE BOTTOM OF THE DRAIN SHOULD BE AT LEAST 2 INCHES BELOW BOTTOM OF FOOTING AT THE HIGHEST POINT AND SLOPE DOWNWARD TO A POSITIVE GRAVITY OUTLET OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPING.



NOTES:

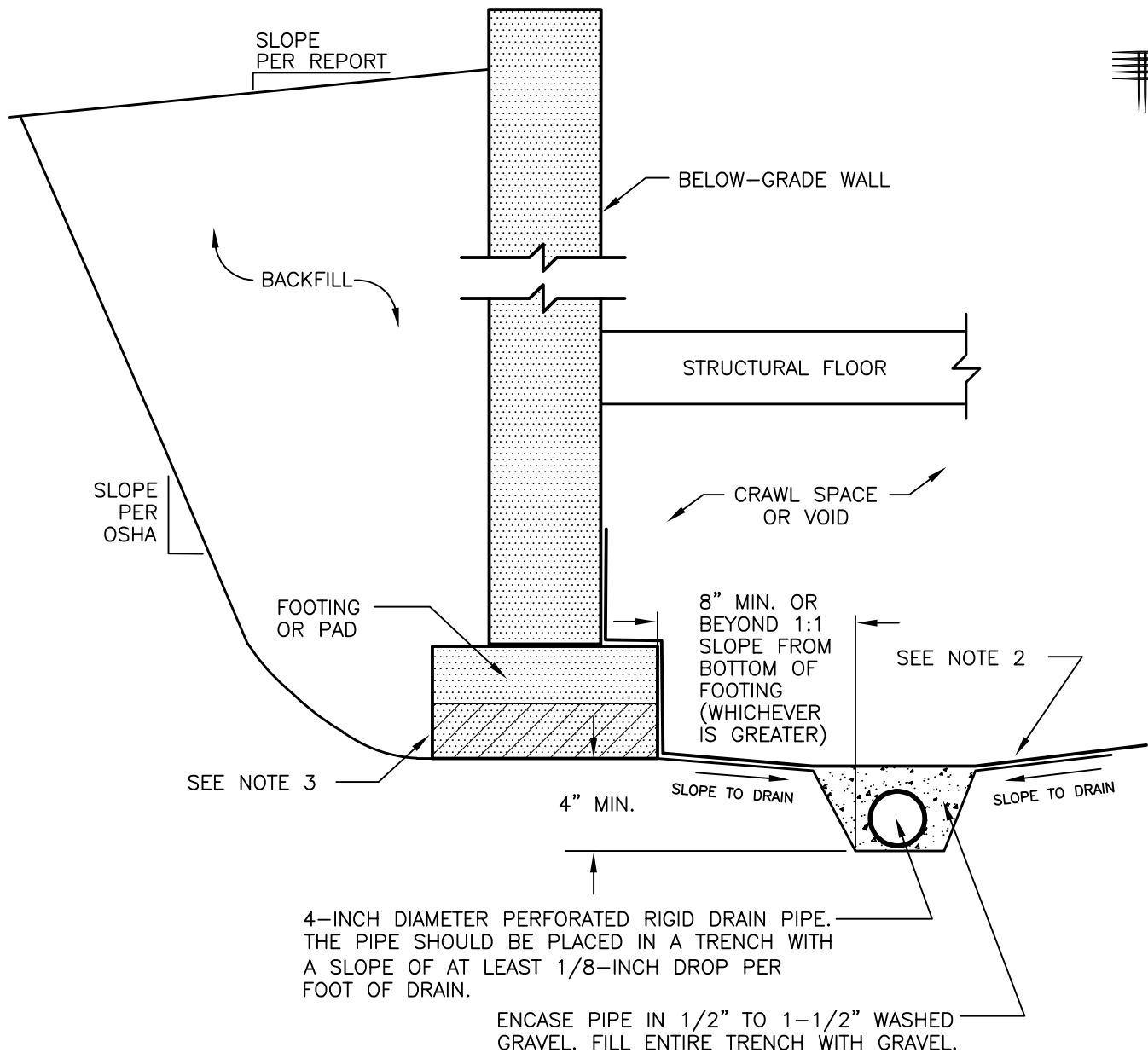
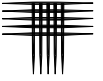
- 1) THE BOTTOM OF THE DRAIN SHOULD BE AT LEAST 4 INCHES BELOW BOTTOM OF FOOTING AT THE HIGHEST POINT AND SLOPE DOWNWARD TO A POSITIVE GRAVITY OUTLET OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPING.
- 2) FOR FOOTINGS IN BASEMENT AREAS, WE RECOMMEND PLACING AN 8-INCH THICK, 8-INCH WIDE SECTION OF VOID FORM PERPENDICULAR TO THE FOOTING ABOUT EVERY 10 TO 15 FEET AND AT WINDOW WELLS, 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



NOTES:

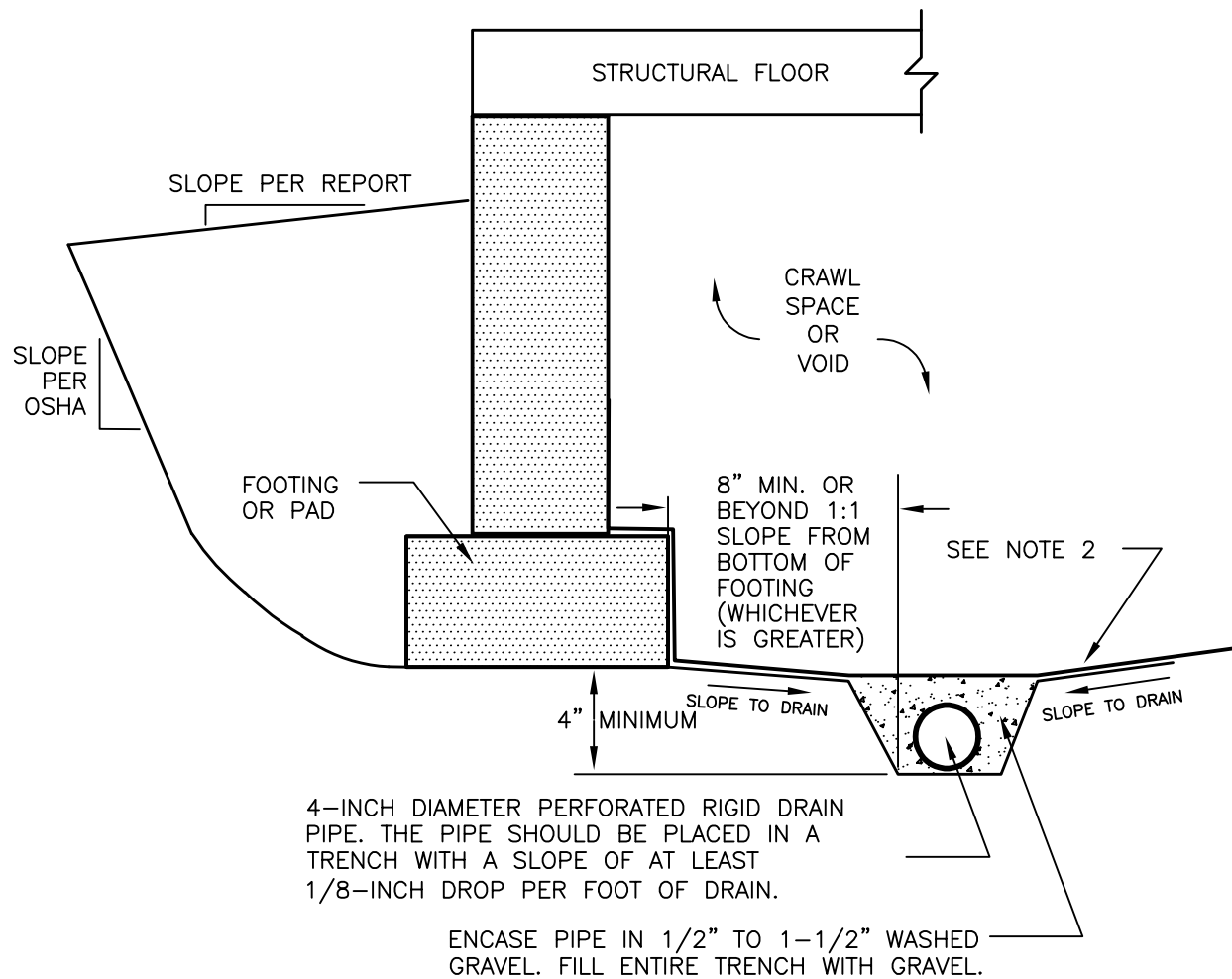
- 1) THE BOTTOM OF THE DRAIN SHOULD BE AT LEAST 2 INCHES BELOW BOTTOM OF FOOTING AT THE HIGHEST POINT AND SLOPE DOWNWARD TO A POSITIVE GRAVITY OUTLET OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPING.
- 2) TO HELP CONTROL THE HUMIDITY IN THE CRAWL SPACE, A MINIMUM 10-MIL POLYETHYLENE VAPOR RETARDER 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.



NOTES:

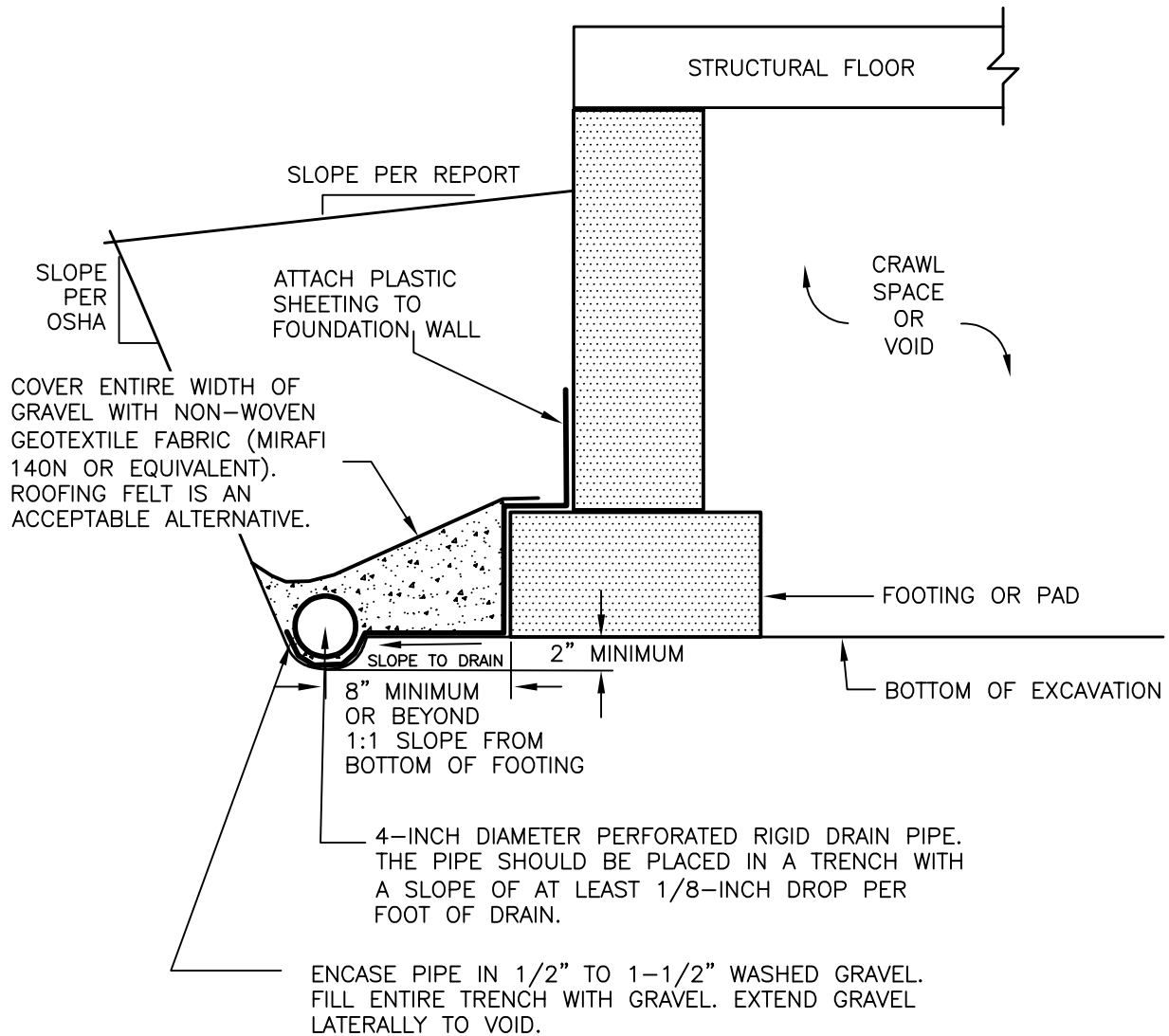
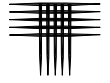
- 1) THE BOTTOM OF THE DRAIN SHOULD BE AT LEAST 4 INCHES BELOW BOTTOM OF FOOTING AT THE HIGHEST POINT AND SLOPE DOWNWARD TO A POSITIVE GRAVITY OUTLET OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPING.
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- 3) FOR FOOTINGS IN BASEMENT AREAS, WE RECOMMEND PLACING AN 8-INCH THICK, 8-INCH WIDE SECTION OF VOID FORM PERPENDICULAR TO FOOTING ABOUT EVERY 10 TO 15 FEET AND AT WINDOW WELLS, 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



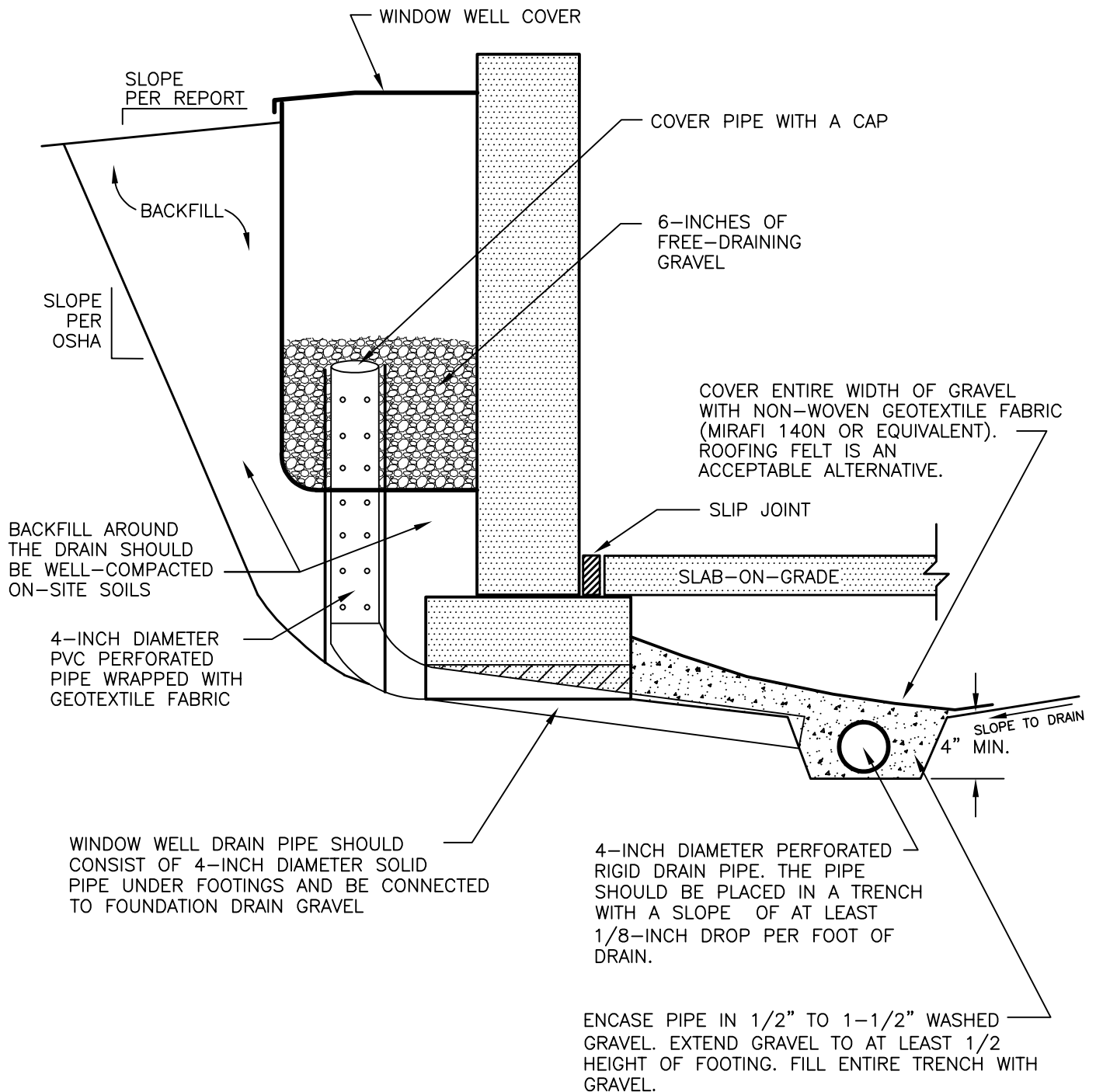
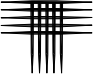
NOTES:

- 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.



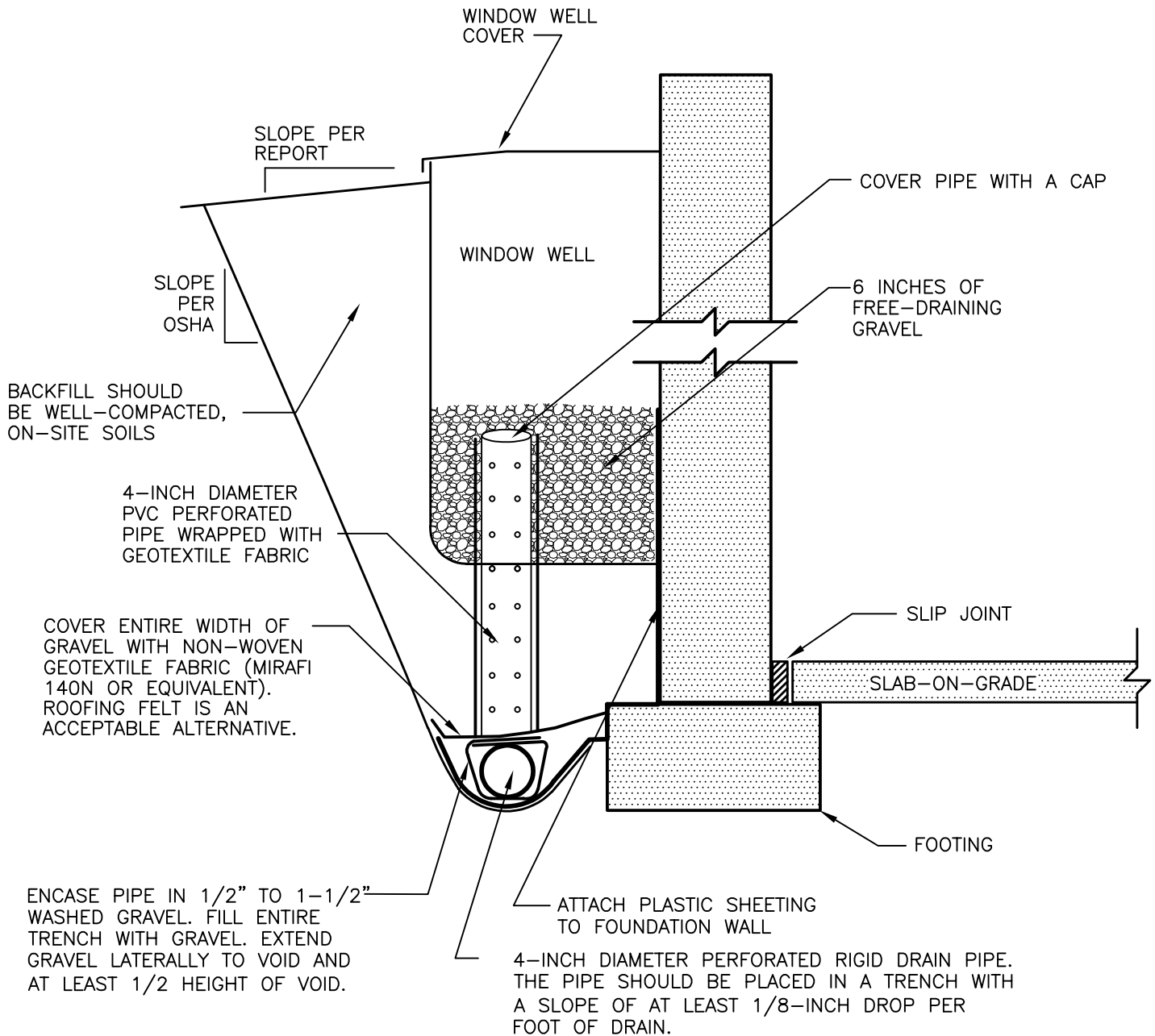
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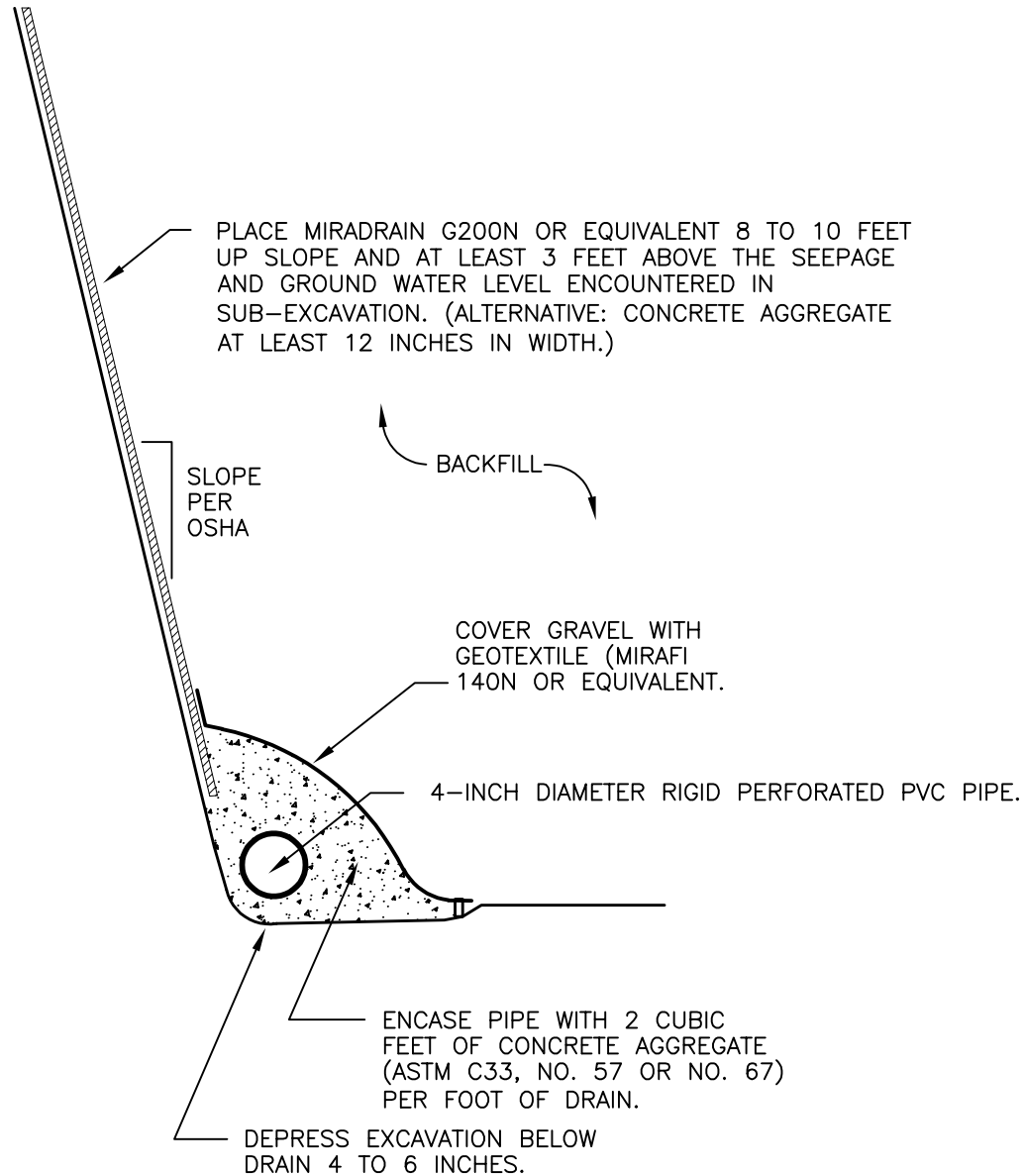
THE BOTTOM OF THE DRAIN SHOULD BE AT LEAST 2 INCHES BELOW BOTTOM OF FOOTING AT THE HIGHEST POINT AND SLOPE DOWNWARD TO A POSITIVE GRAVITY OUTLET OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPING.



NOTE:

THE BOTTOM OF THE INTERIOR 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.





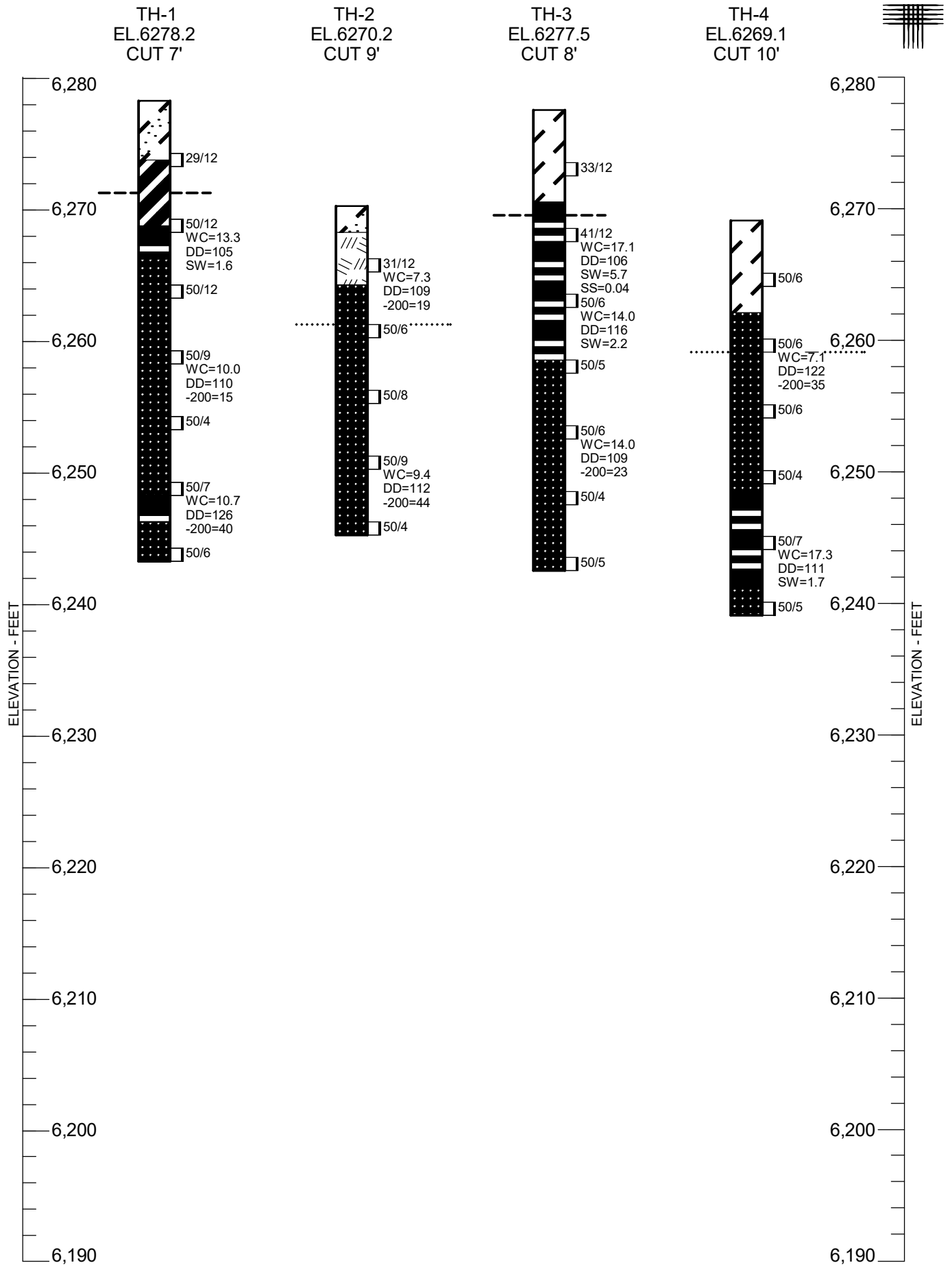
NOTE:
THE BOTTOM OF THE DRAIN SHOULD SLOPE TO A POSITIVE GRAVITY OUTLET WITH A MINIMUM SLOPE OF 0.5 PERCENT.

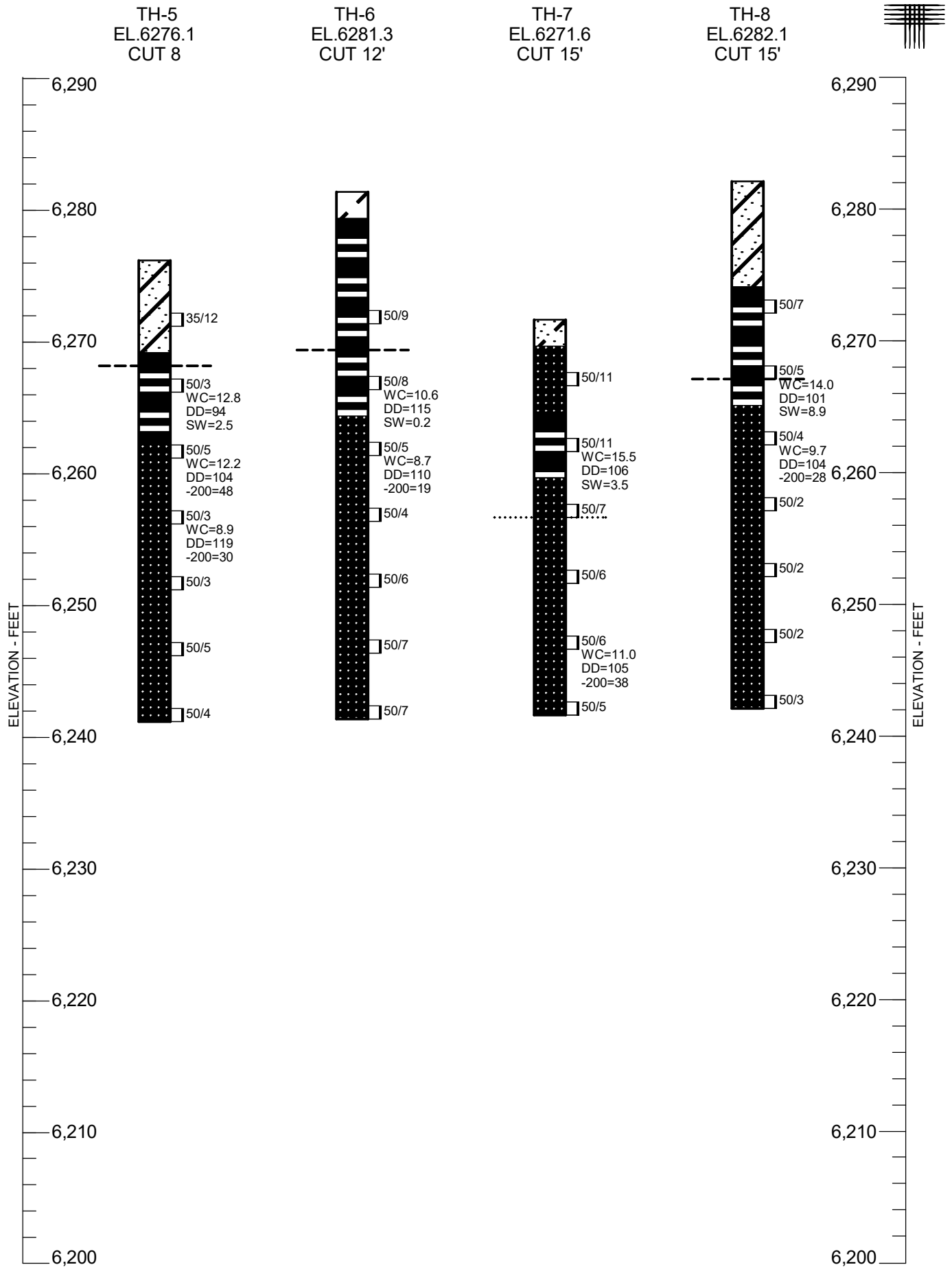
Sub-Excavation Interceptor Drain

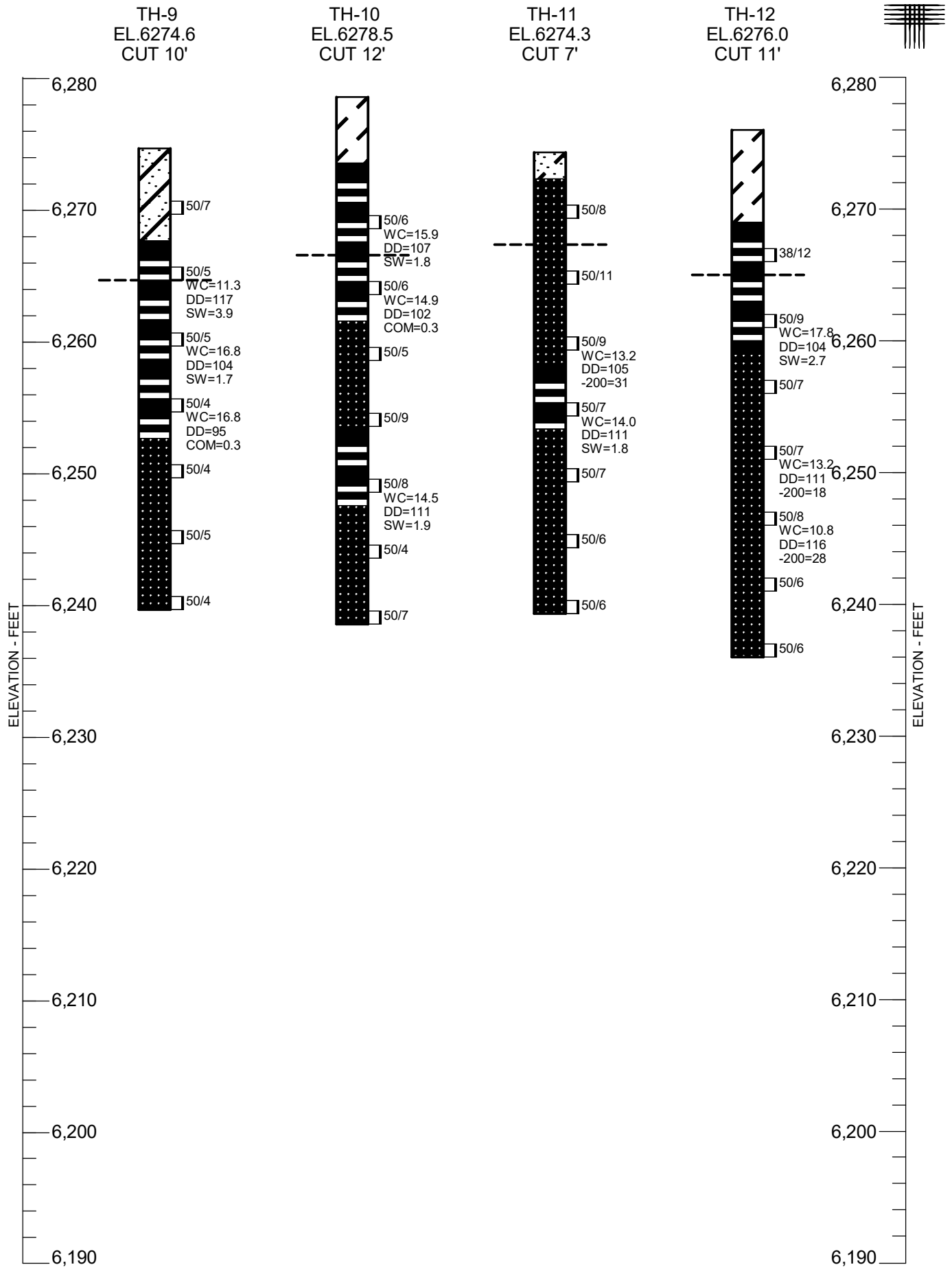


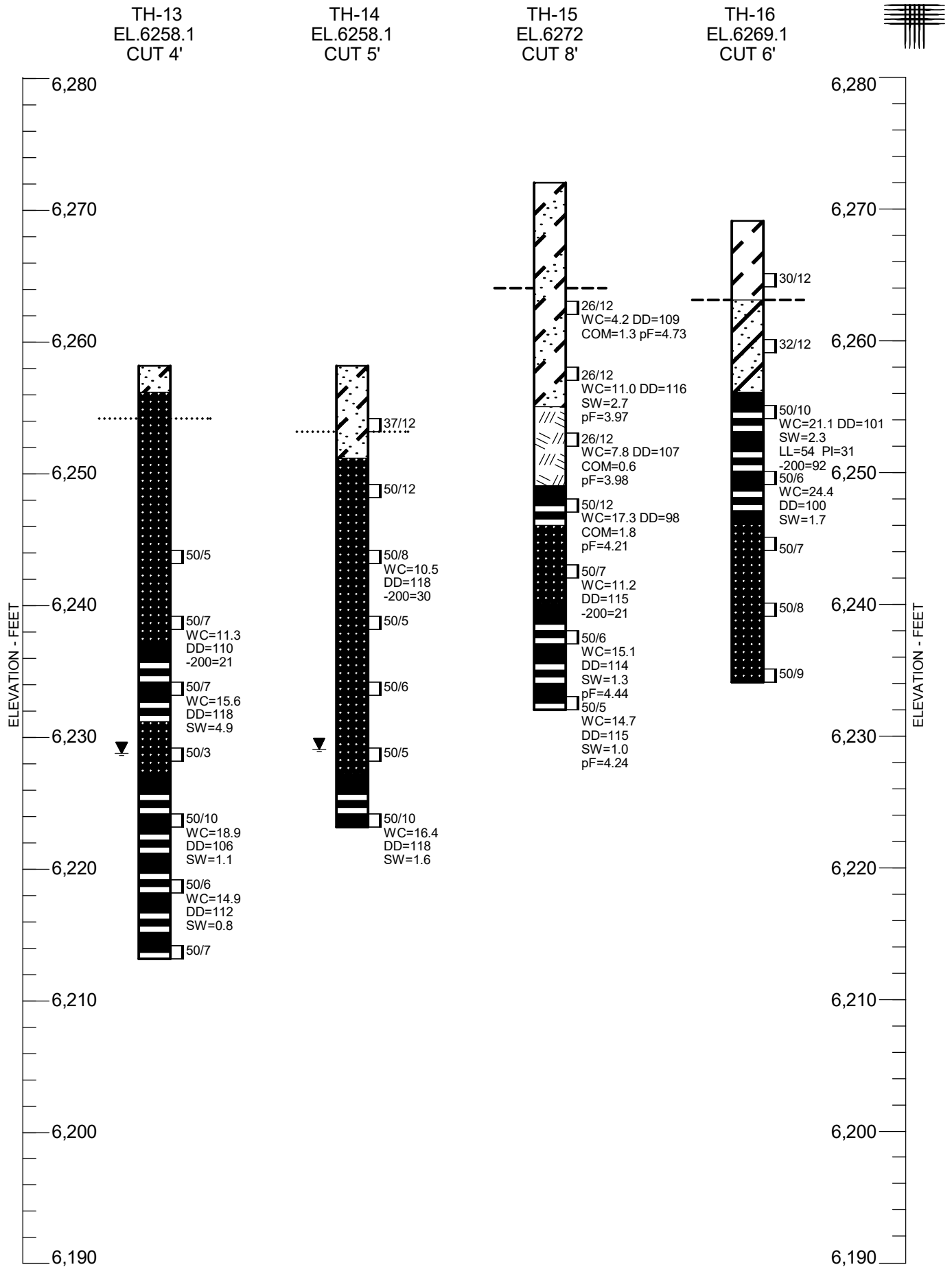
APPENDIX A

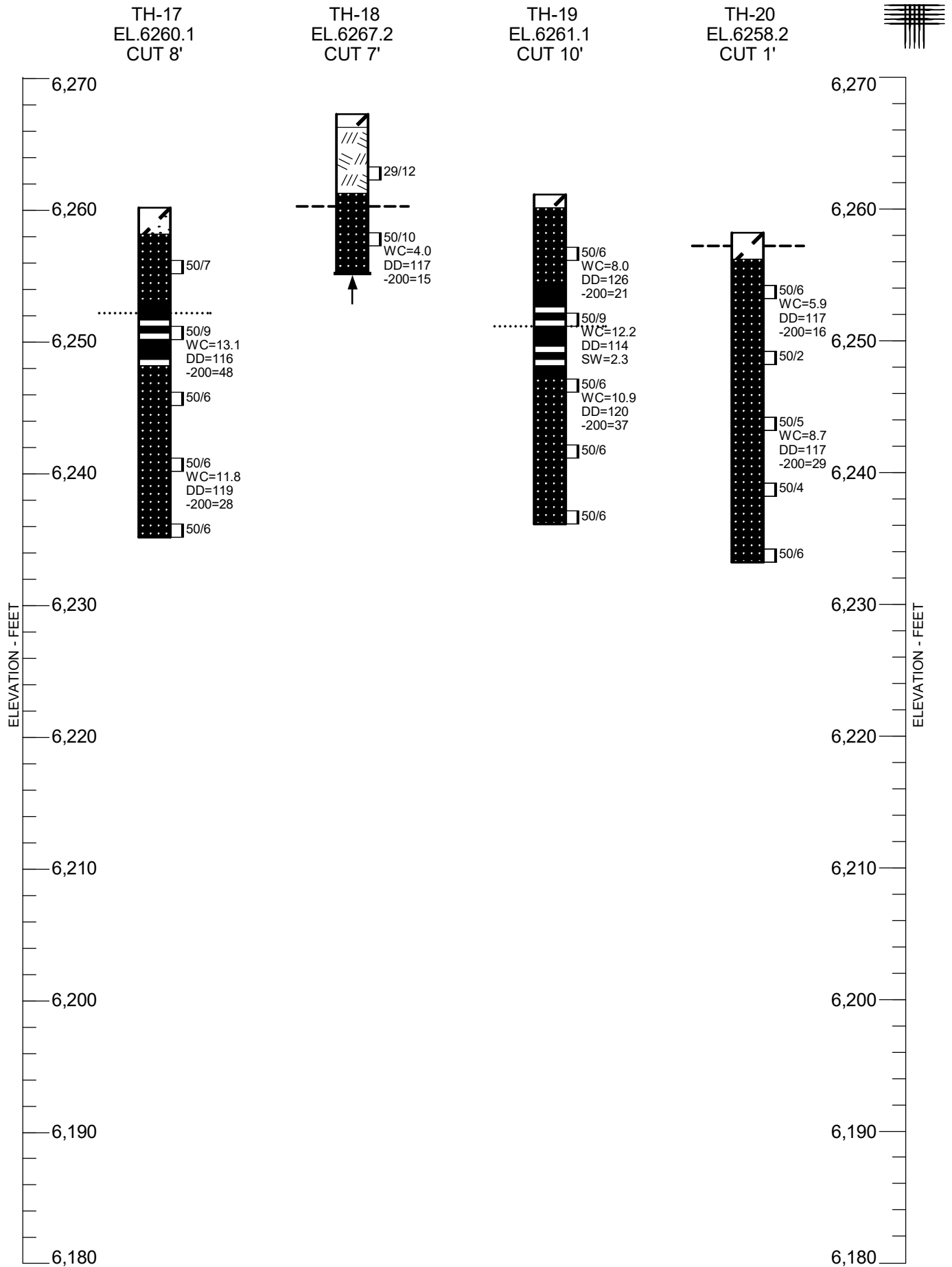
SUMMARY LOGS OF EXPLORATORY BORINGS

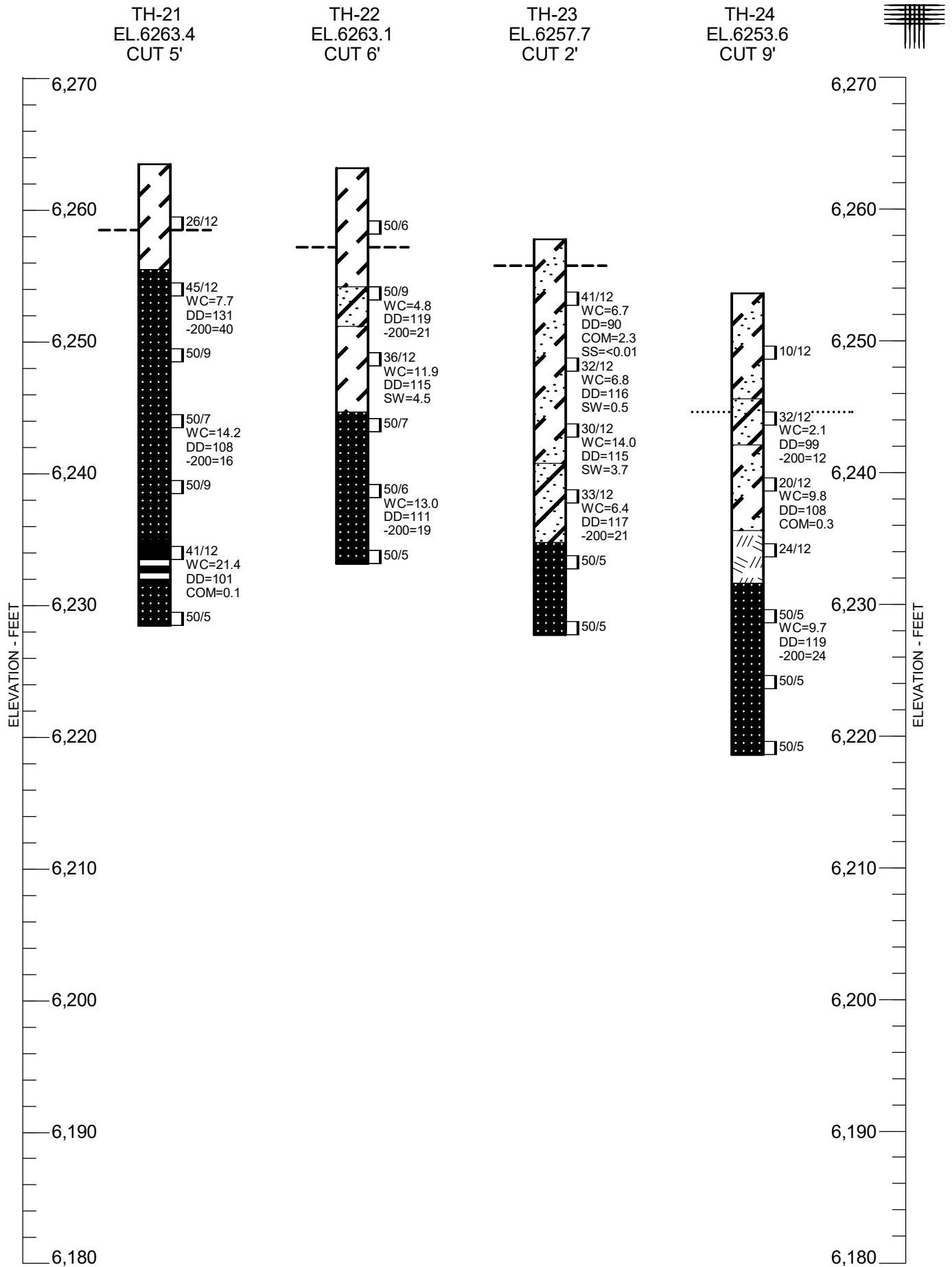


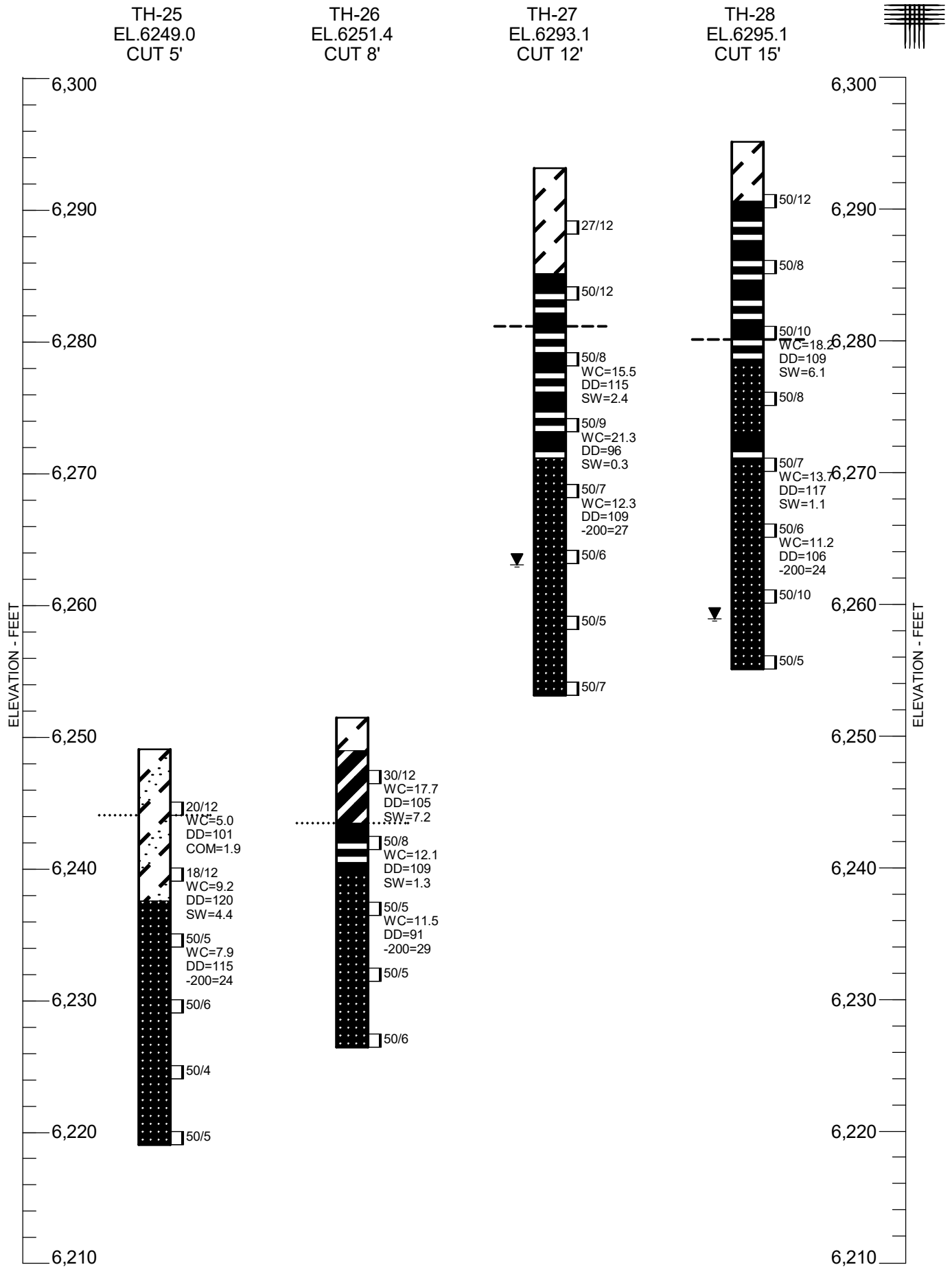


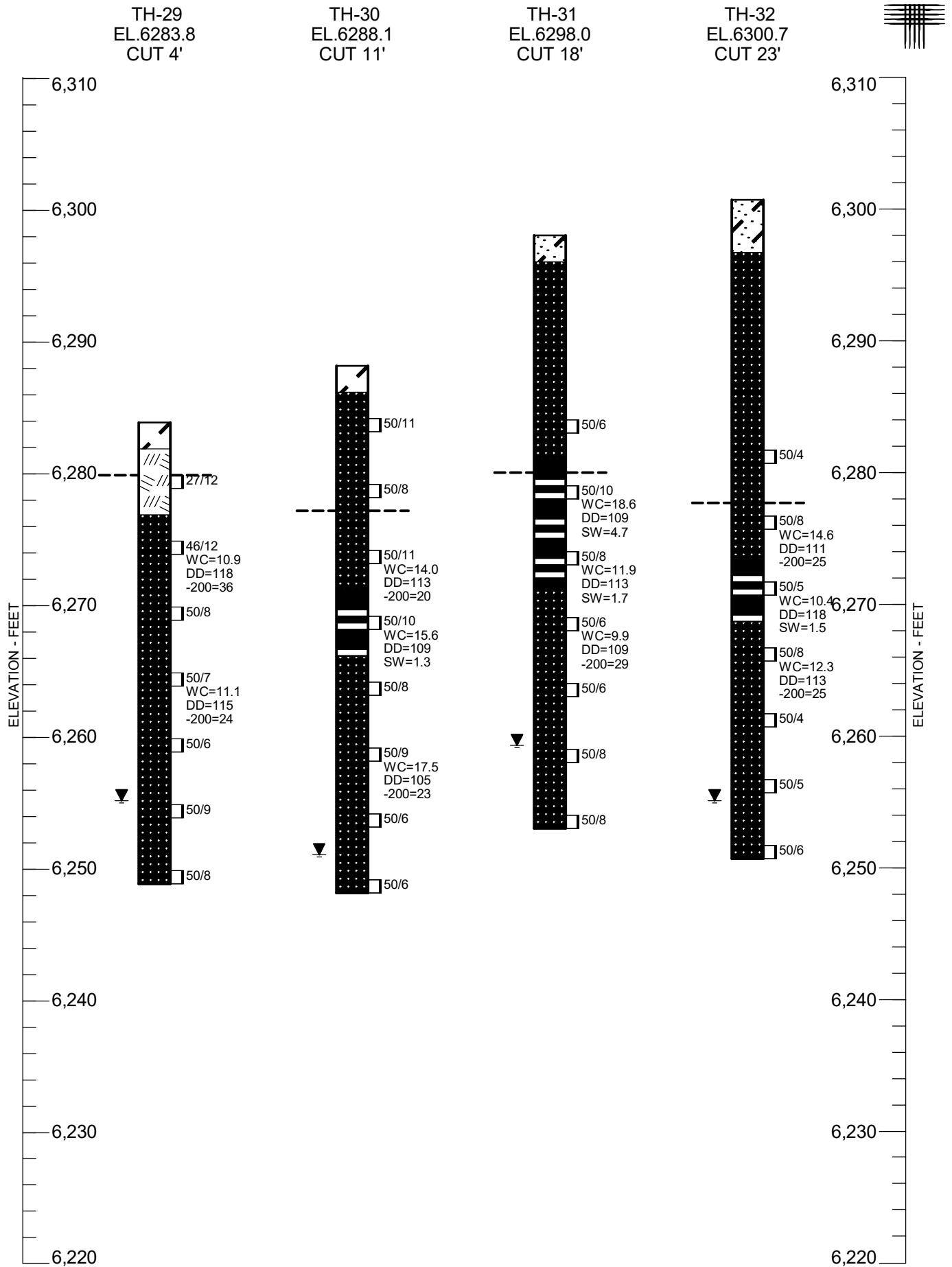


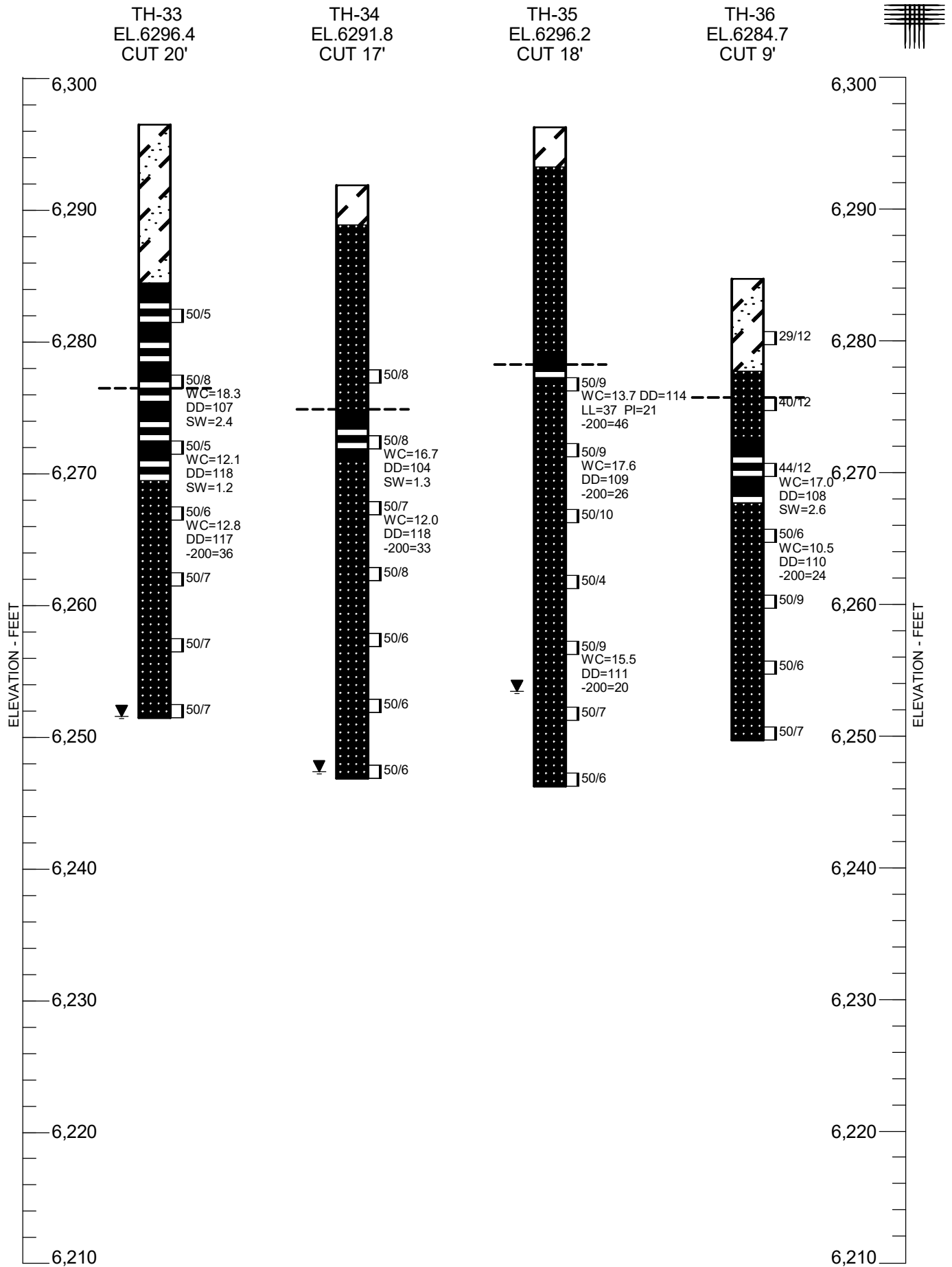


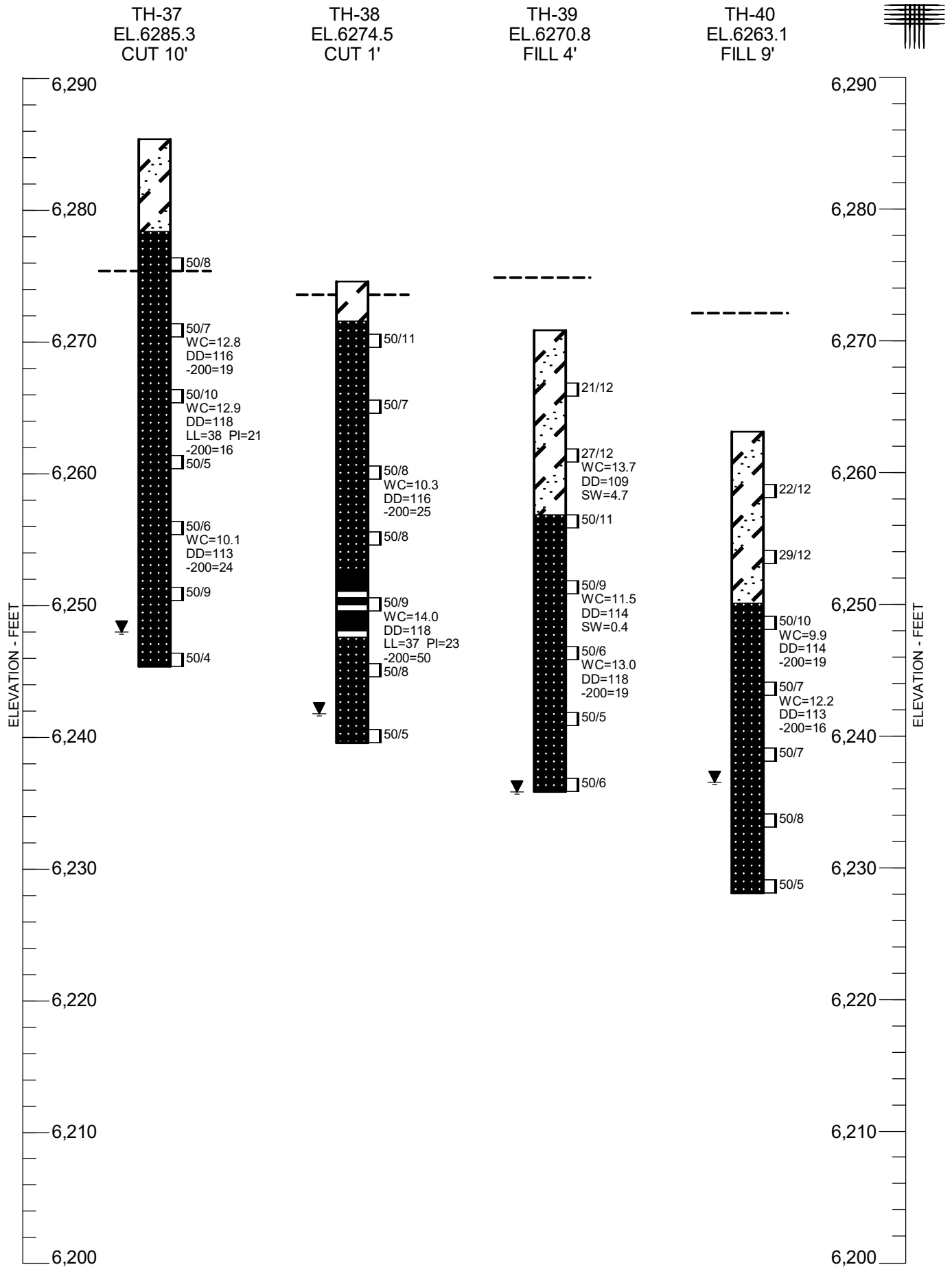








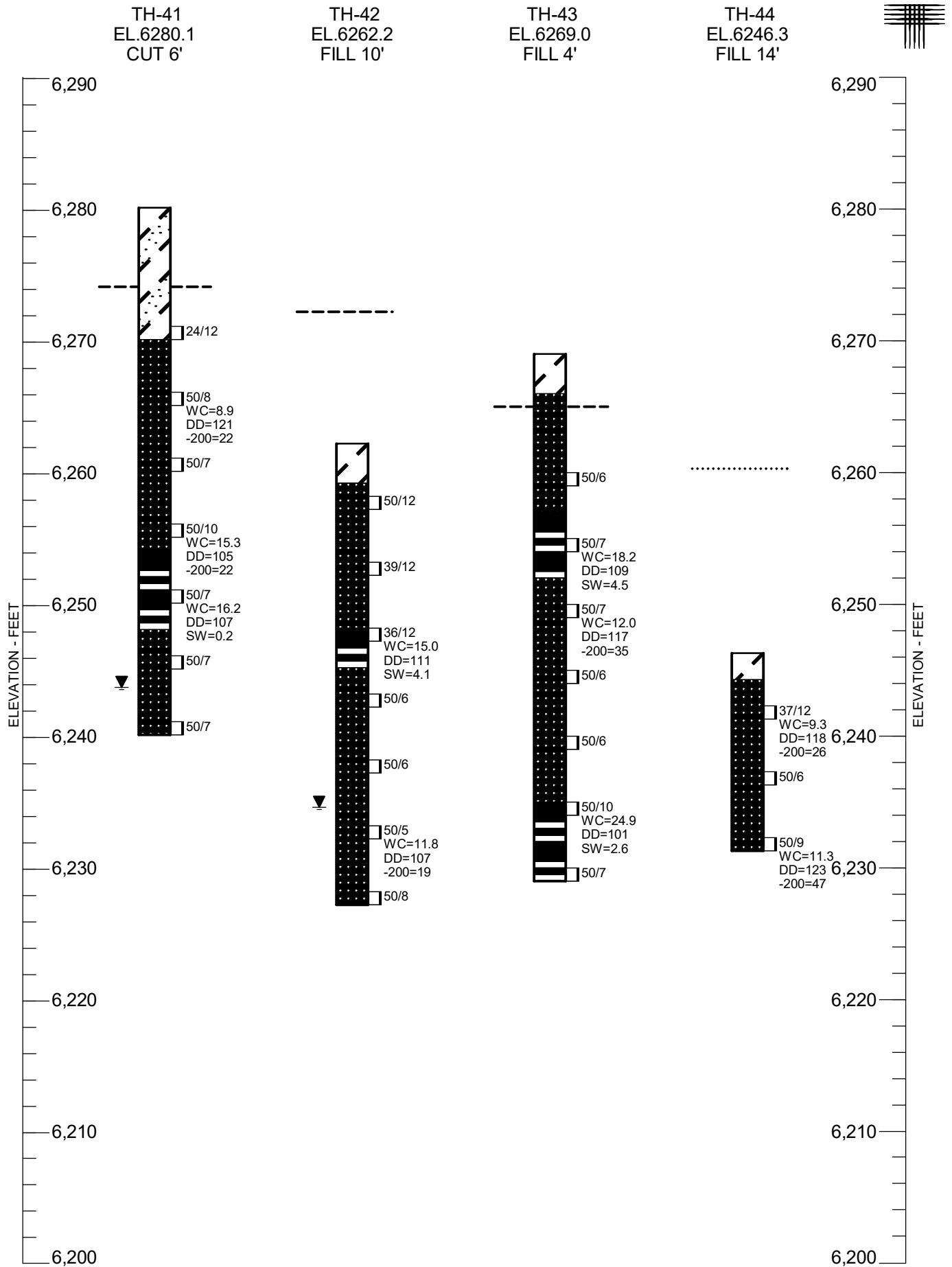




SUMMARY LOGS OF EXPLORATORY BORINGS

BMC INVESTMENTS
CANYONSIDE - SINGLE & MULTI-FAMILY DEVELOPMENT
CTL/T PROJECT NO. DN51,859-125-R1

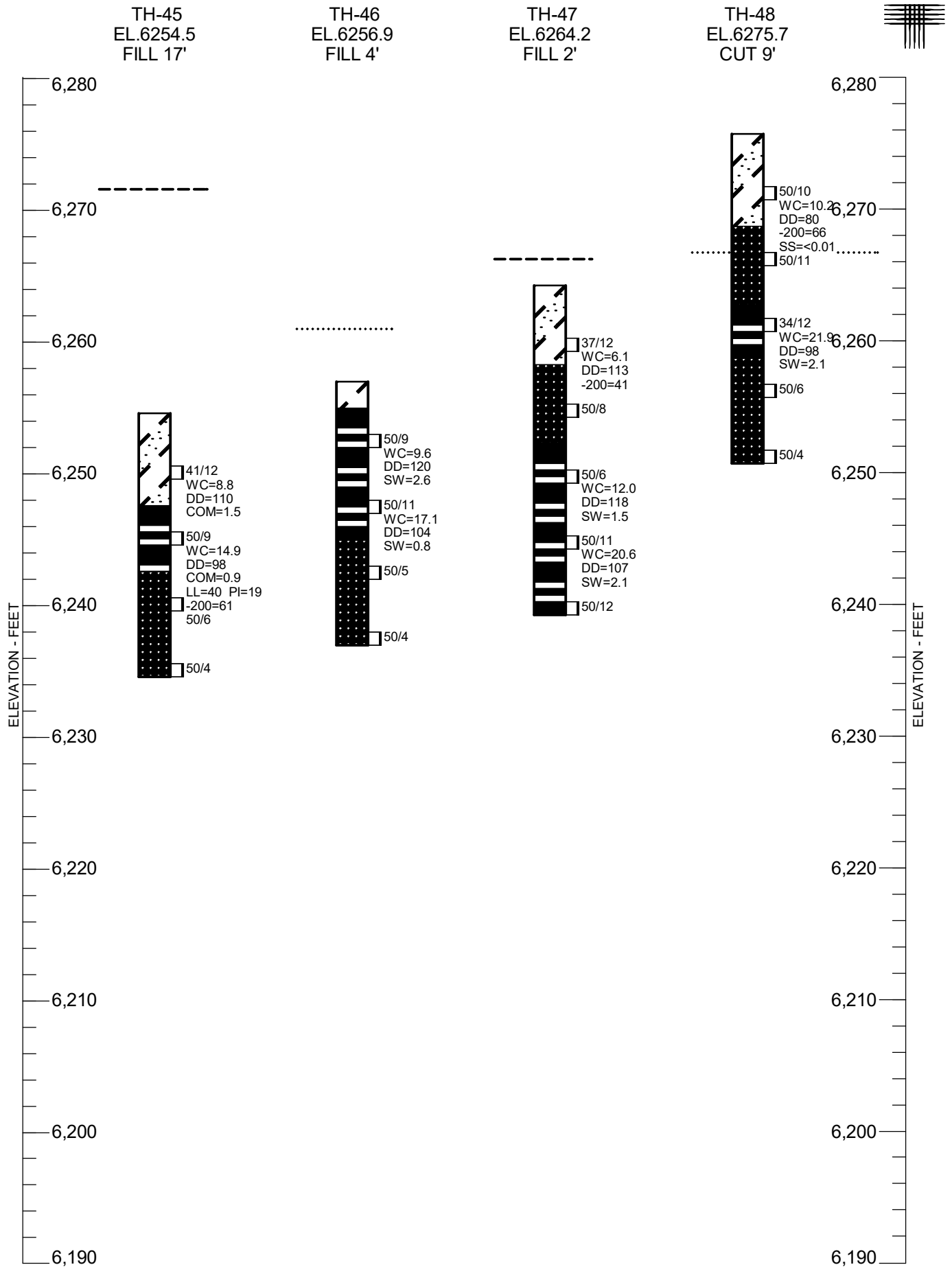
FIG. A-10



SUMMARY LOGS OF EXPLORATORY BORINGS

BMC INVESTMENTS
CANYONSIDE - SINGLE & MULTI-FAMILY DEVELOPMENT
CTL/T PROJECT NO. DN51,859-125-R1

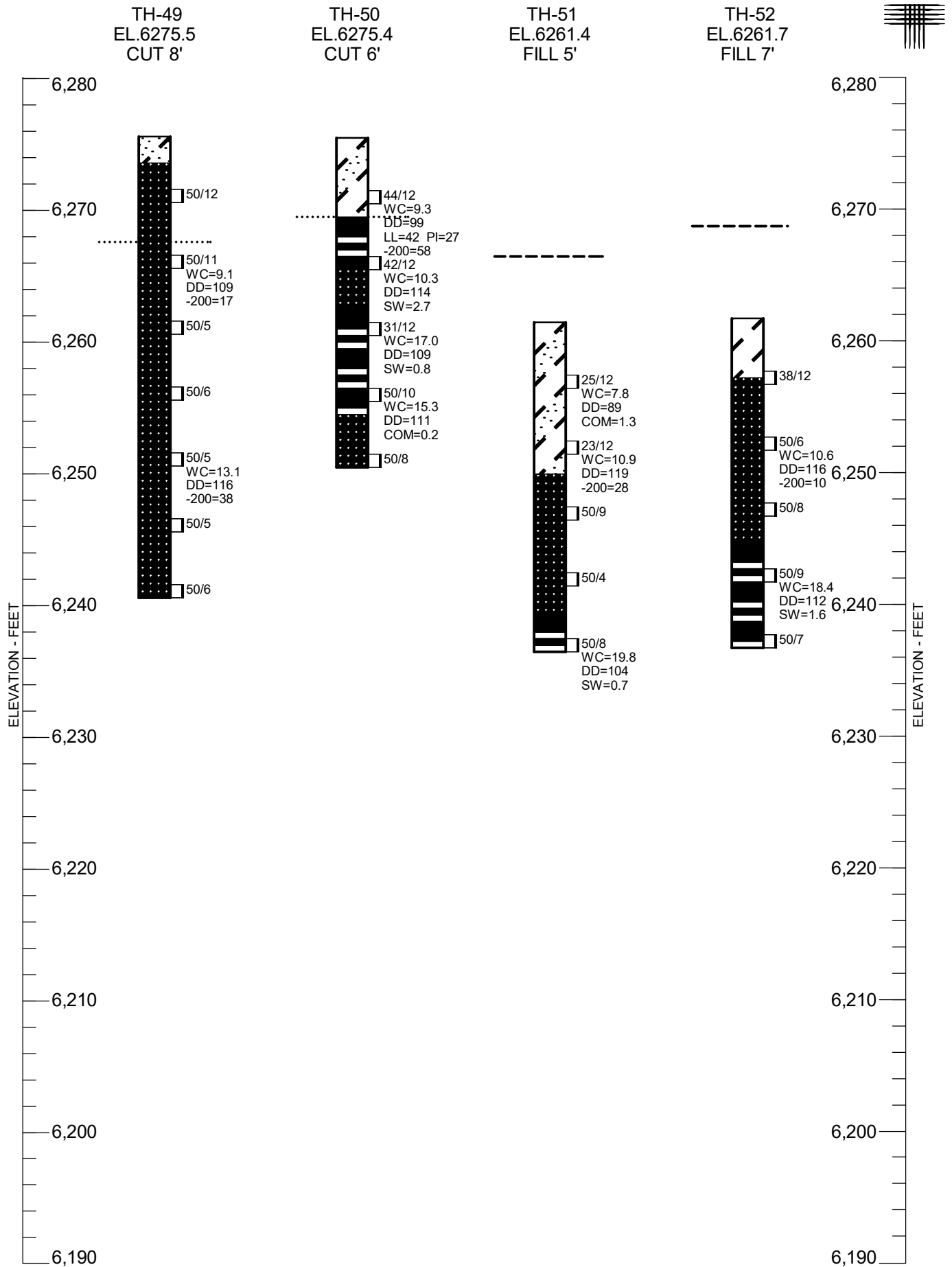
FIG. A-11

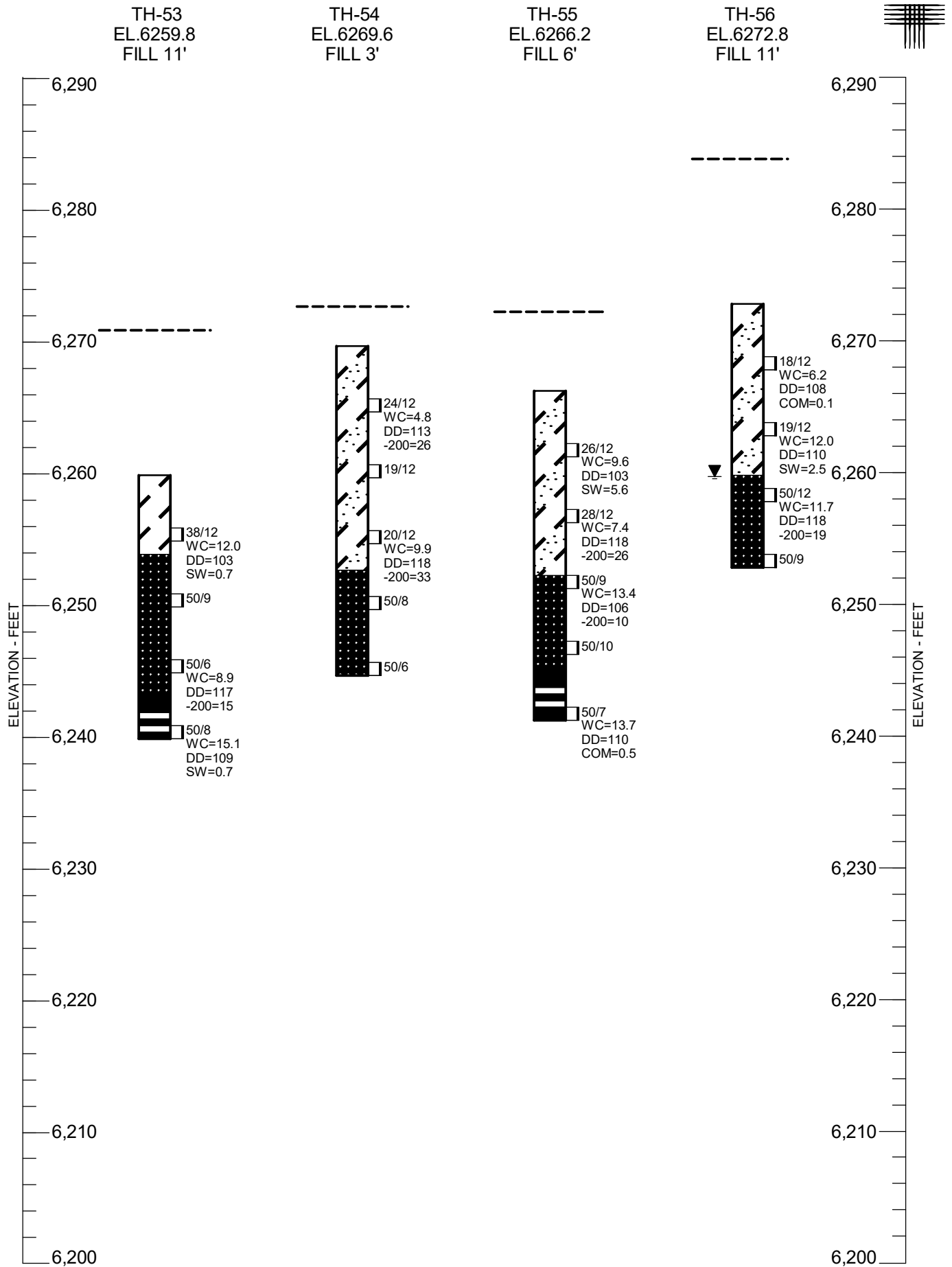


SUMMARY LOGS OF EXPLORATORY BORINGS

BMC INVESTMENTS
CANYONSIDE - SINGLE & MULTI-FAMILY DEVELOPMENT
CTL/T PROJECT NO. DN51,859-125-R1

FIG. A-12

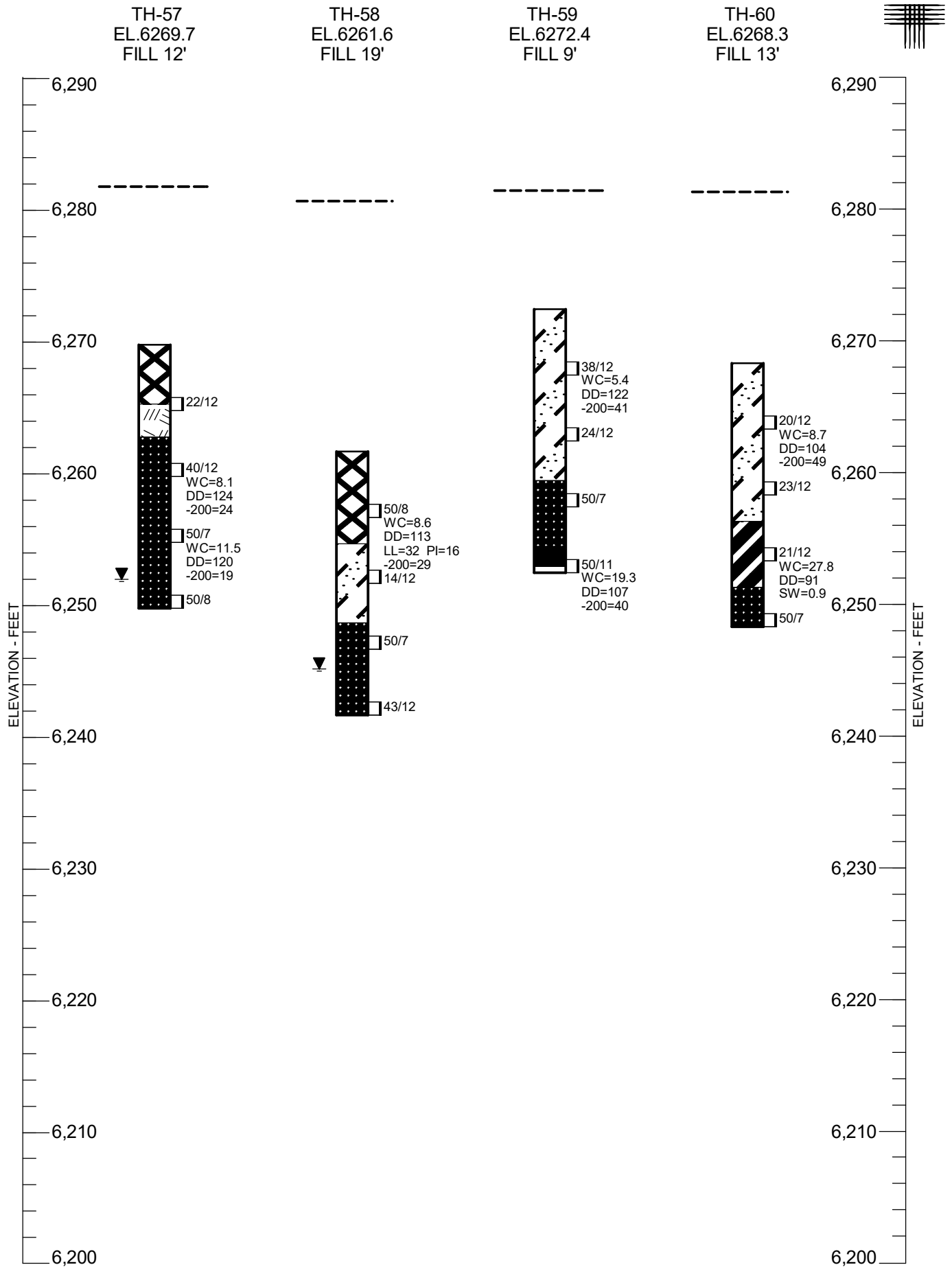


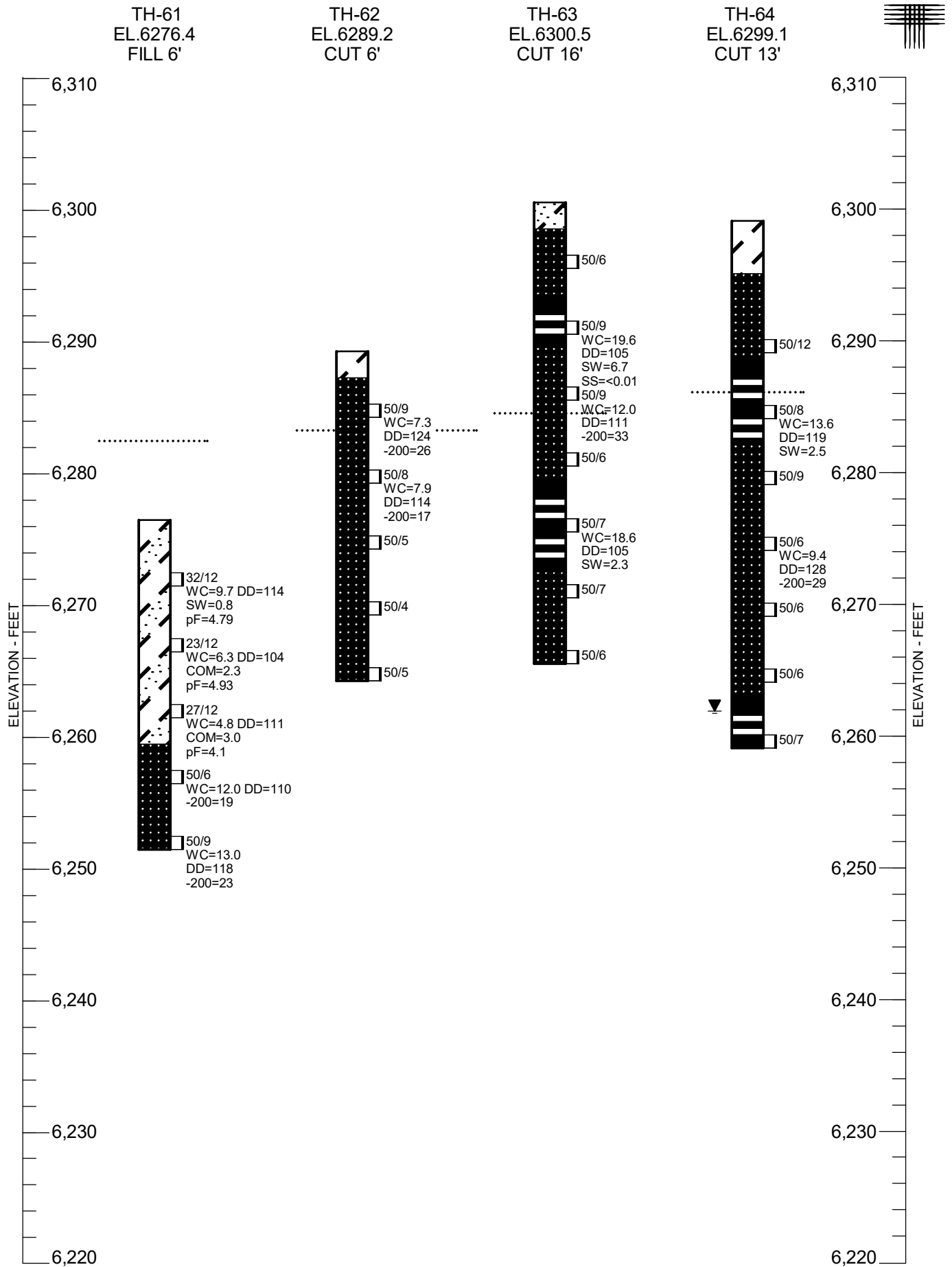


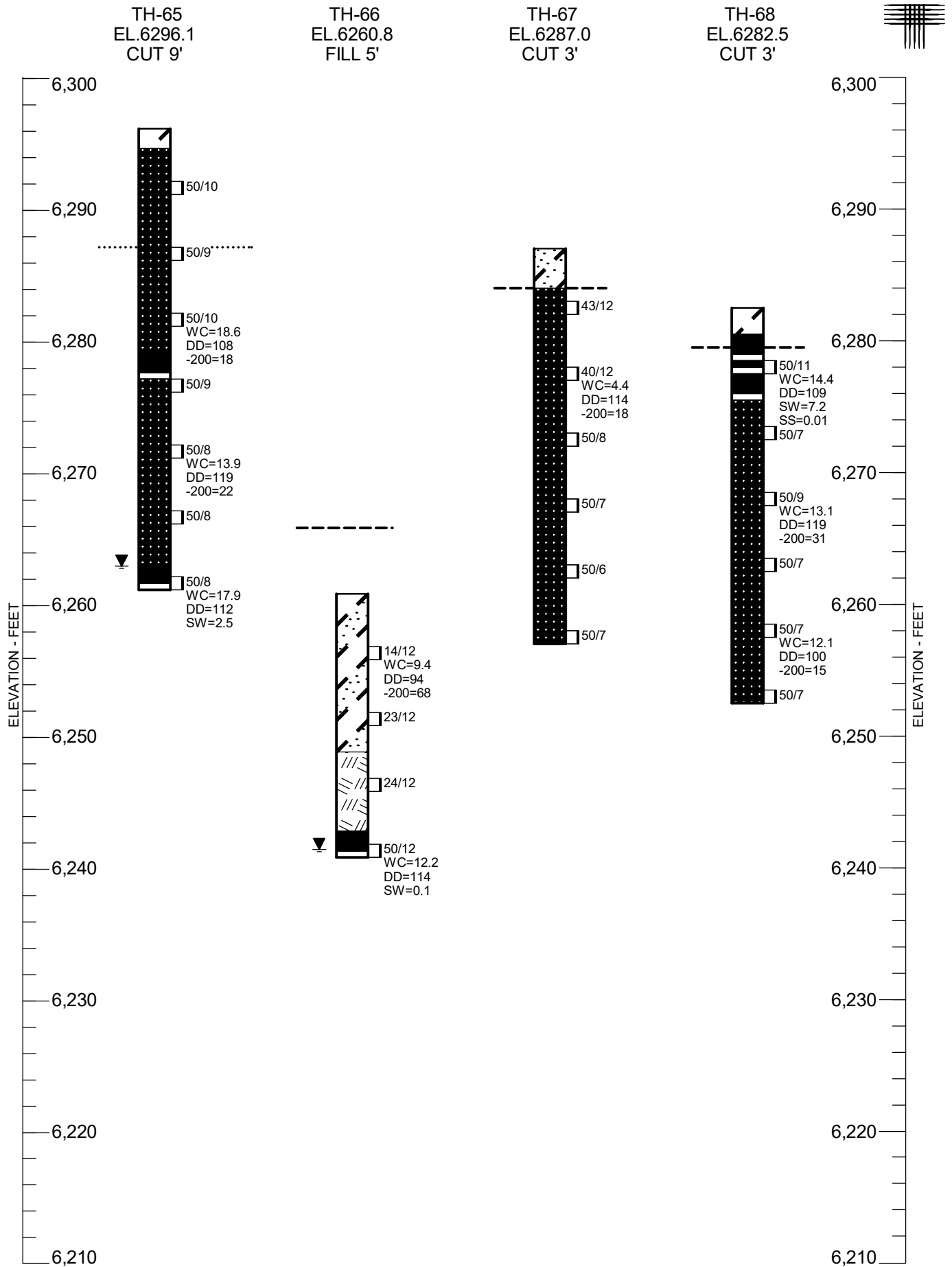
SUMMARY LOGS OF EXPLORATORY BORINGS

BMC INVESTMENTS
CANYONSIDE - SINGLE & MULTI-FAMILY DEVELOPMENT
CTL/T PROJECT NO. DN51,859-125-R1

FIG. A-14



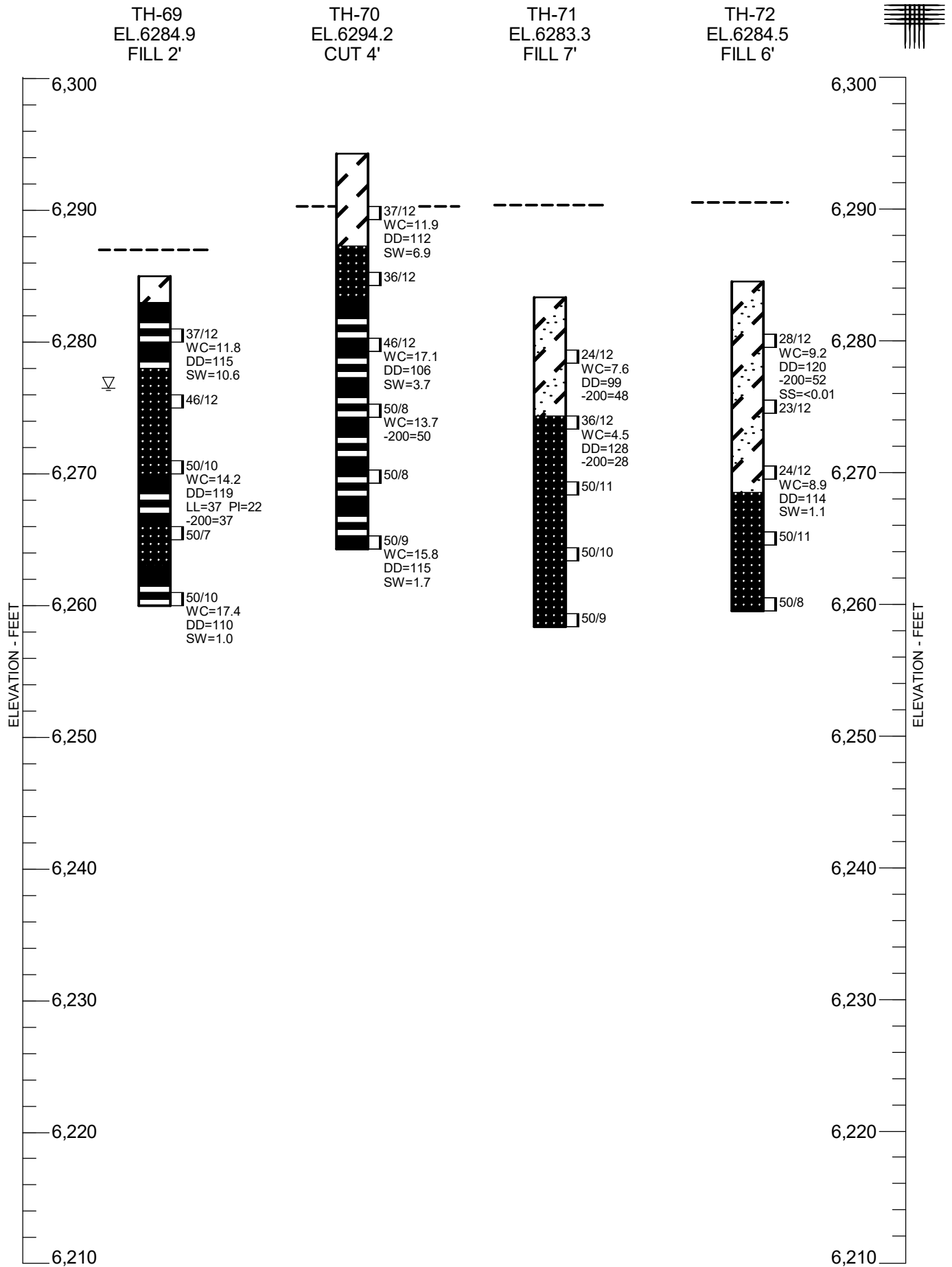


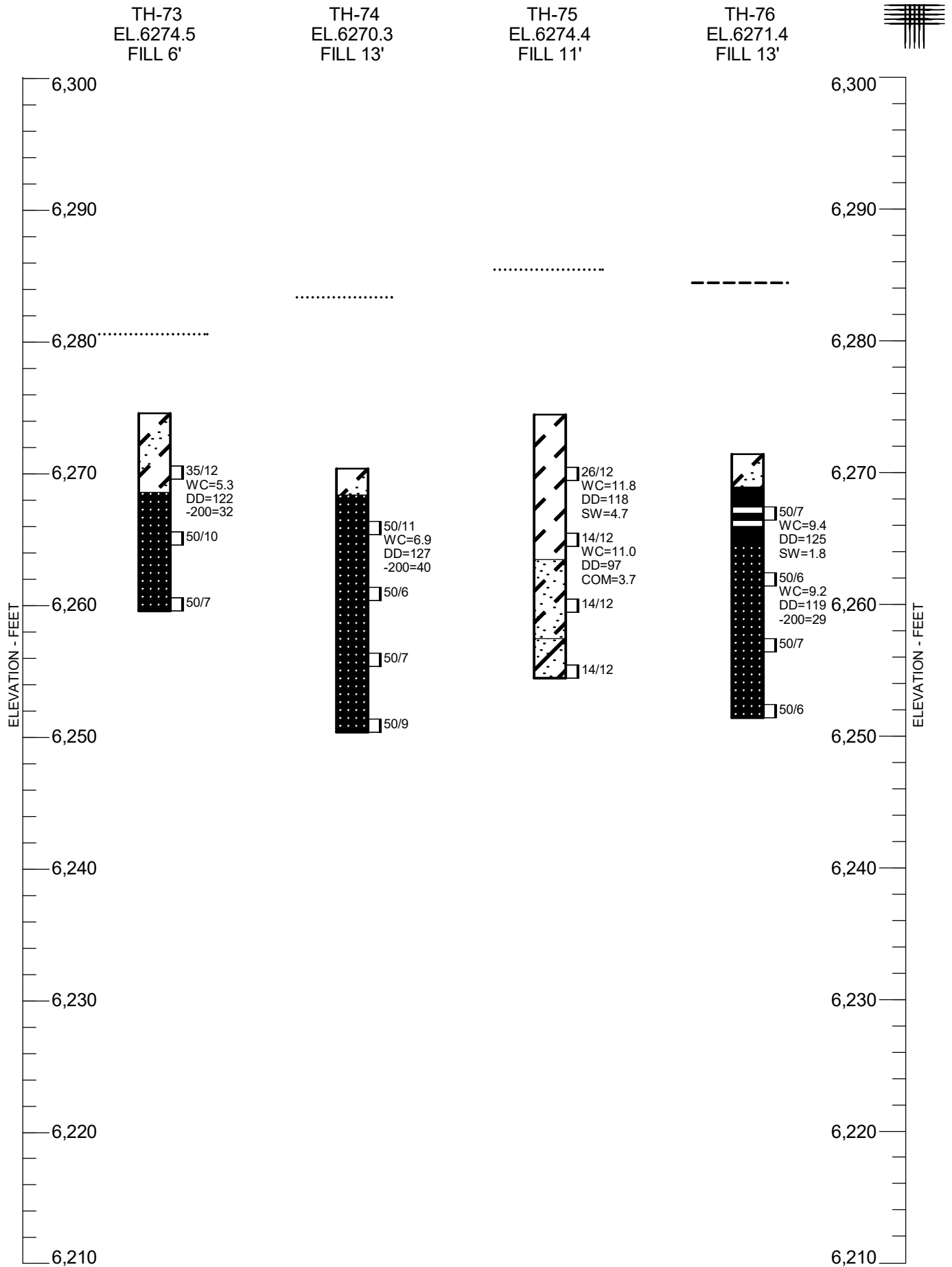


SUMMARY LOGS OF EXPLORATORY BORINGS

BMC INVESTMENTS
CANYONSIDE - SINGLE & MULTI-FAMILY DEVELOPMENT
CTL/T PROJECT NO. DN51,859-125-R1

FIG. A-17

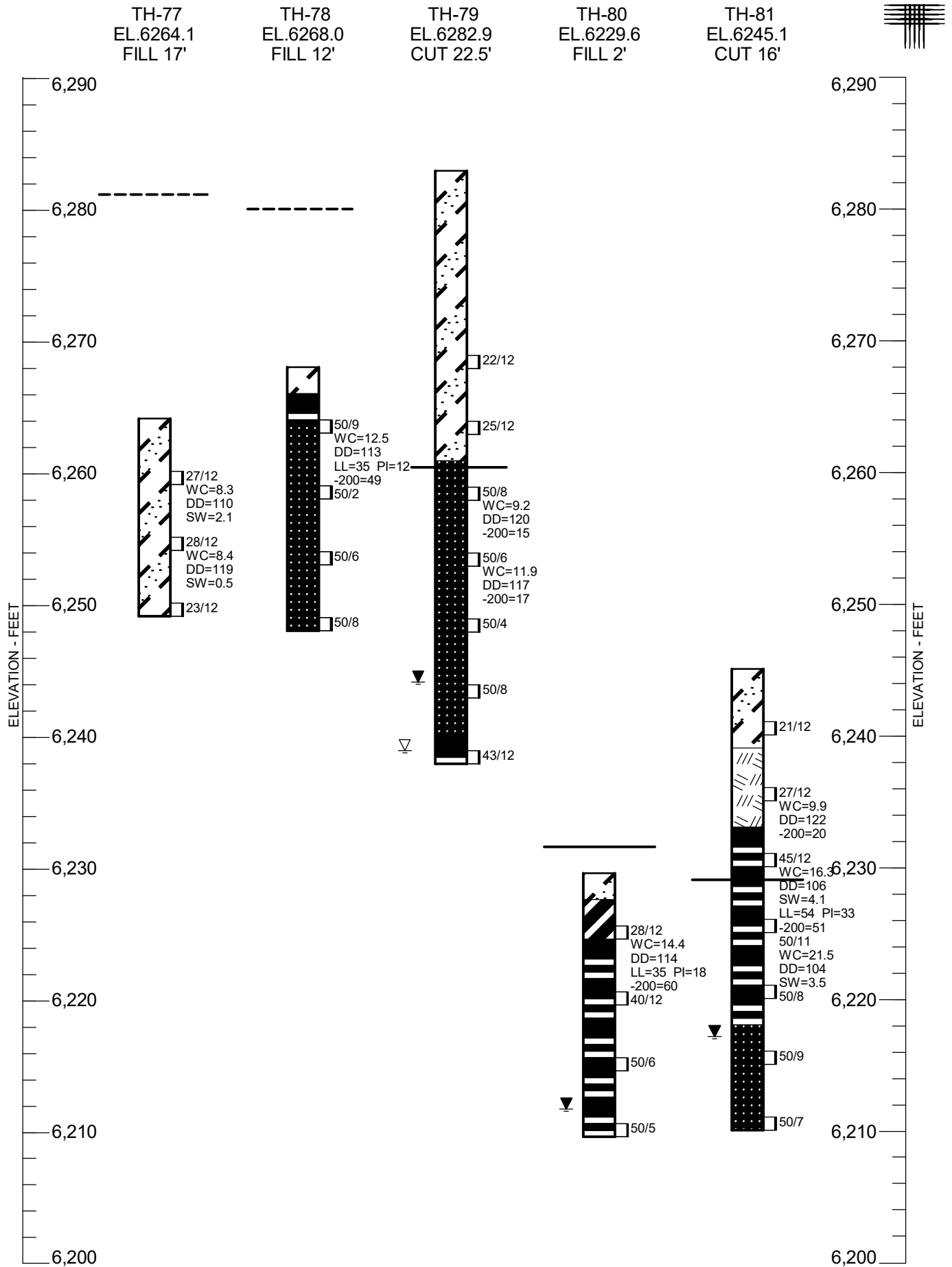


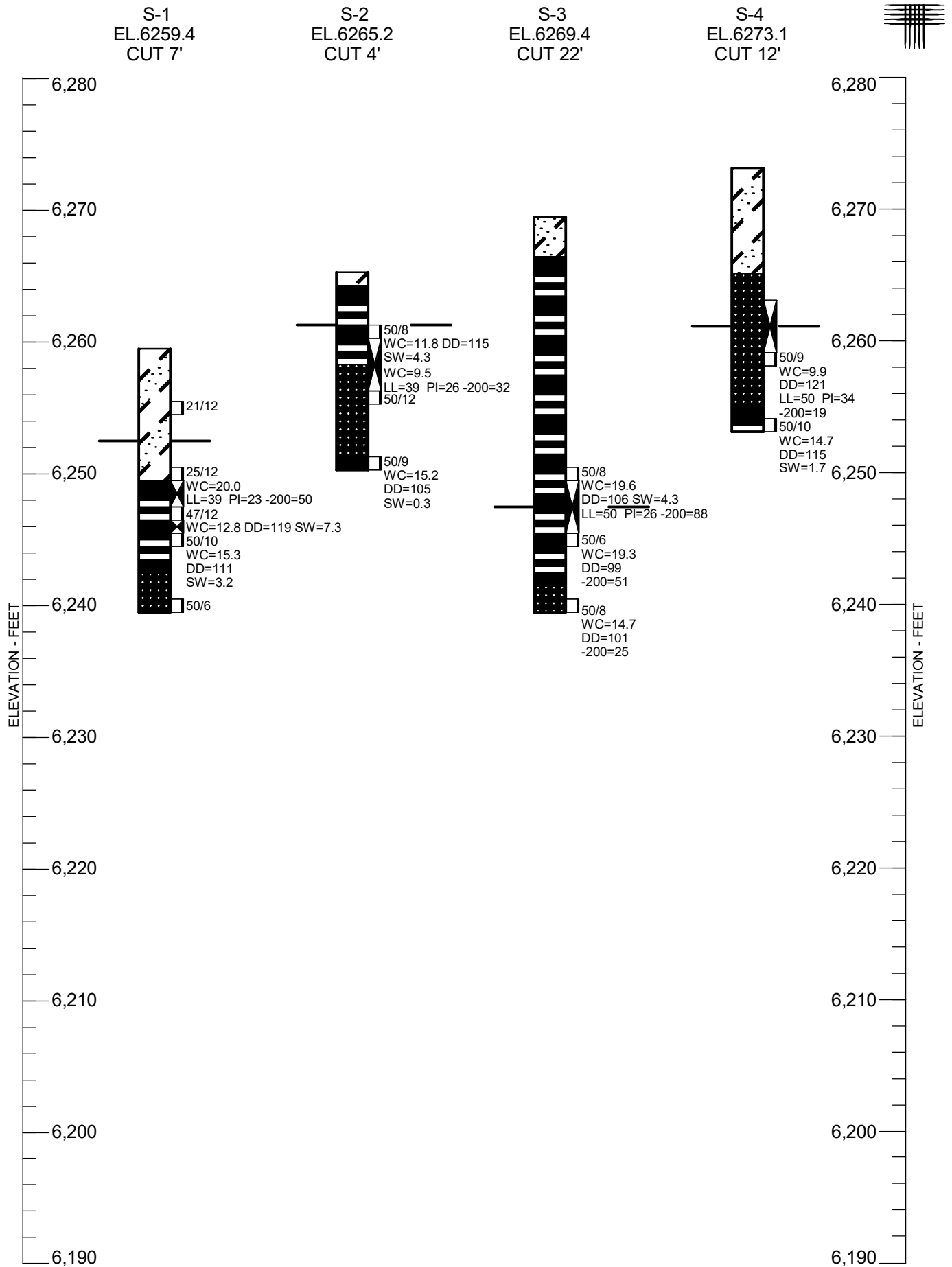


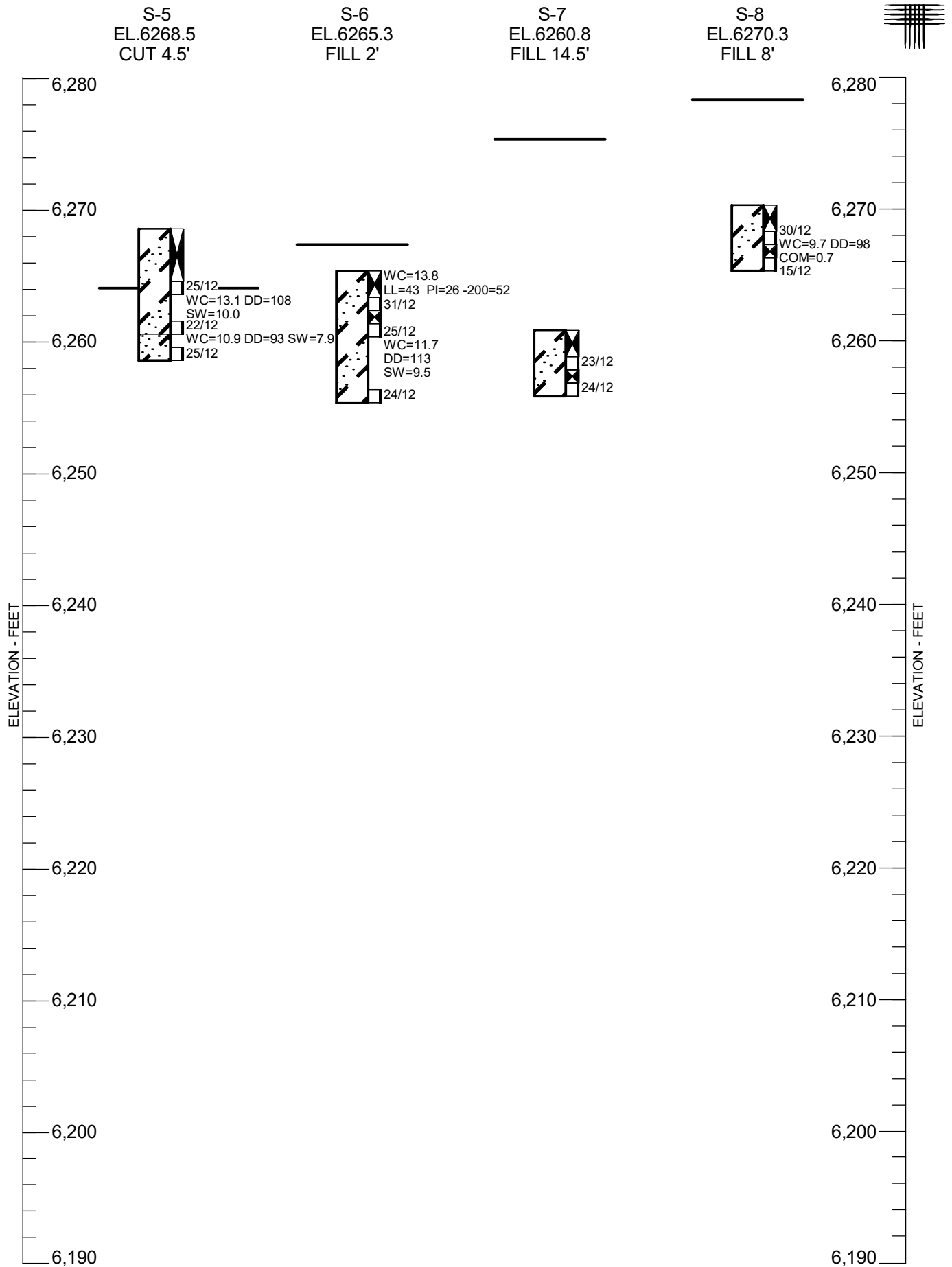
SUMMARY LOGS OF EXPLORATORY BORINGS

BMC INVESTMENTS
CANYONSIDE - SINGLE & MULTI-FAMILY DEVELOPMENT
CTL/T PROJECT NO. DN51,859-125-R1

FIG. A-19



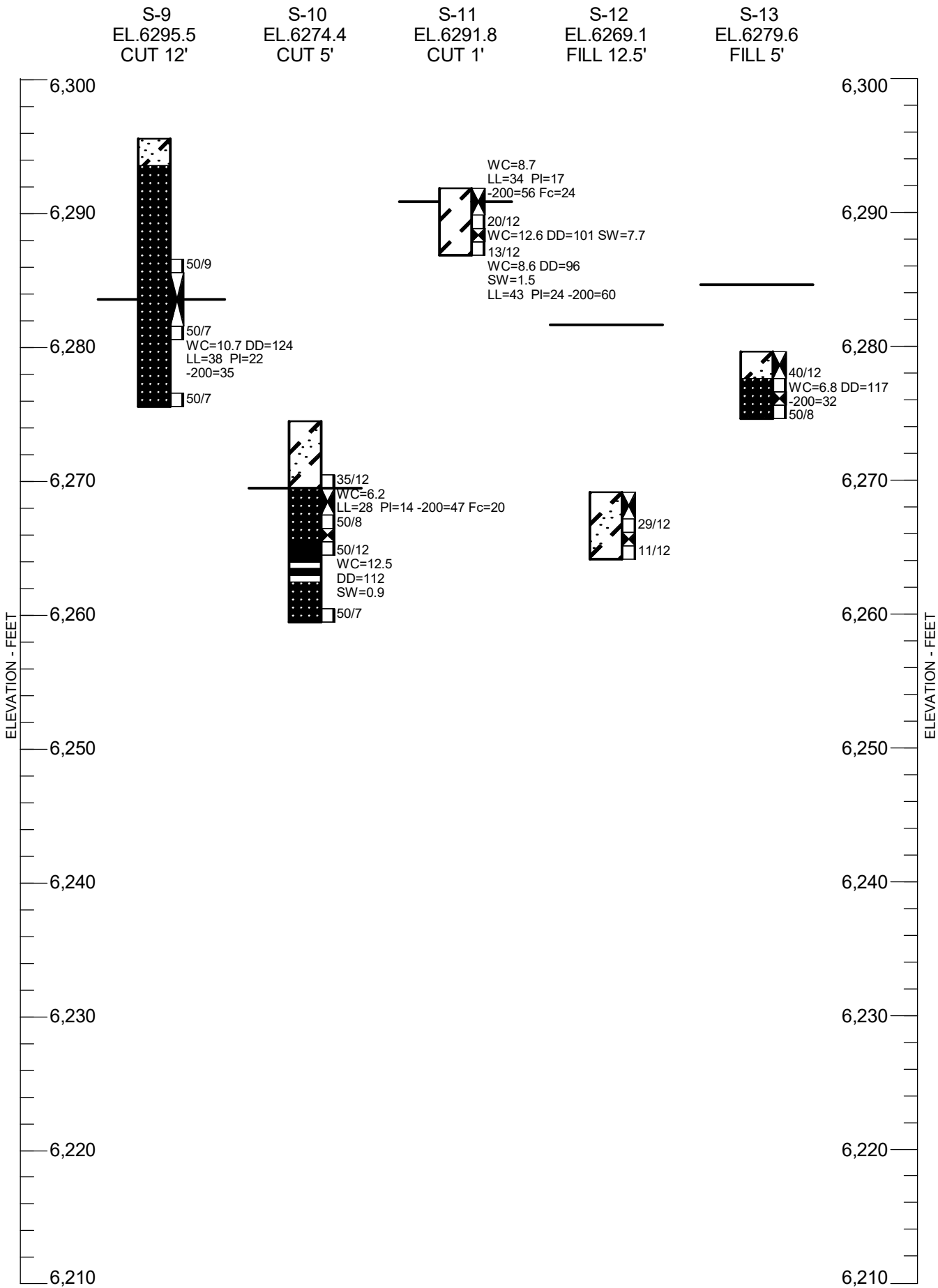




SUMMARY LOGS OF EXPLORATORY BORINGS

BMC INVESTMENTS
CANYONSIDE - SINGLE & MULTI-FAMILY DEVELOPMENT
CTL/T PROJECT NO. DN51,859-125-R1

FIG. A-22



LEGEND:

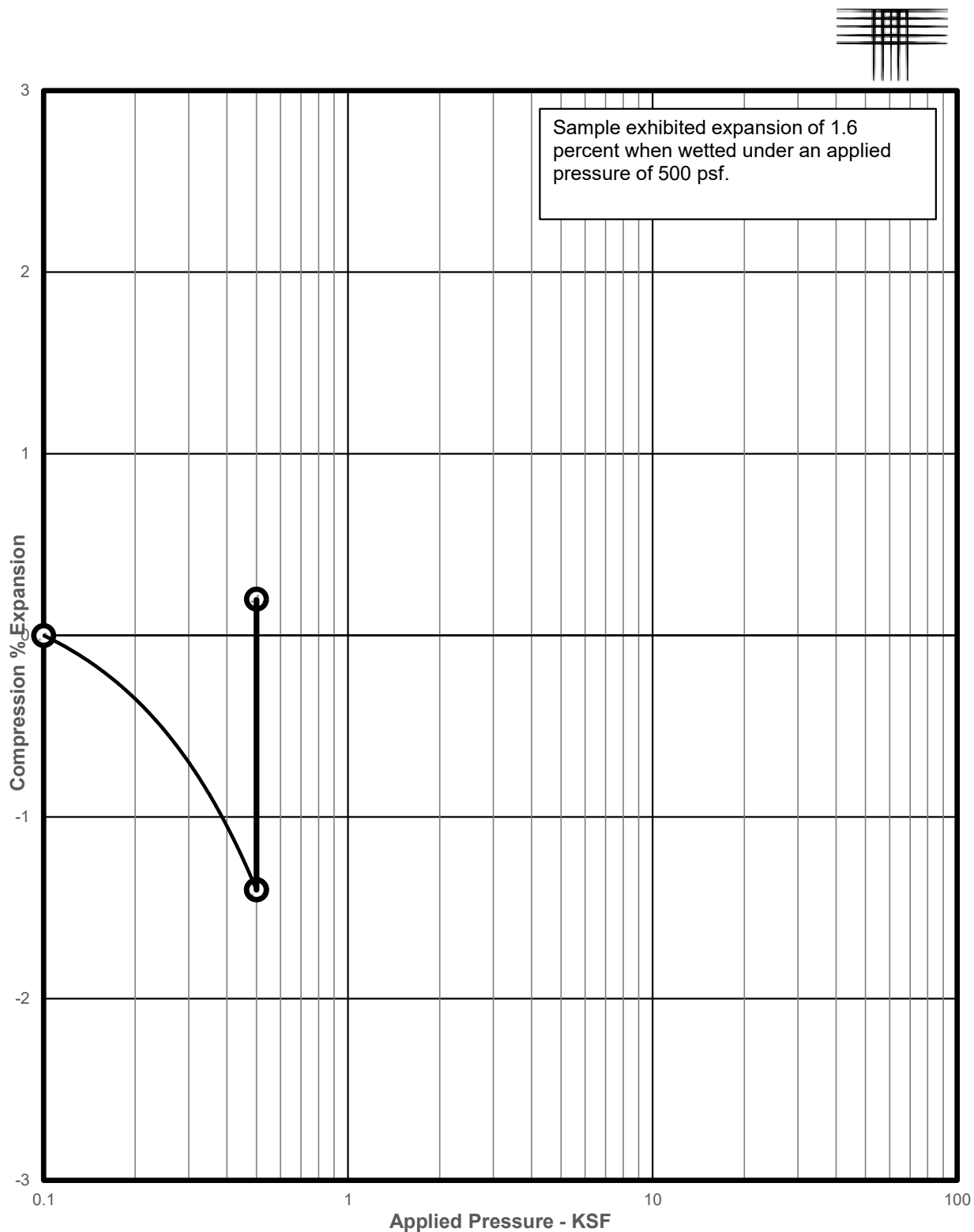
- FILL, SAND, CLAYEY, VERY DENSE, SLIGHTLY MOIST, BLACK, DARK BROWN.
- CLAY, SANDY TO VERY SANDY, OCCASIONAL GRAVEL, STIFF TO VERY STIFF, SLIGHTLY MOIST, BROWN, WHITE, DARK BROWN (CL).
- SAND, CLAYEY, OCCASIONAL GRAVEL, MEDIUM DENSE TO DENSE, SLIGHTLY MOIST, TAN, GRAY (SC).
- SAND, SILTY, OCCASIONAL GRAVEL, MEDIUM DENSE TO VERY DENSE, SLIGHTLY MOIST, WHITE, RUST, TAN (SM).
- INTERLAYERED CLAY/SAND, CLAY DOMINATE, OCCASIONAL GRAVEL, STIFF TO VERY STIFF OR MEDIUM DENSE TO VERY DENSE, SLIGHTLY MOIST, BROWN, TAN, WHITE (CL, SC).
- WEATHERED CLAYSTONE, SLIGHTLY MOIST TO MOIST, BROWN, RUST, GRAY, WHITE (CL).
- WEATHERED SANDSTONE, SLIGHTLY MOIST TO MOIST, TAN, WHITE, RUST.
- BEDROCK, SANDSTONE, CLAYEY, OCCASIONAL TO FREQUENT CLAYSTONE LAYERS, MEDIUM HARD TO VERY HARD, SLIGHTLY MOIST TO MOIST, BROWN, GRAY, WHITE, TAN, RUST (SC).
- BEDROCK, CLAYSTONE, SANDY, OCCASIONAL SANDSTONE LAYERS, MEDIUM HARD TO VERY HARD, SLIGHTLY MOIST TO MOIST, BROWN, GRAY (CL, CH).
- DRIVE SAMPLE. THE SYMBOL 29/12 INDICATES 29 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2.5-INCH O.D. SAMPLER 12 INCHES.
- BULK SAMPLE COLLECTED FROM AUGER CUTTINGS.
- GROUNDWATER LEVEL MEASURED AT TIME OF DRILLING.
- GROUNDWATER LEVEL MEASURED AFTER DRILLING ON APRIL 6, 2023.
- INDICATES PRACTICAL DRILL RIG REFUSAL.
- INDICATES PROPOSED GRADE.
- INDICATES 1ST FLOOR ELEVATION.
- INDICATES BASEMENT FLOOR ELEVATION.

- NOTES:**
- THE BORINGS WERE DRILLED BETWEEN FEBRUARY 18 AND MARCH 8, 2023 USING A 4-INCH DIAMETER, CONTINUOUS-FLIGHT AUGER AND TRACK, BUGGY, OR TRUCK-MOUNTED DRILL RIGS.
 - BORING LOCATIONS AND ELEVATIONS WERE STAKED AND SURVEYED BY R&R ENGINEERS-SURVEYORS, INC.
 - WC - INDICATES MOISTURE CONTENT (%).
DD - INDICATES DRY DENSITY (PCF).
SW - INDICATES SWELL WHEN WETTED UNDER APPROXIMATE OVERBURDEN PRESSURE (%).
COM - INDICATES COMPRESSION 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).
Fc - INDICATES CLAY FRACTION (%).
 - THESE LOGS ARE SUBJECT TO THE EXPLANATIONS, LIMITATIONS, AND CONCLUSIONS AS CONTAINED IN THIS REPORT.

SUMMARY LOGS OF EXPLORATORY BORINGS



APPENDIX B
LABORATORY TEST RESULTS
TABLE B-I – SUMMARY OF LABORATORY TESTING
TABLE B-II – ESTIMATED POTENTIAL GROUND SURFACE HEAVE

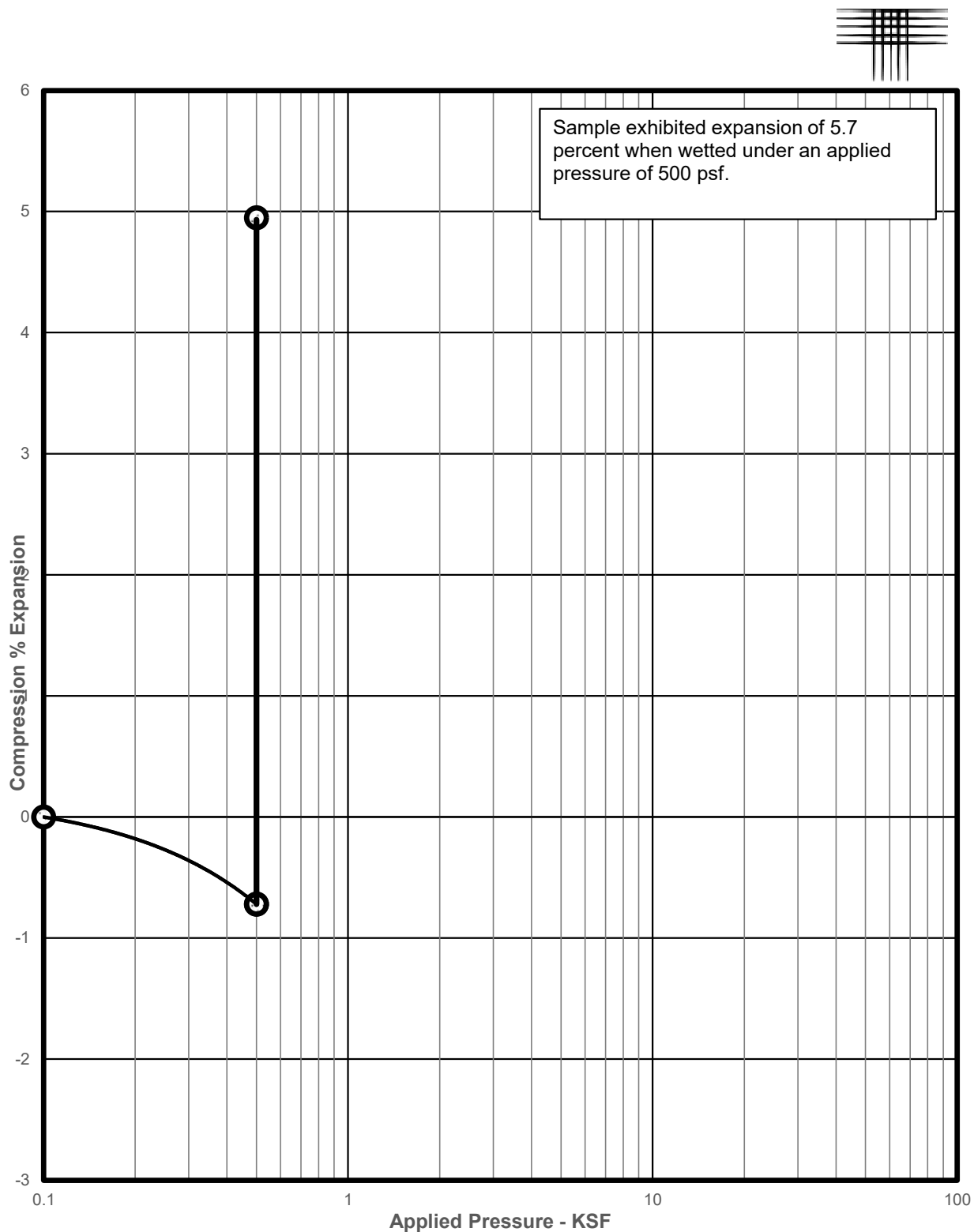


SAMPLE OF: CLAYSTONE
 FROM: TH-1 AT 9 FEET

DRY UNIT WEIGHT: 105 pcf
 MOISTURE CONTENT: 13.3 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 1

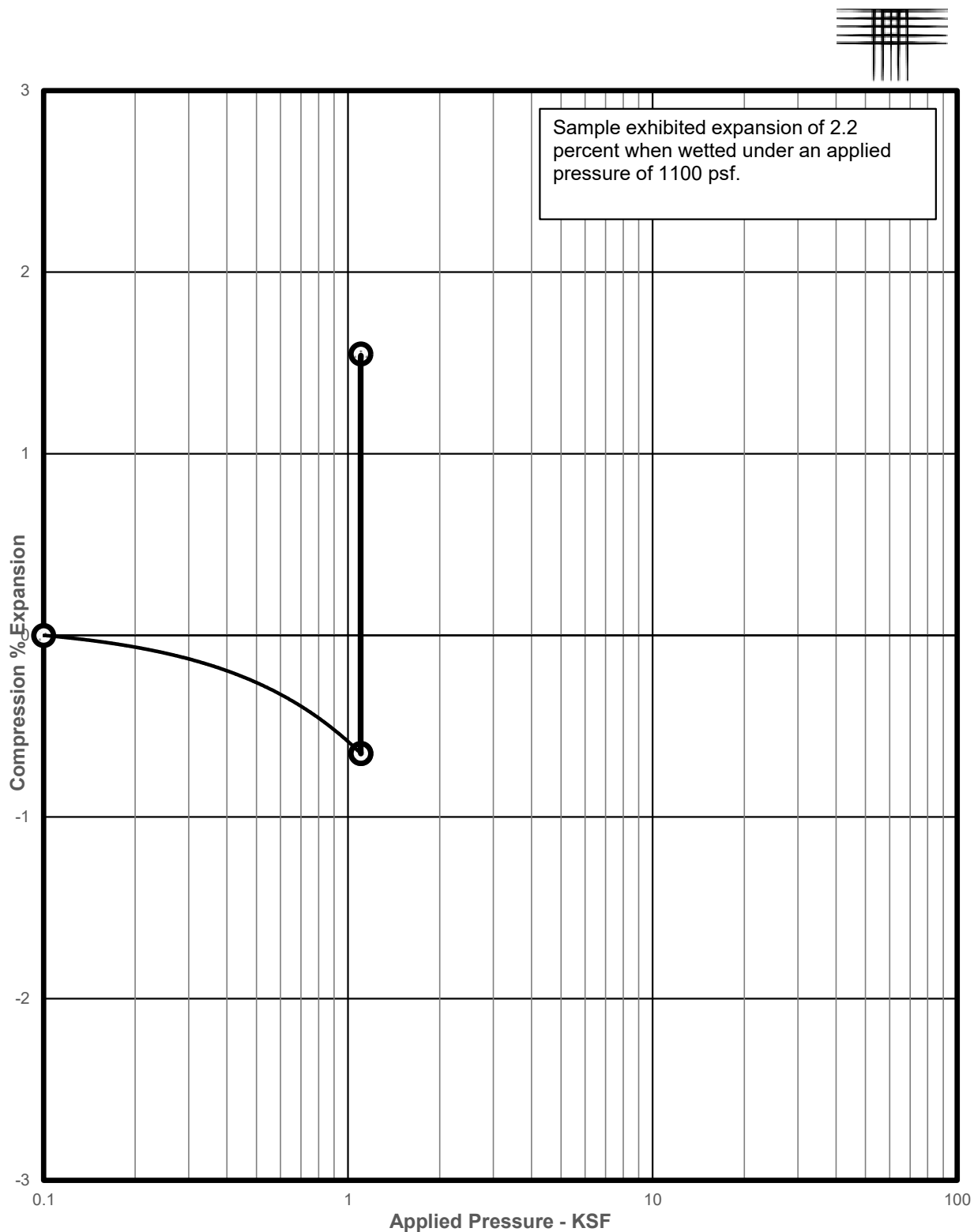


SAMPLE OF: CLAYSTONE
 FROM: TH-3 AT 9 FEET

DRY UNIT WEIGHT: 106 pcf
 MOISTURE CONTENT: 17.1 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 2

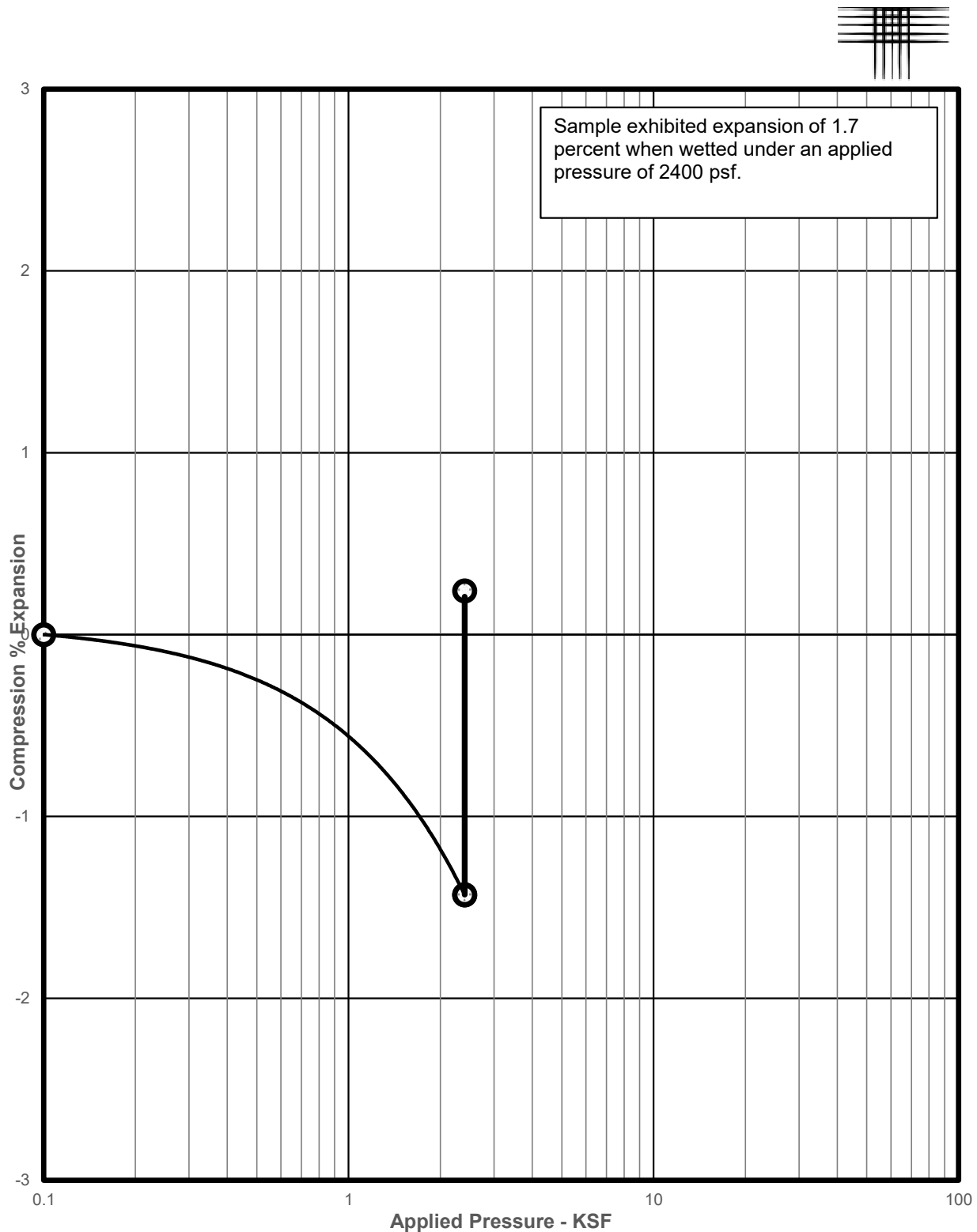


SAMPLE OF: CLAYSTONE
 FROM: TH-3 AT 14 FEET

DRY UNIT WEIGHT: 116 pcf
 MOISTURE CONTENT: 14.0 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 3

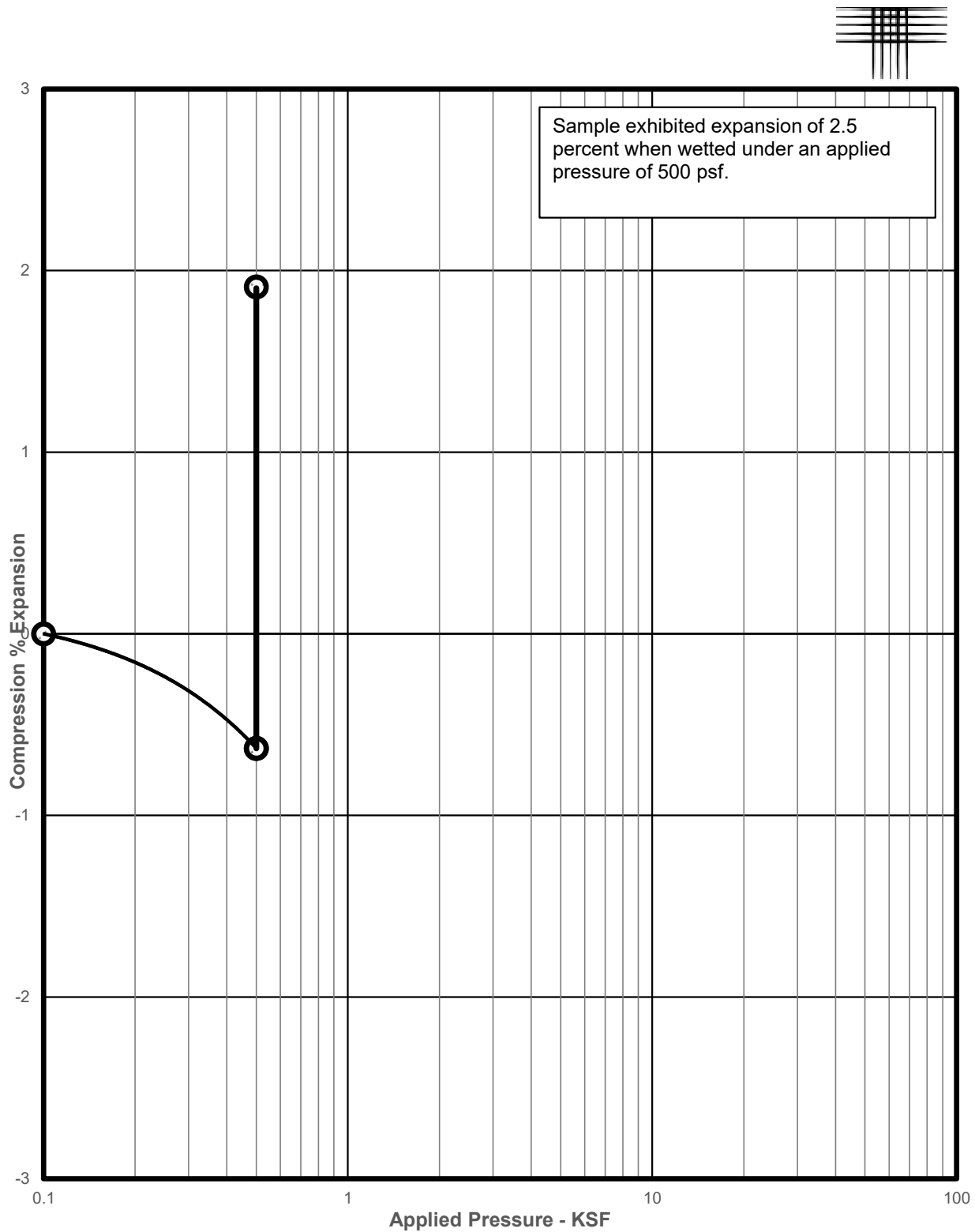


SAMPLE OF: CLAYSTONE
 FROM: TH-4 AT 24 FEET

DRY UNIT WEIGHT: 111 pcf
 MOISTURE CONTENT: 17.3 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 4

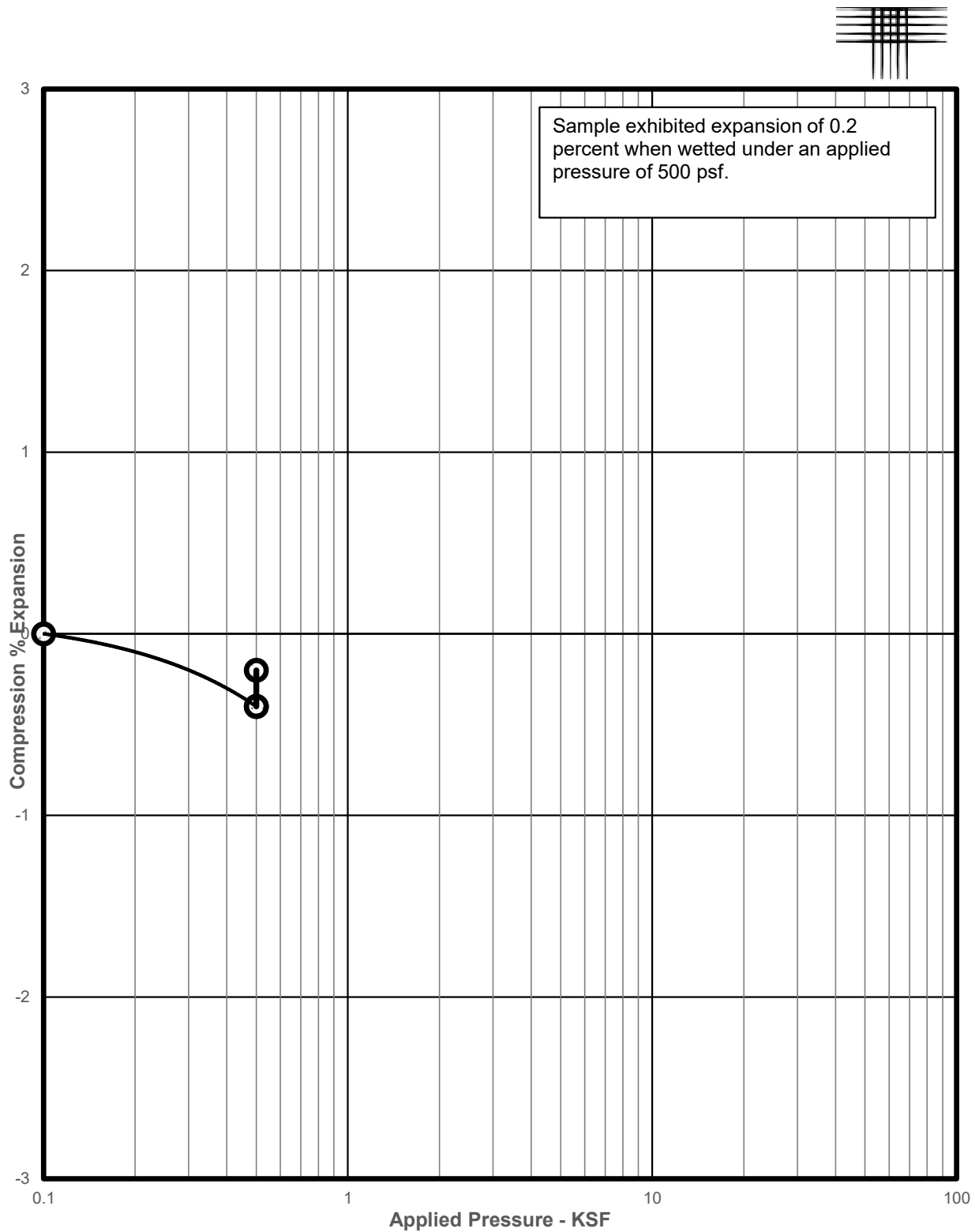


SAMPLE OF: CLAYSTONE
 FROM: TH-5 AT 9 FEET

DRY UNIT WEIGHT: 94 pcf
 MOISTURE CONTENT: 12.8 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 5

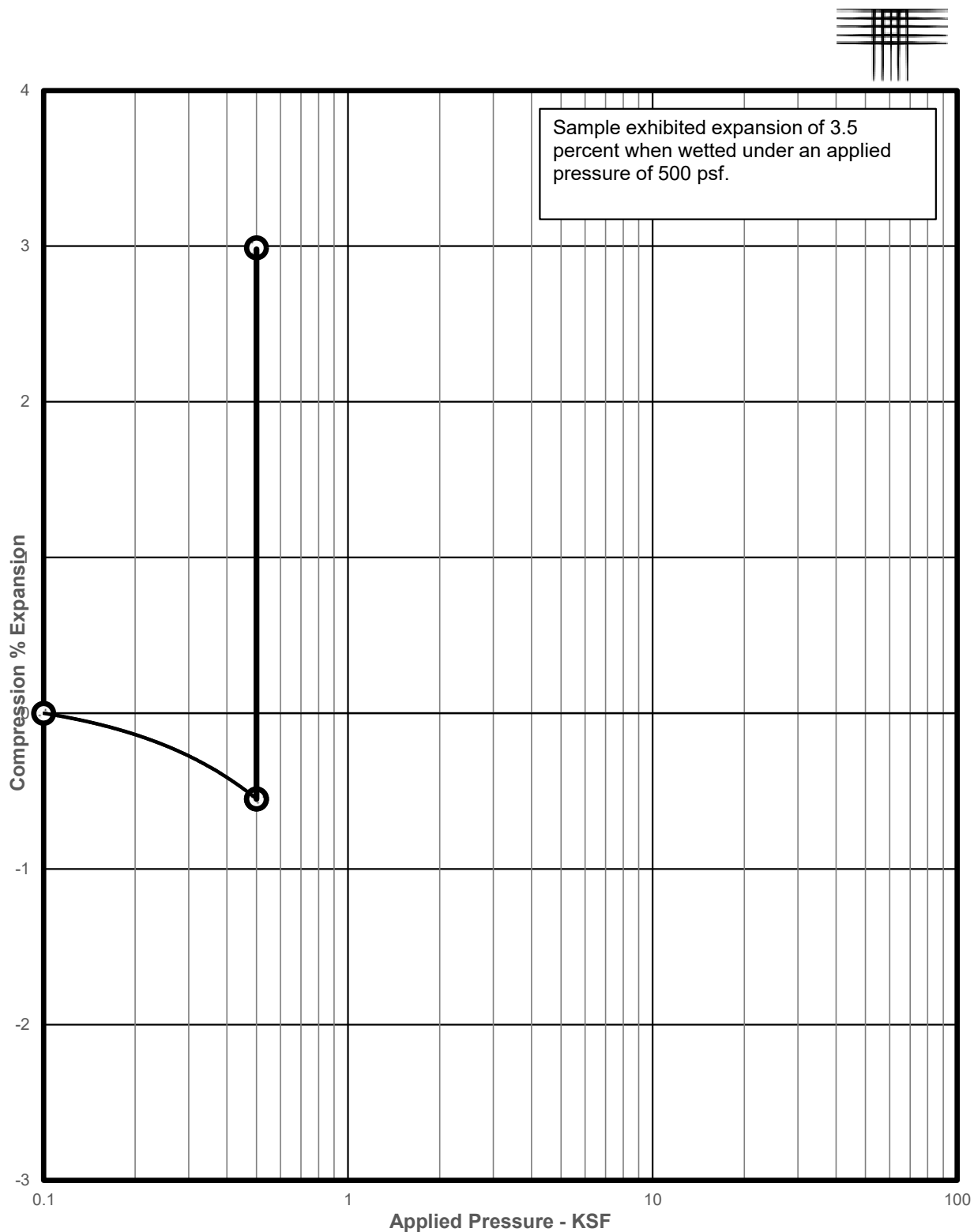


SAMPLE OF: CLAYSTONE
 FROM: TH-6 AT 14 FEET

DRY UNIT WEIGHT: 115 pcf
 MOISTURE CONTENT: 10.6 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 6

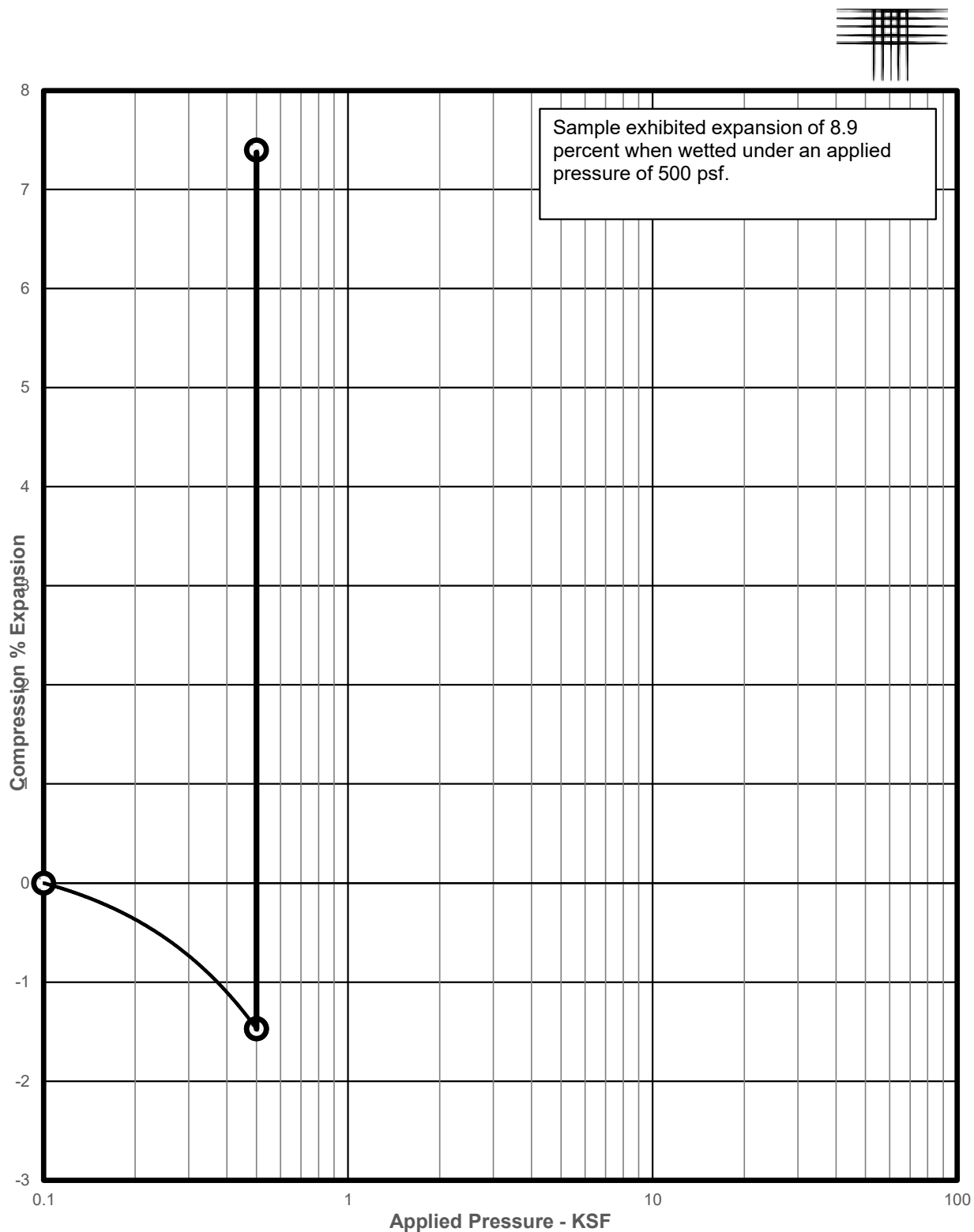


SAMPLE OF: CLAYSTONE
 FROM: TH-7 AT 9 FEET

DRY UNIT WEIGHT: 106 pcf
 MOISTURE CONTENT: 15.5 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 7

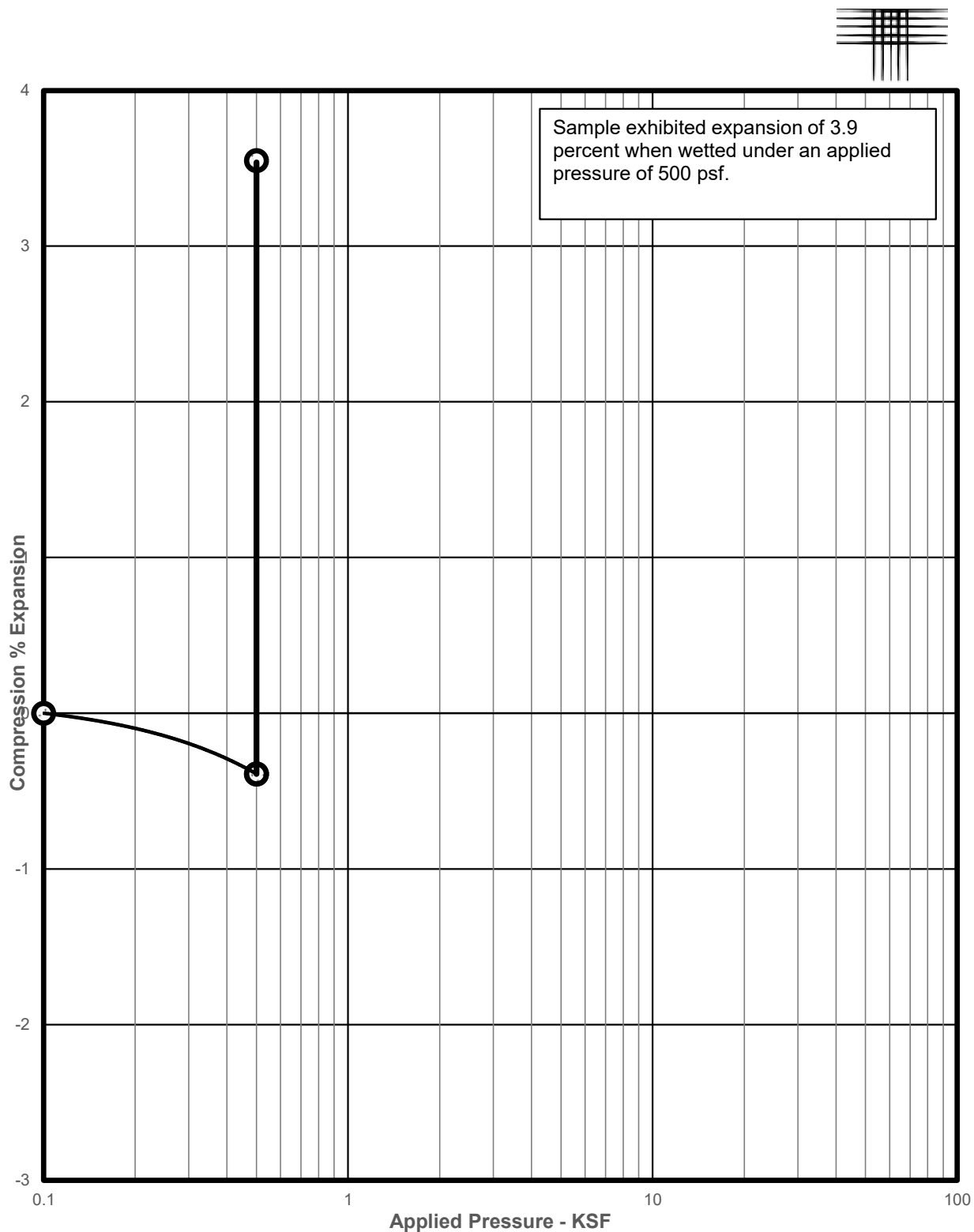


SAMPLE OF: CLAYSTONE
 FROM: TH-8 AT 14 FEET

DRY UNIT WEIGHT: 101 pcf
 MOISTURE CONTENT: 14.0 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 8

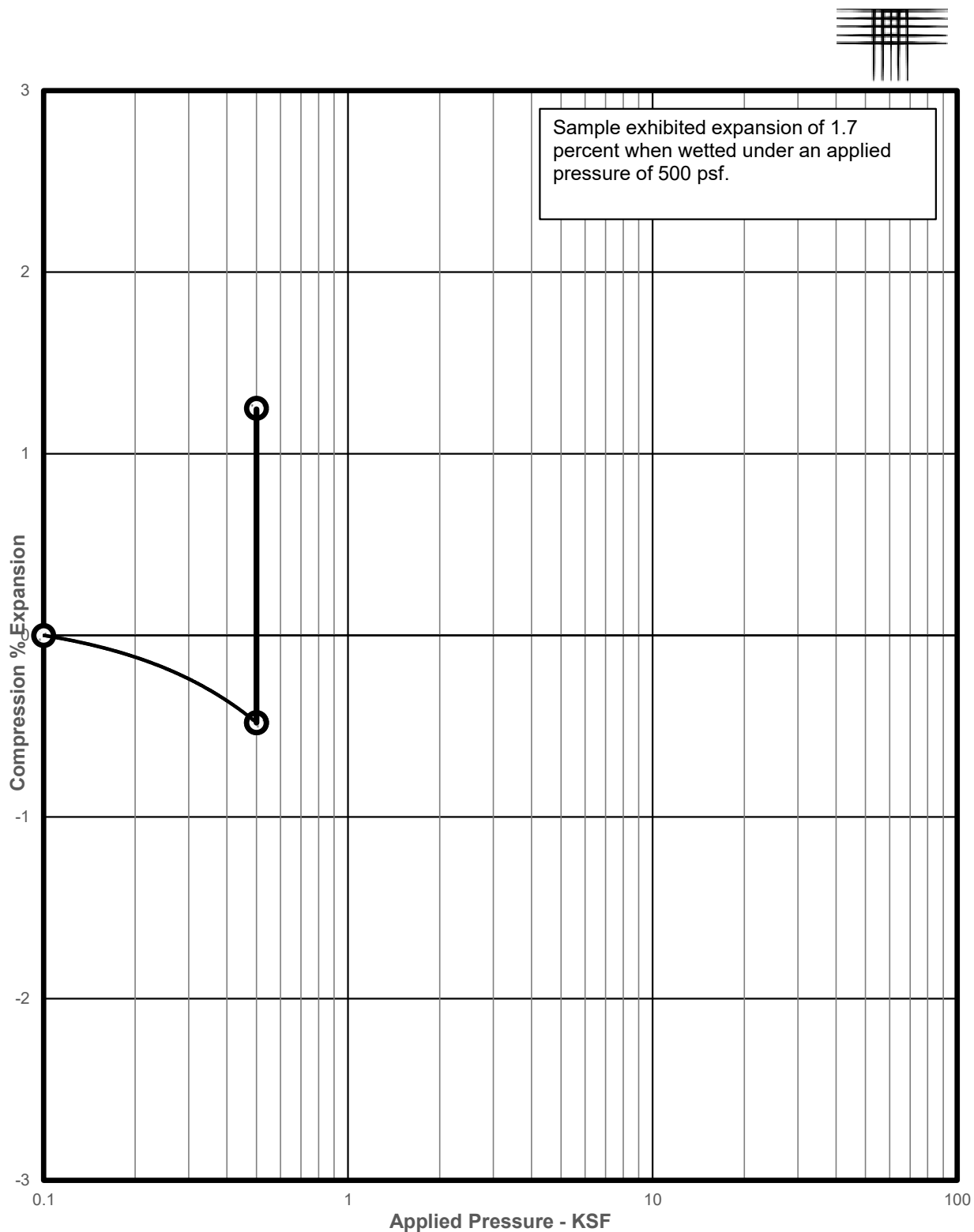


SAMPLE OF: CLAYSTONE
 FROM: TH-9 AT 9 FEET

DRY UNIT WEIGHT: 117 pcf
 MOISTURE CONTENT: 11.3 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 9

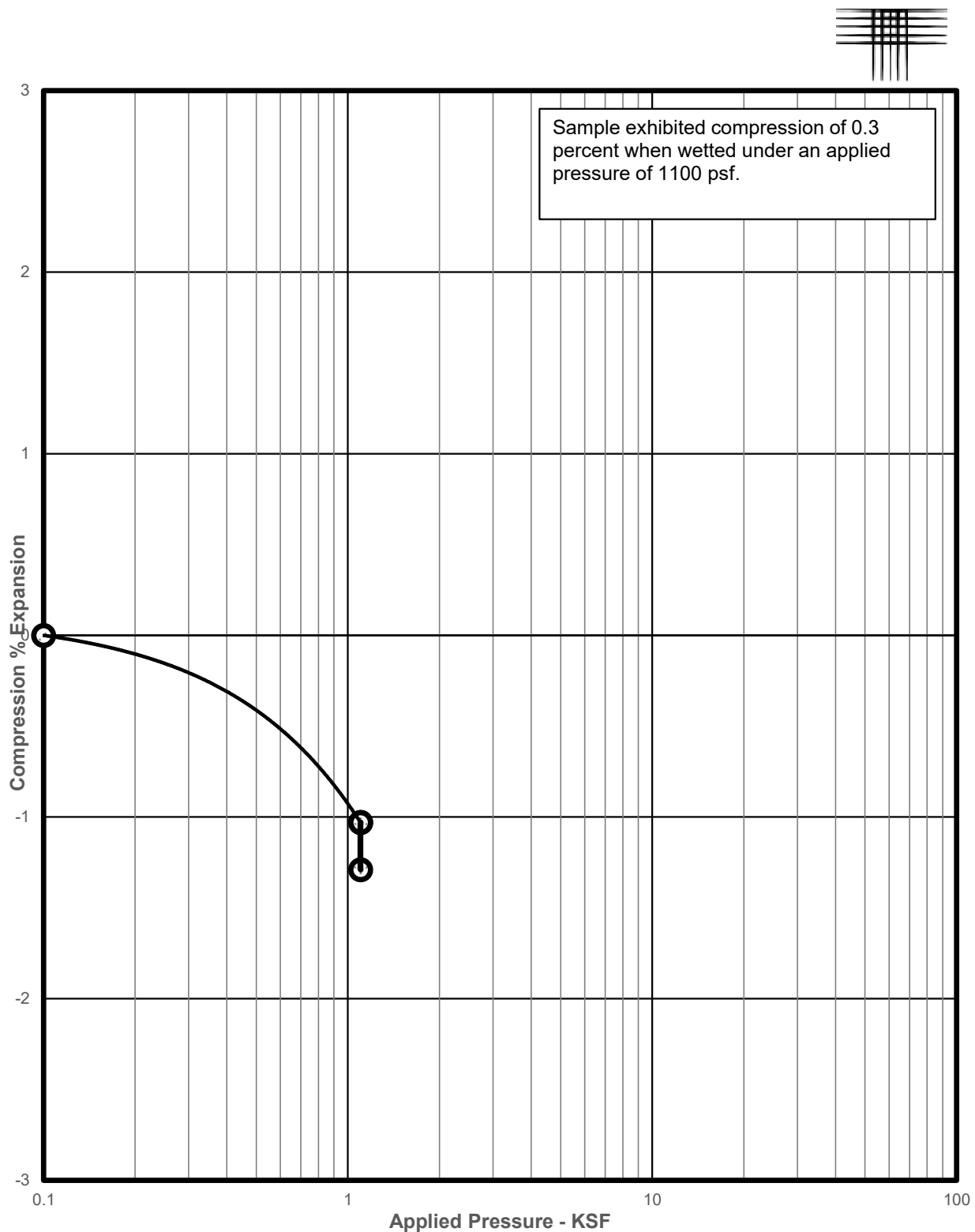


SAMPLE OF: CLAYSTONE
 FROM: TH-9 AT 14 FEET

DRY UNIT WEIGHT: 104 pcf
 MOISTURE CONTENT: 16.8 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 10

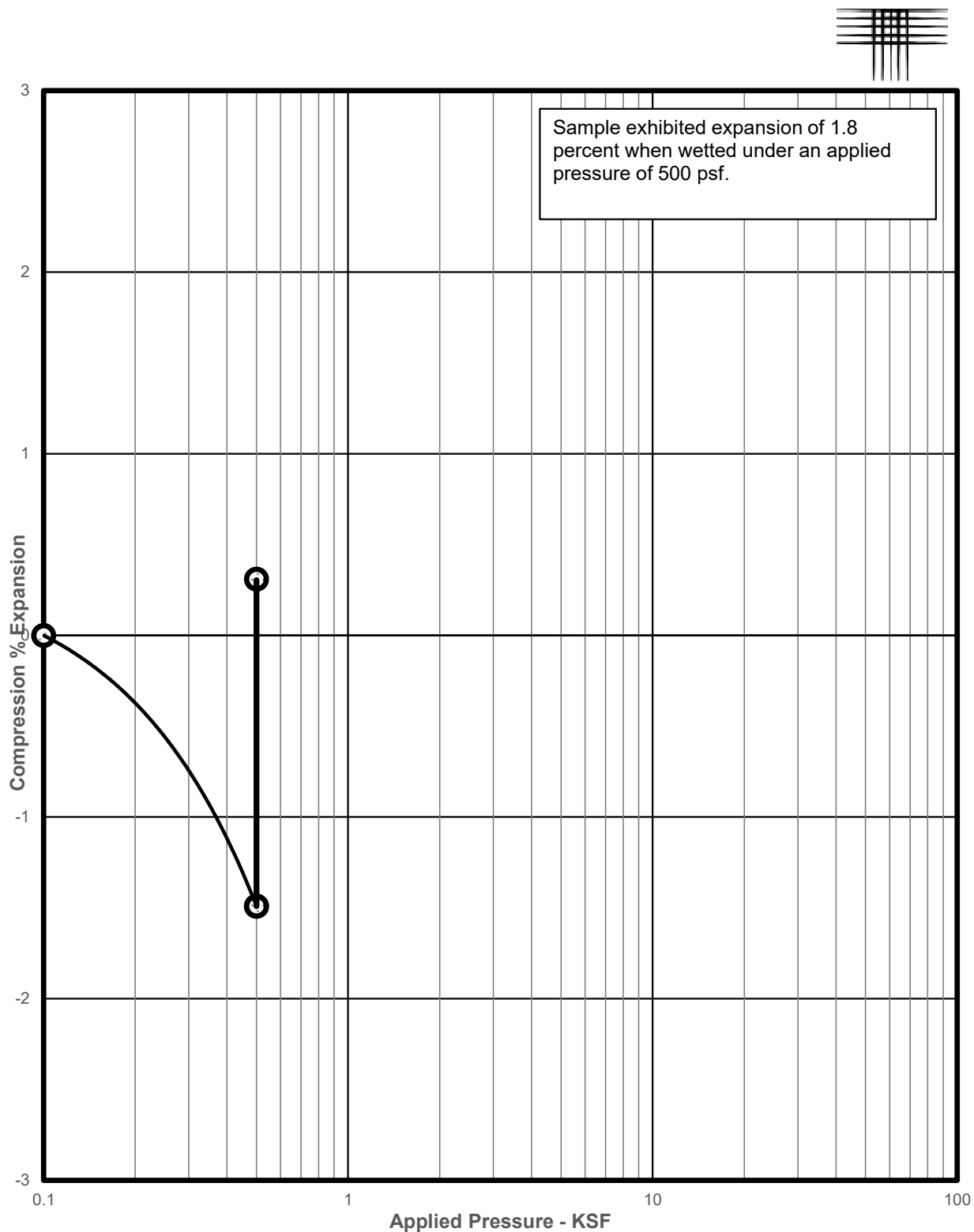


SAMPLE OF: CLAYSTONE
 FROM: TH-9 AT 19 FEET

DRY UNIT WEIGHT: 95 pcf
 MOISTURE CONTENT: 16.8 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 11

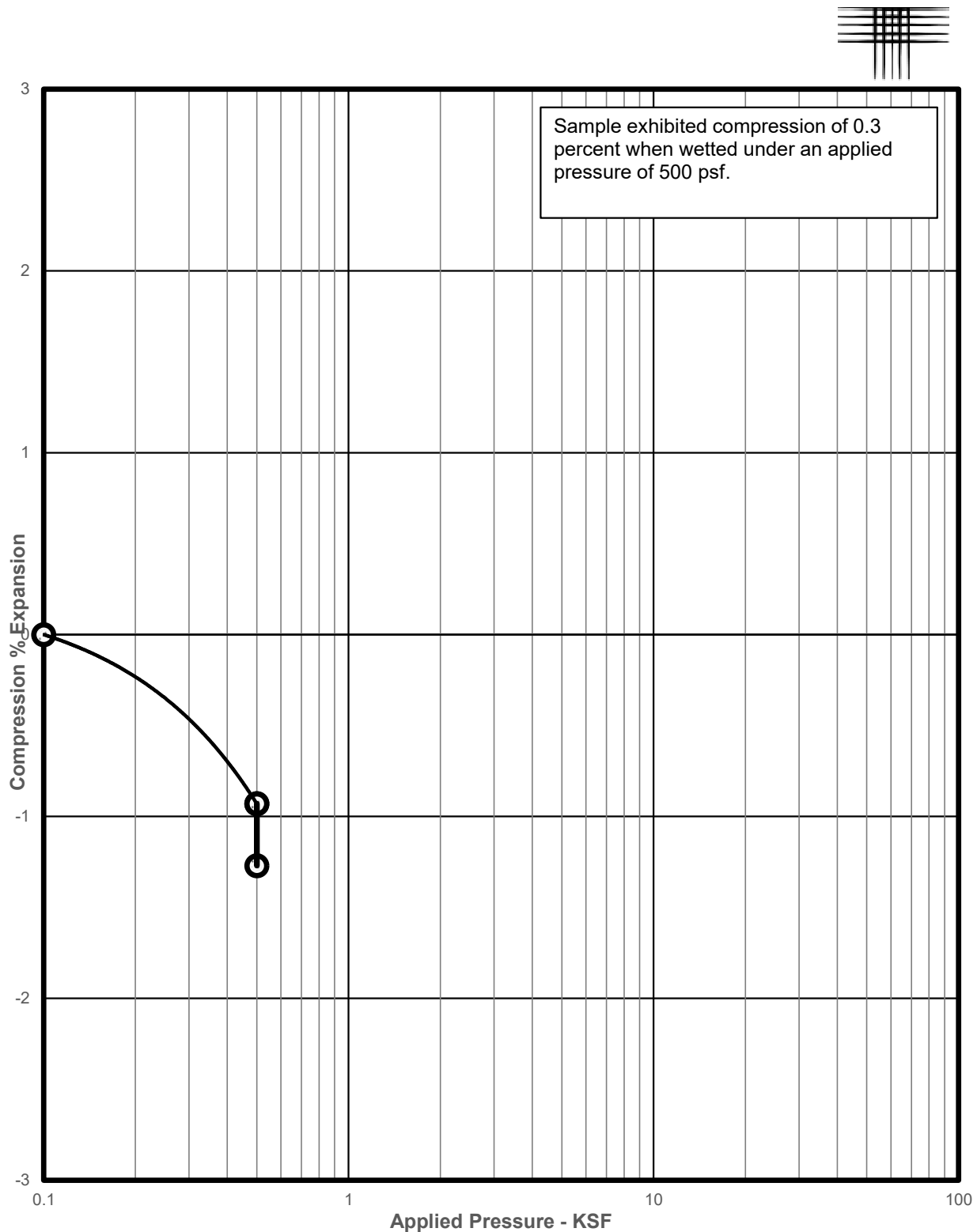


SAMPLE OF: CLAYSTONE
 FROM: TH-10 AT 9 FEET

DRY UNIT WEIGHT: 107 pcf
 MOISTURE CONTENT: 15.9 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 12

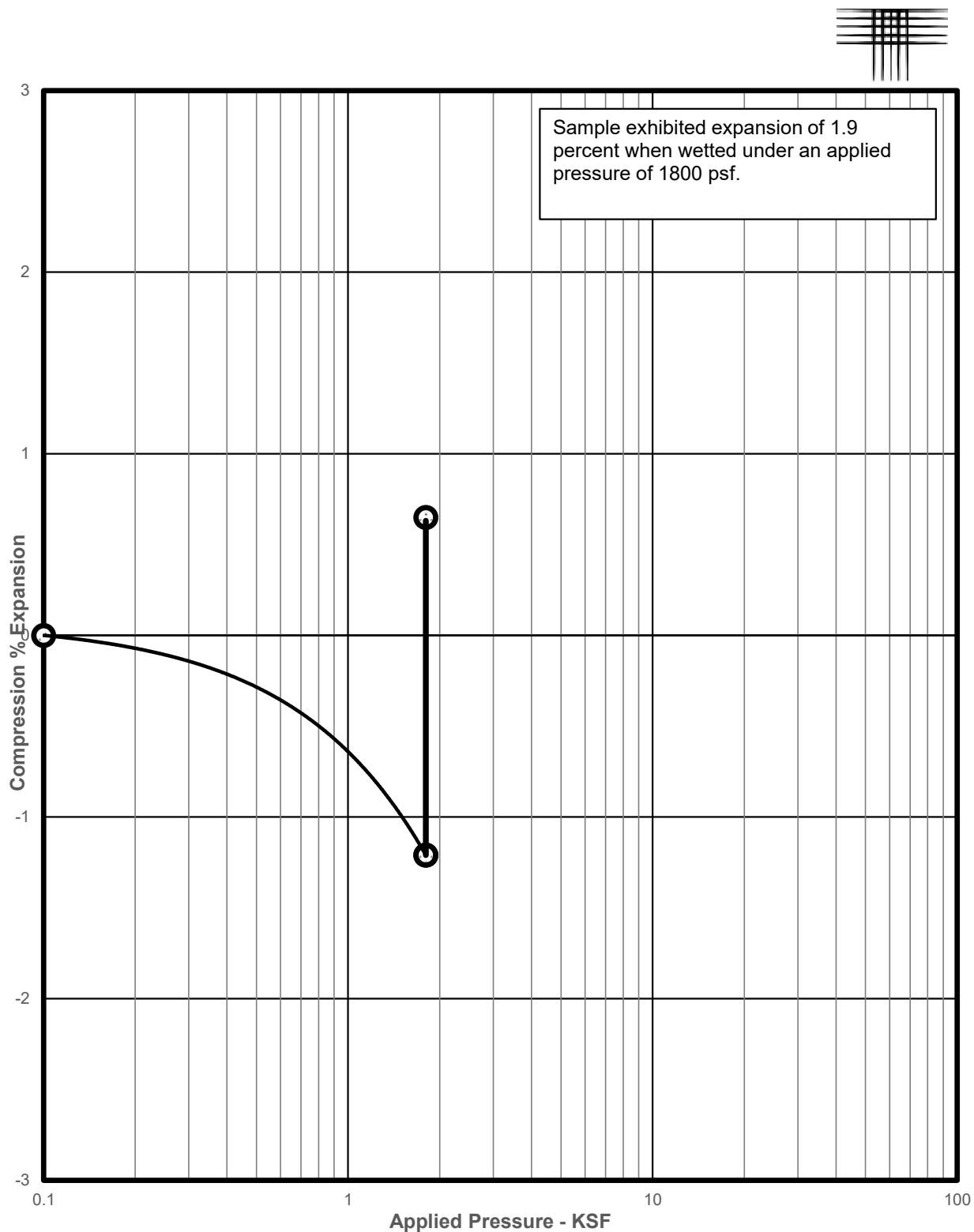


SAMPLE OF: CLAYSTONE
 FROM: TH-10 AT 14 FEET

DRY UNIT WEIGHT: 102 pcf
 MOISTURE CONTENT: 14.9 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 13

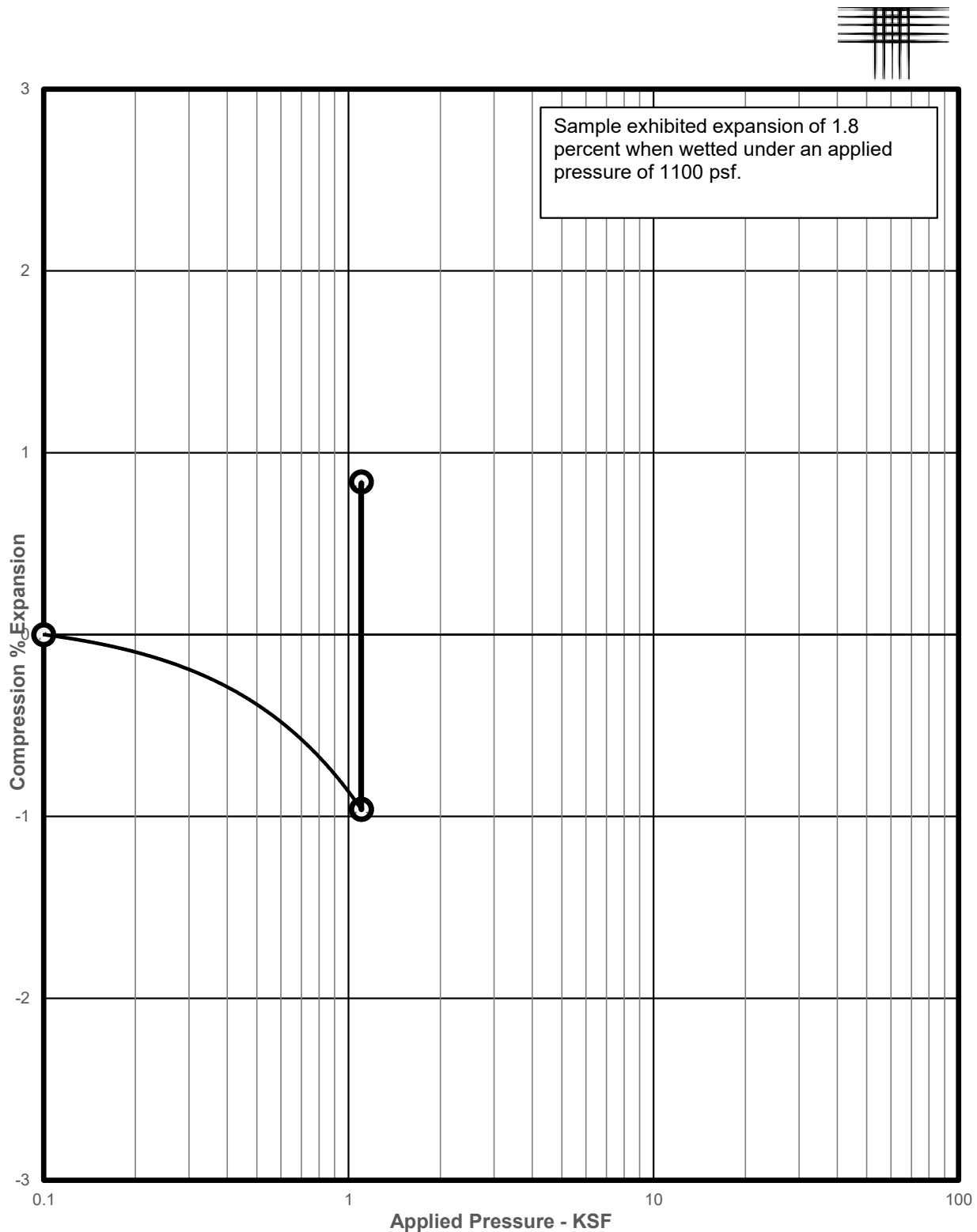


SAMPLE OF: CLAYSTONE
 FROM: TH-10 AT 29 FEET

DRY UNIT WEIGHT: 111 pcf
 MOISTURE CONTENT: 14.5 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 14

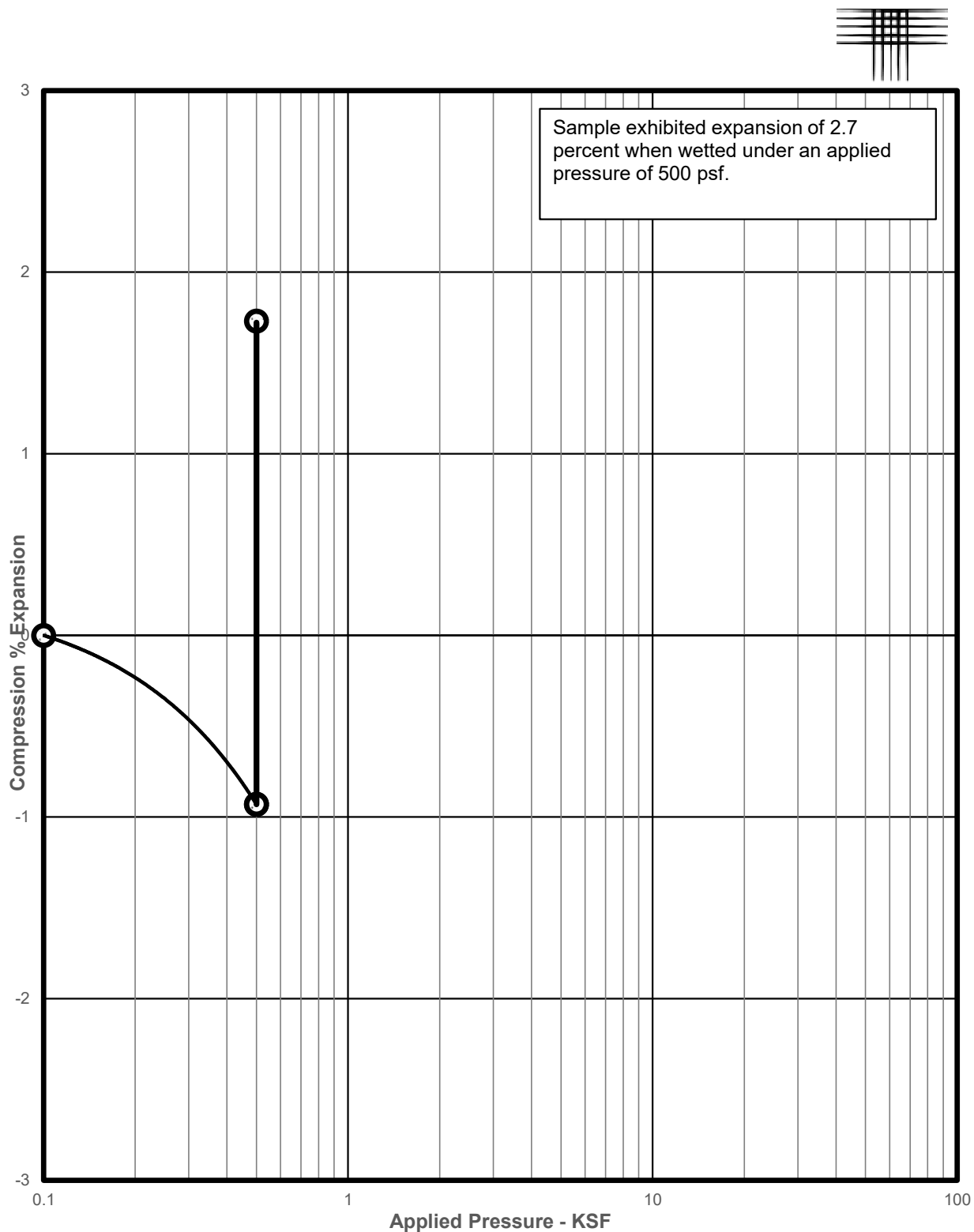


SAMPLE OF: CLAYSTONE
 FROM: TH-11 AT 19 FEET

DRY UNIT WEIGHT: 111 pcf
 MOISTURE CONTENT: 14.0 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 15

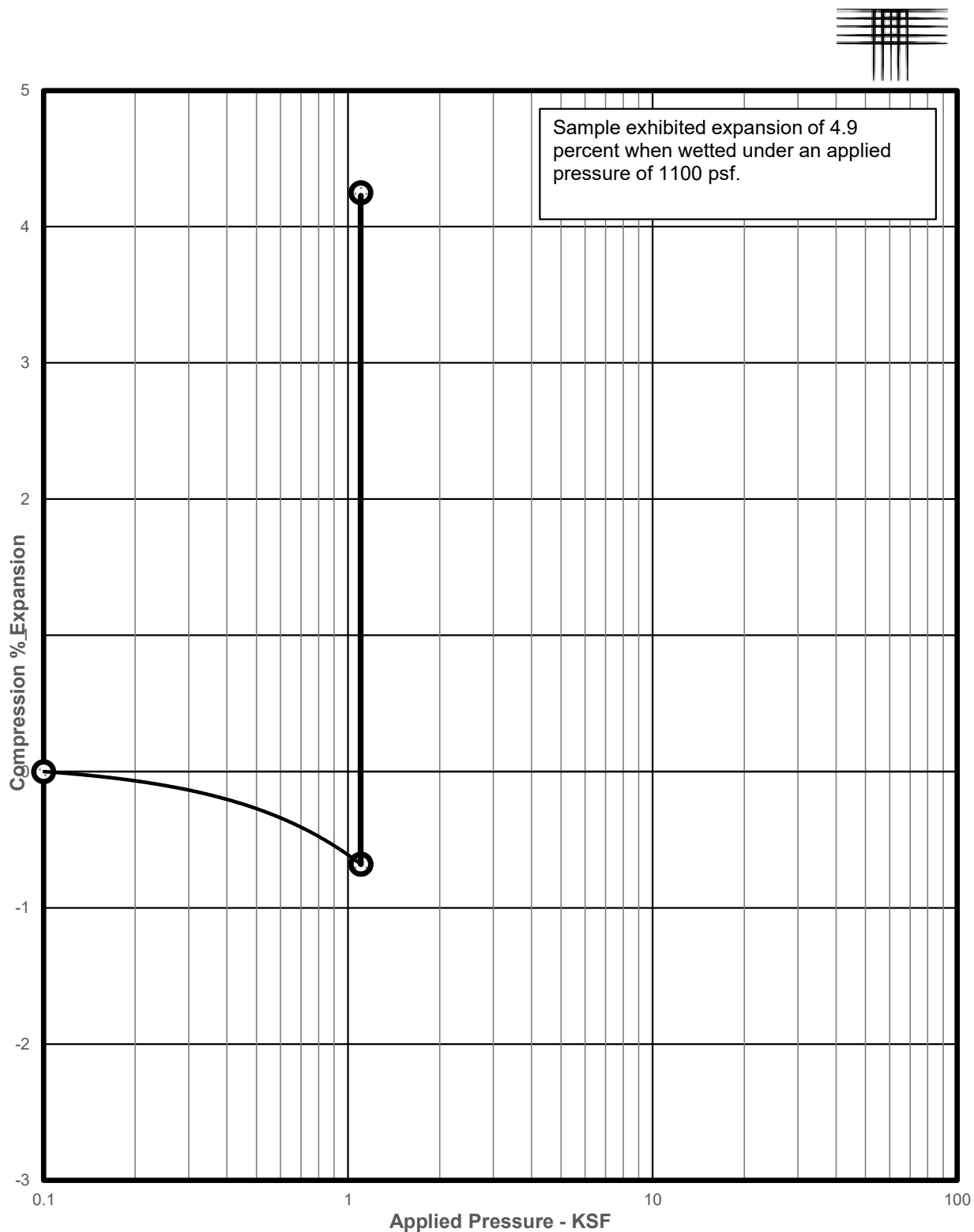


SAMPLE OF: CLAYSTONE
 FROM: TH-12 AT 14 FEET

DRY UNIT WEIGHT: 104 pcf
 MOISTURE CONTENT: 17.8 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 16

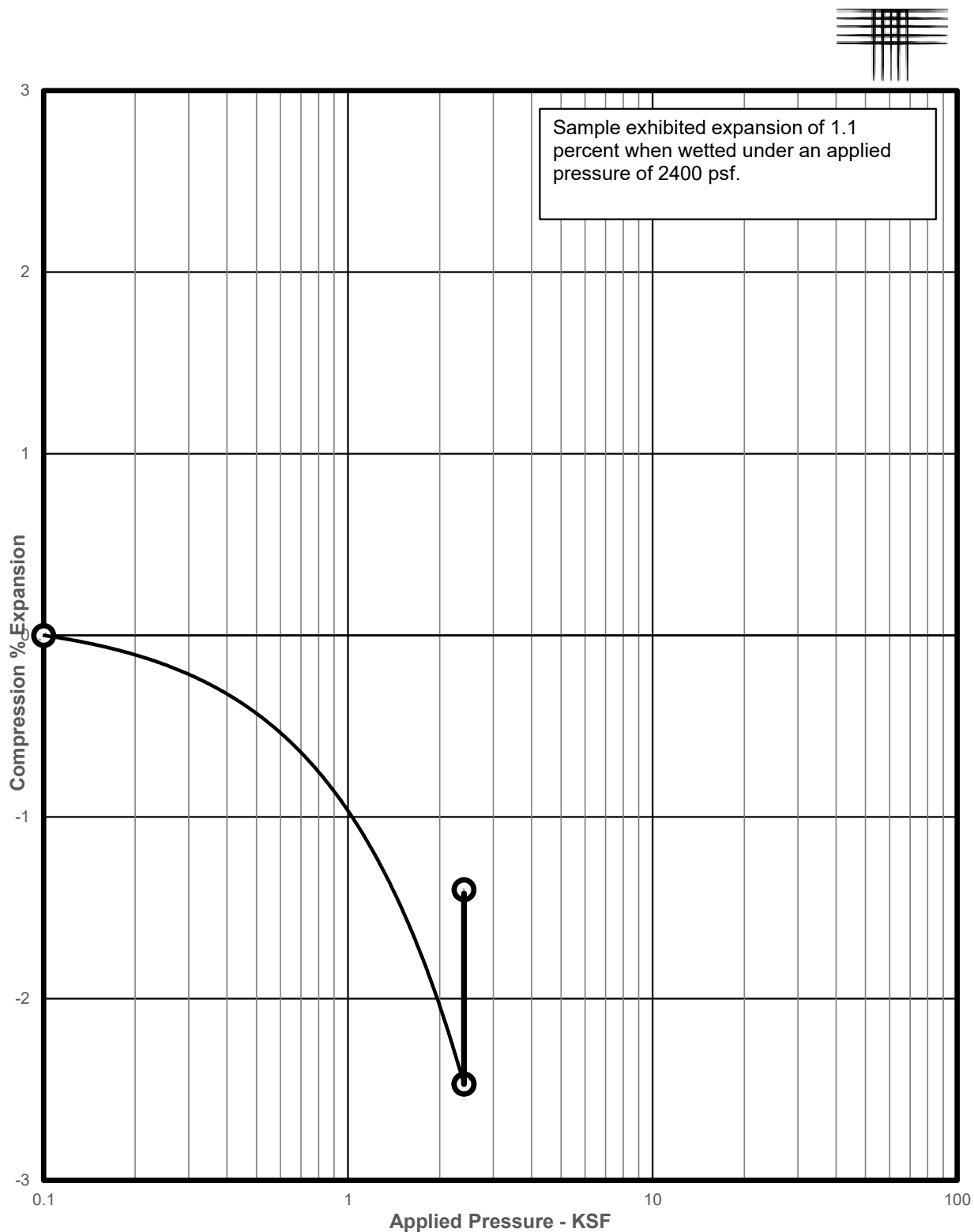


SAMPLE OF: CLAYSTONE
 FROM: TH-13 AT 24 FEET

DRY UNIT WEIGHT: 118 pcf
 MOISTURE CONTENT: 15.6 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 17

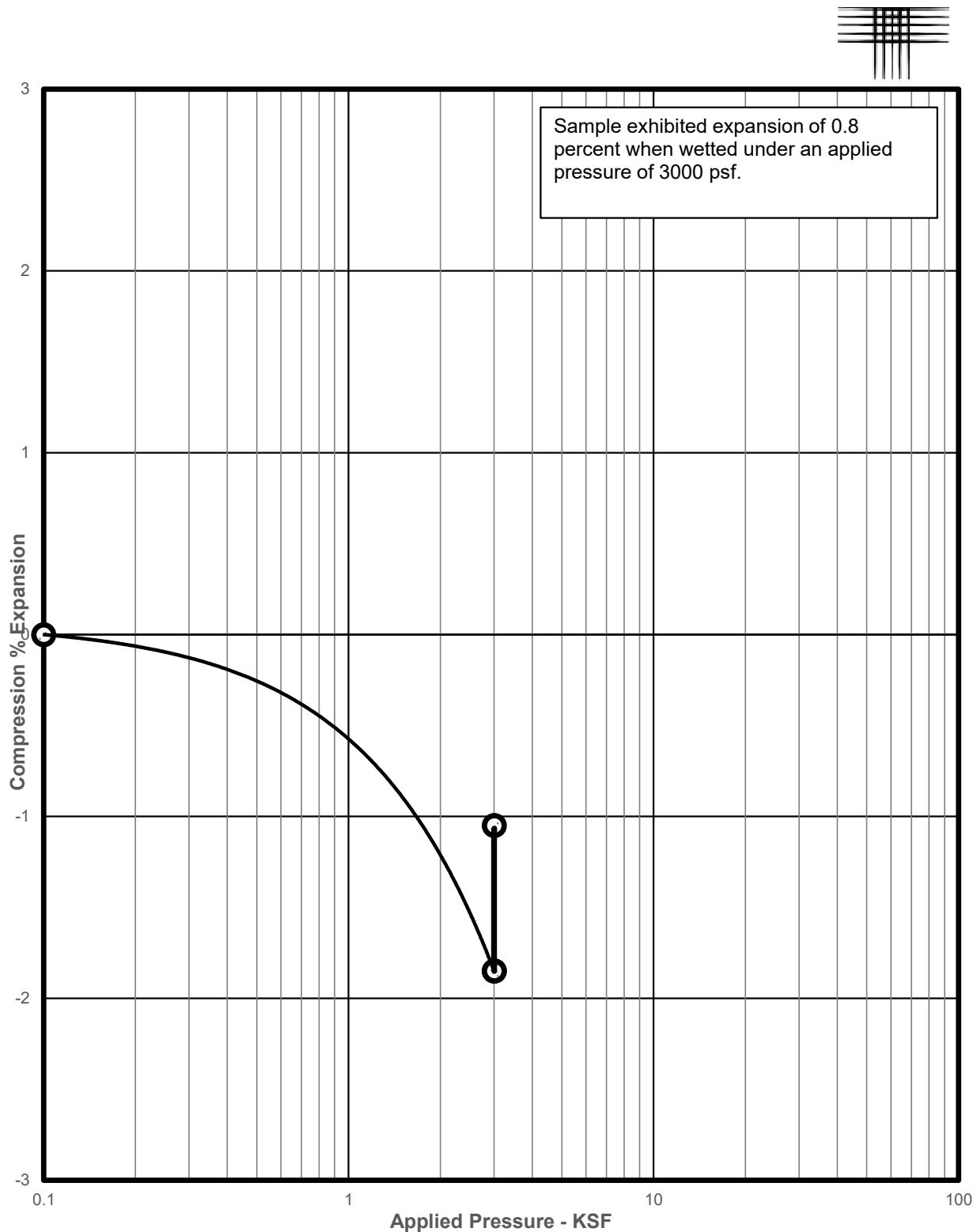


SAMPLE OF: CLAYSTONE
 FROM: TH-13 AT 34 FEET

DRY UNIT WEIGHT: 106 pcf
 MOISTURE CONTENT: 18.9 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 18

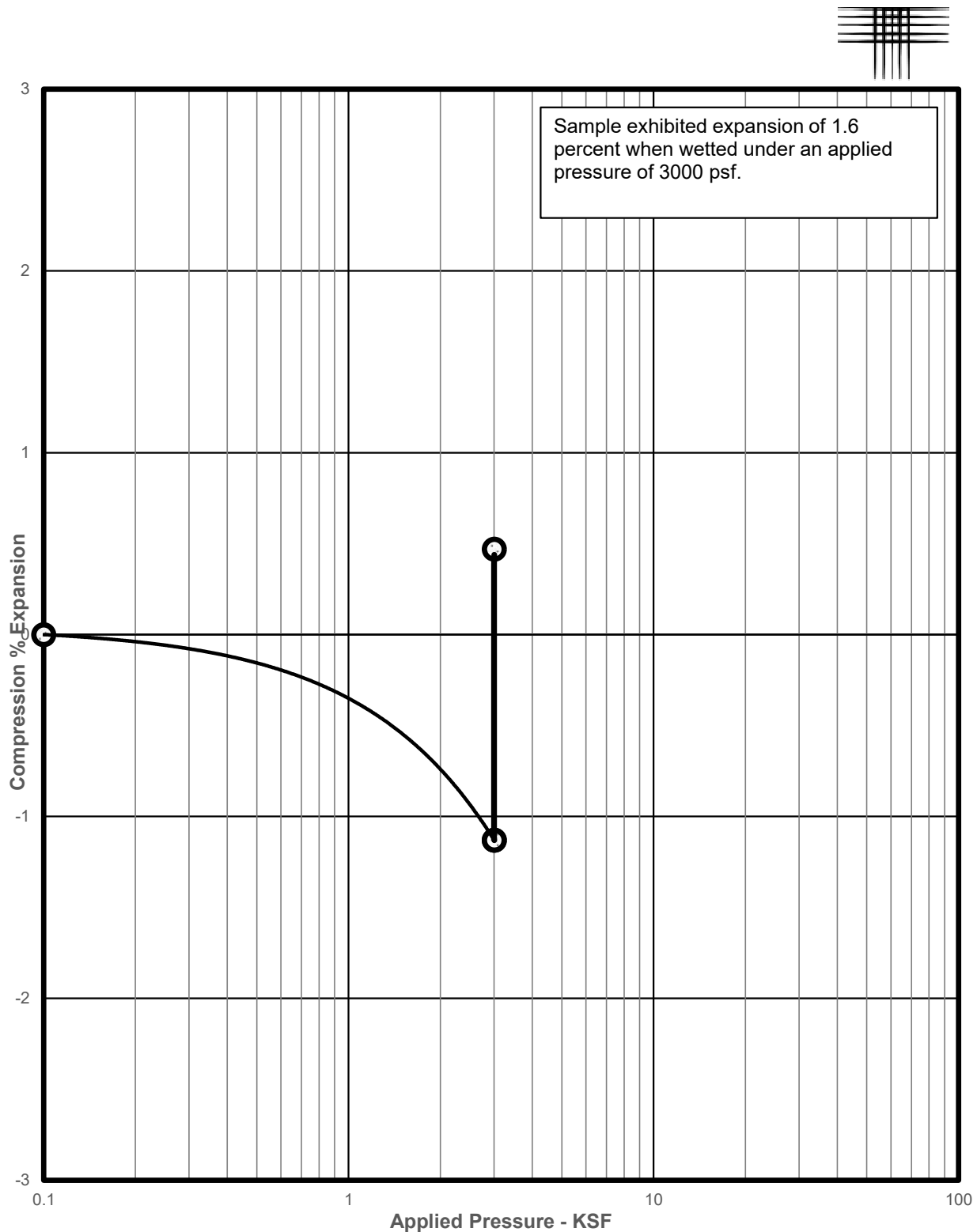


SAMPLE OF: CLAYSTONE
 FROM: TH-13 AT 39 FEET

DRY UNIT WEIGHT: 112 pcf
 MOISTURE CONTENT: 14.9 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 19

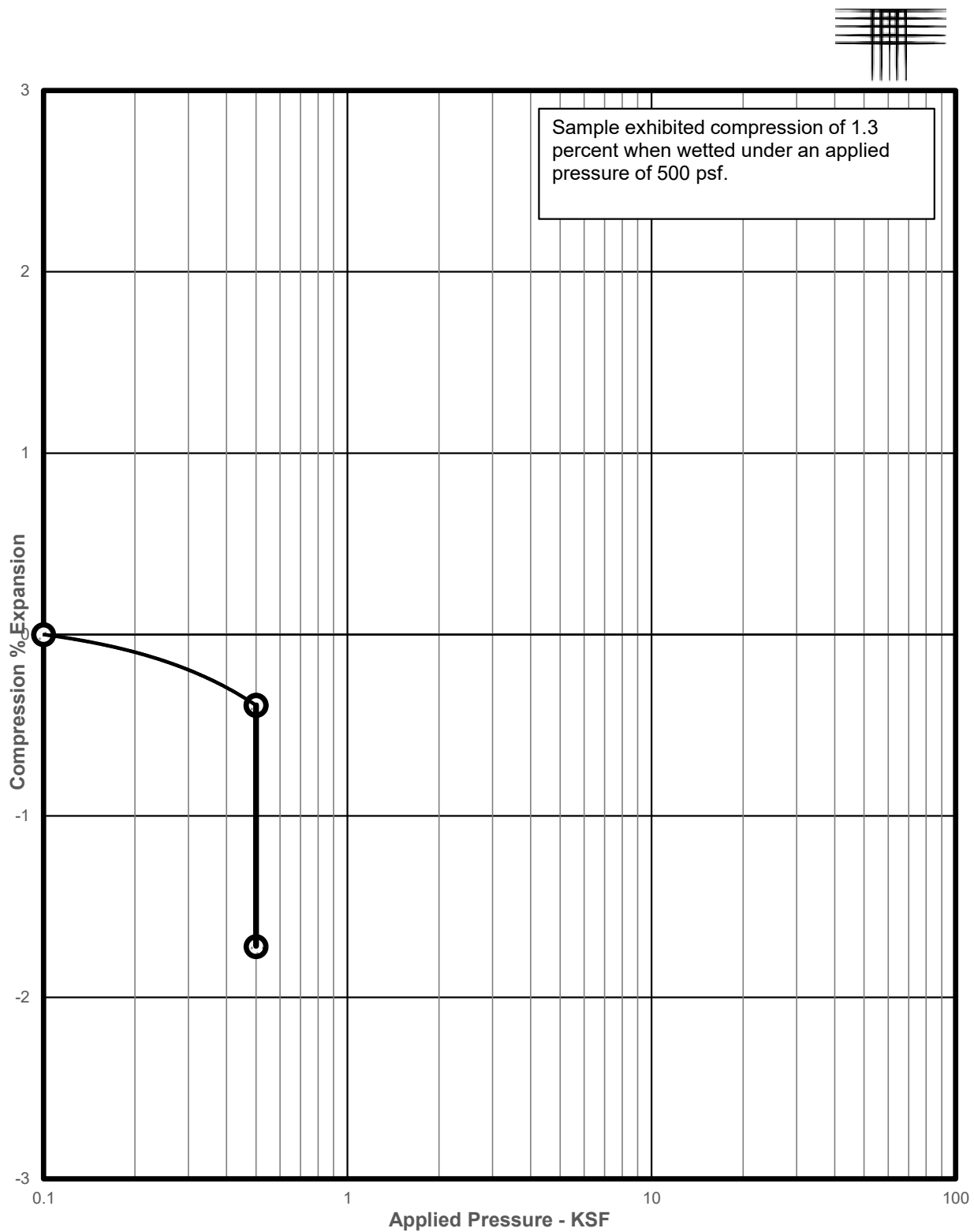


SAMPLE OF: CLAYSTONE
 FROM: TH-14 AT 34 FEET

DRY UNIT WEIGHT: 118 pcf
 MOISTURE CONTENT: 16.4 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 20

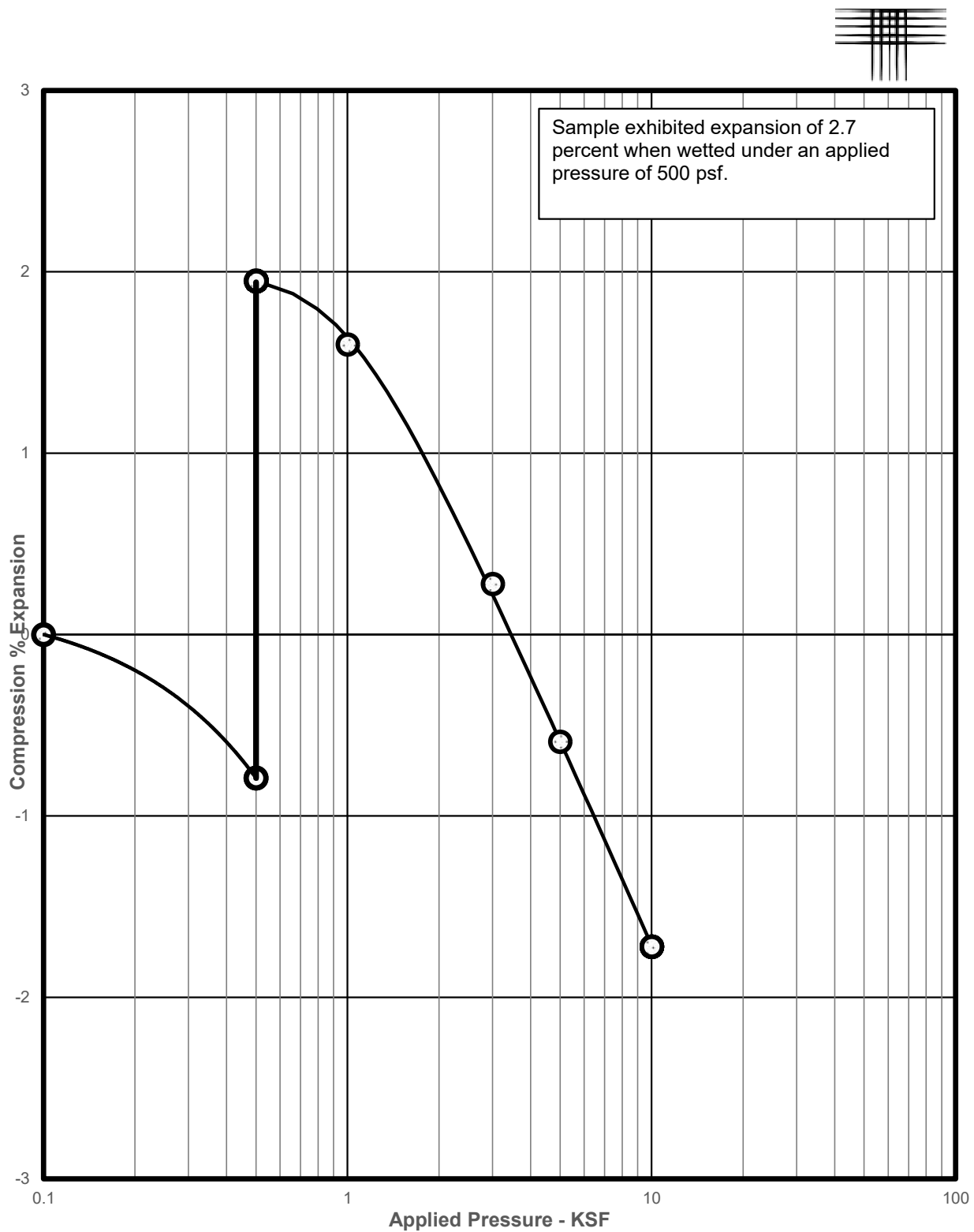


SAMPLE OF: SAND, CLAYEY, GRAVELLY (SC)
 FROM: TH-15 AT 9 FEET

DRY UNIT WEIGHT: 109 pcf
 MOISTURE CONTENT: 4.2 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 21

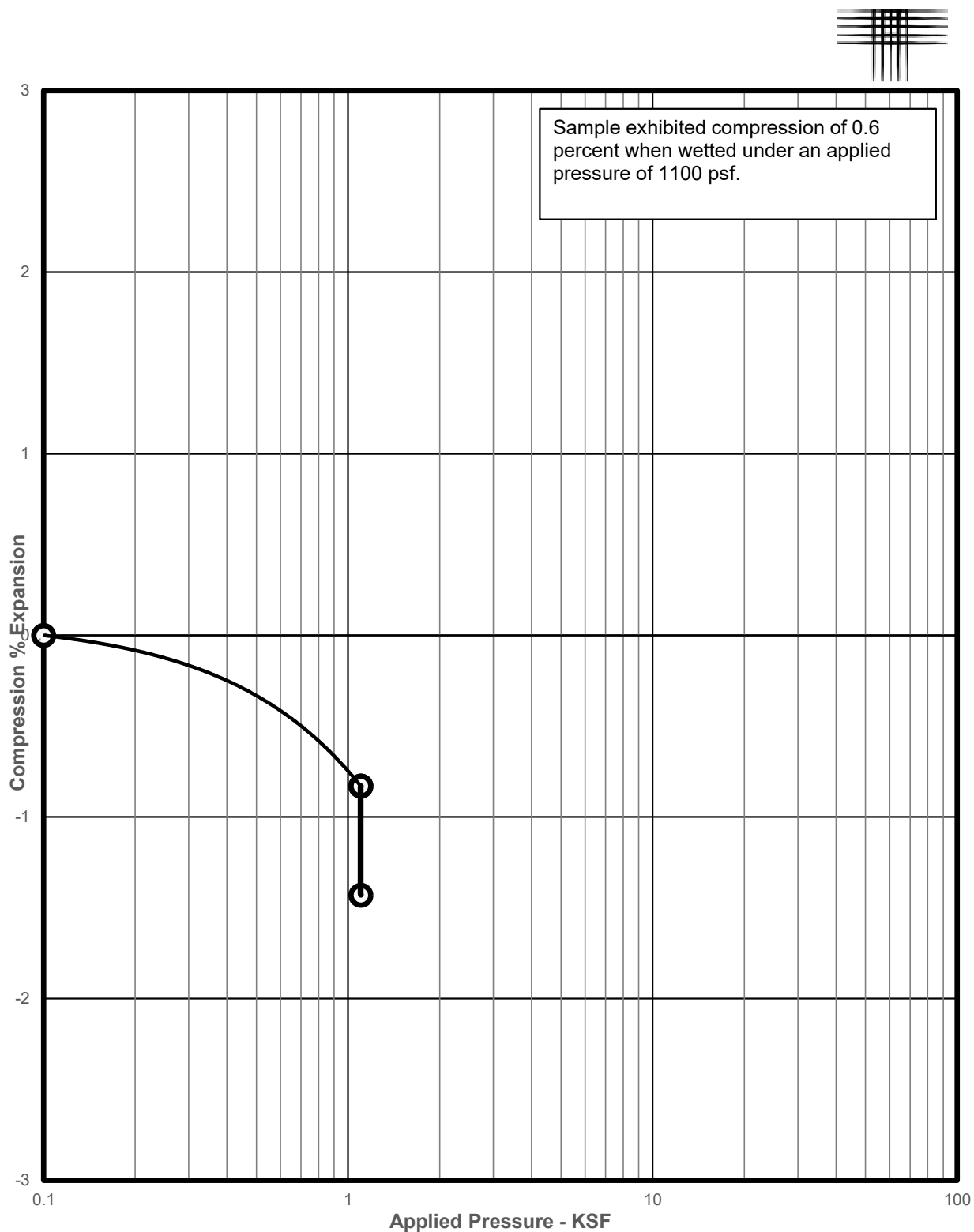


SAMPLE OF: CLAY, SANDY (CL)
 FROM: TH-15 AT 14 FEET

DRY UNIT WEIGHT: 116 pcf
 MOISTURE CONTENT: 11.0 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 22

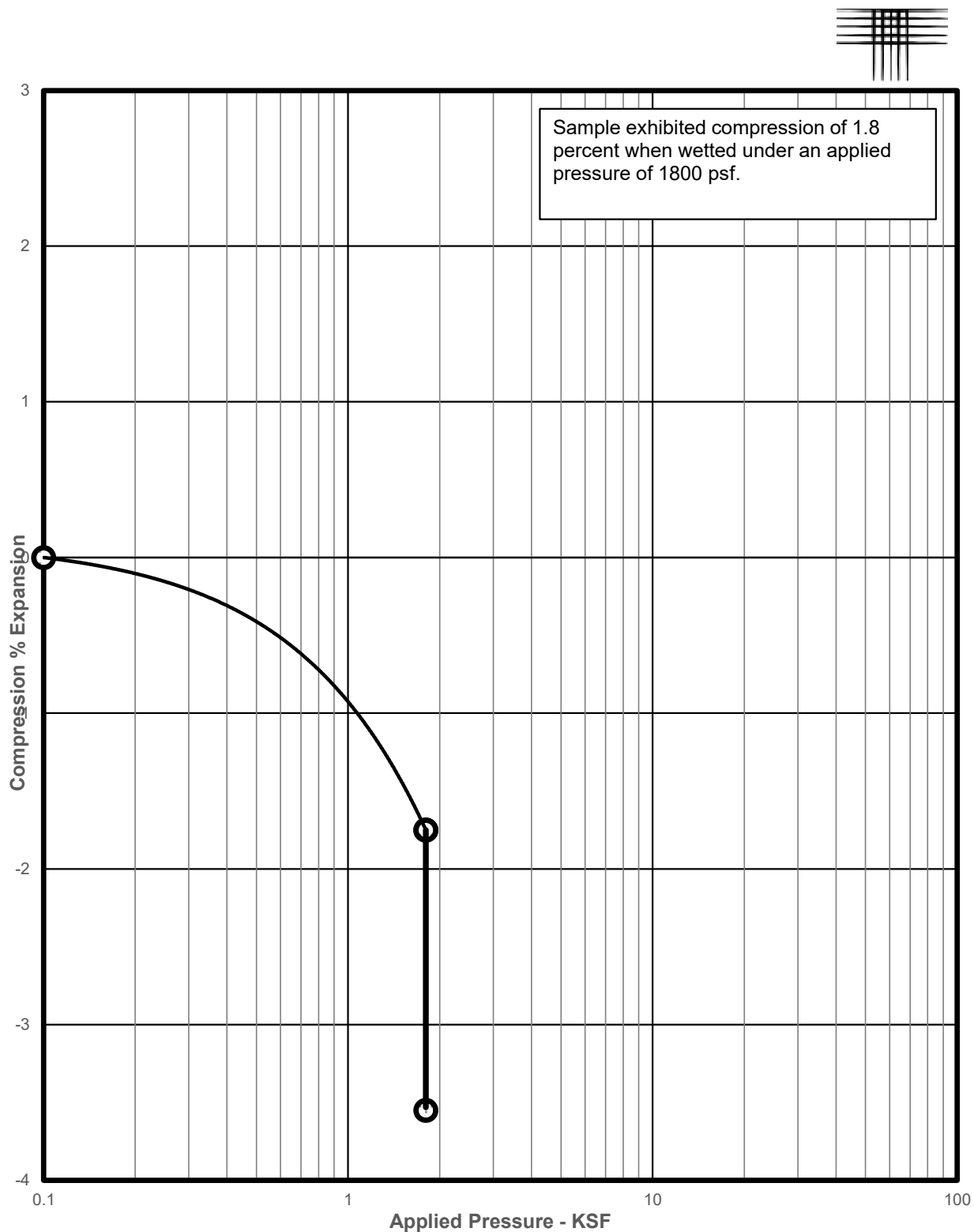


SAMPLE OF: WEATHERED SANDSTONE
 FROM: TH-15 AT 19 FEET

DRY UNIT WEIGHT: 107 pcf
 MOISTURE CONTENT: 7.8 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 23

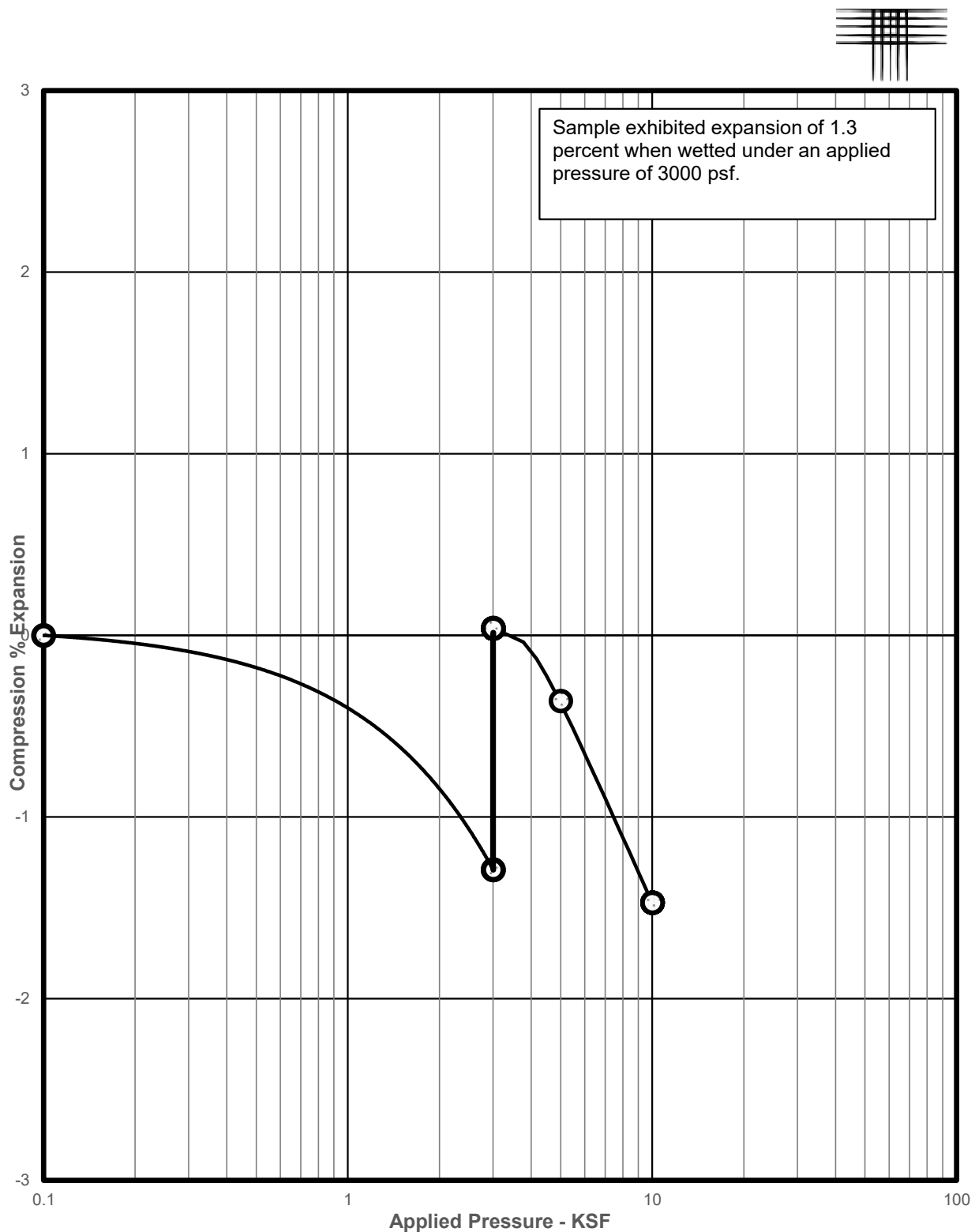


SAMPLE OF: CLAYSTONE
 FROM: TH-15 AT 24 FEET

DRY UNIT WEIGHT: 98 pcf
 MOISTURE CONTENT: 17.3 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 24

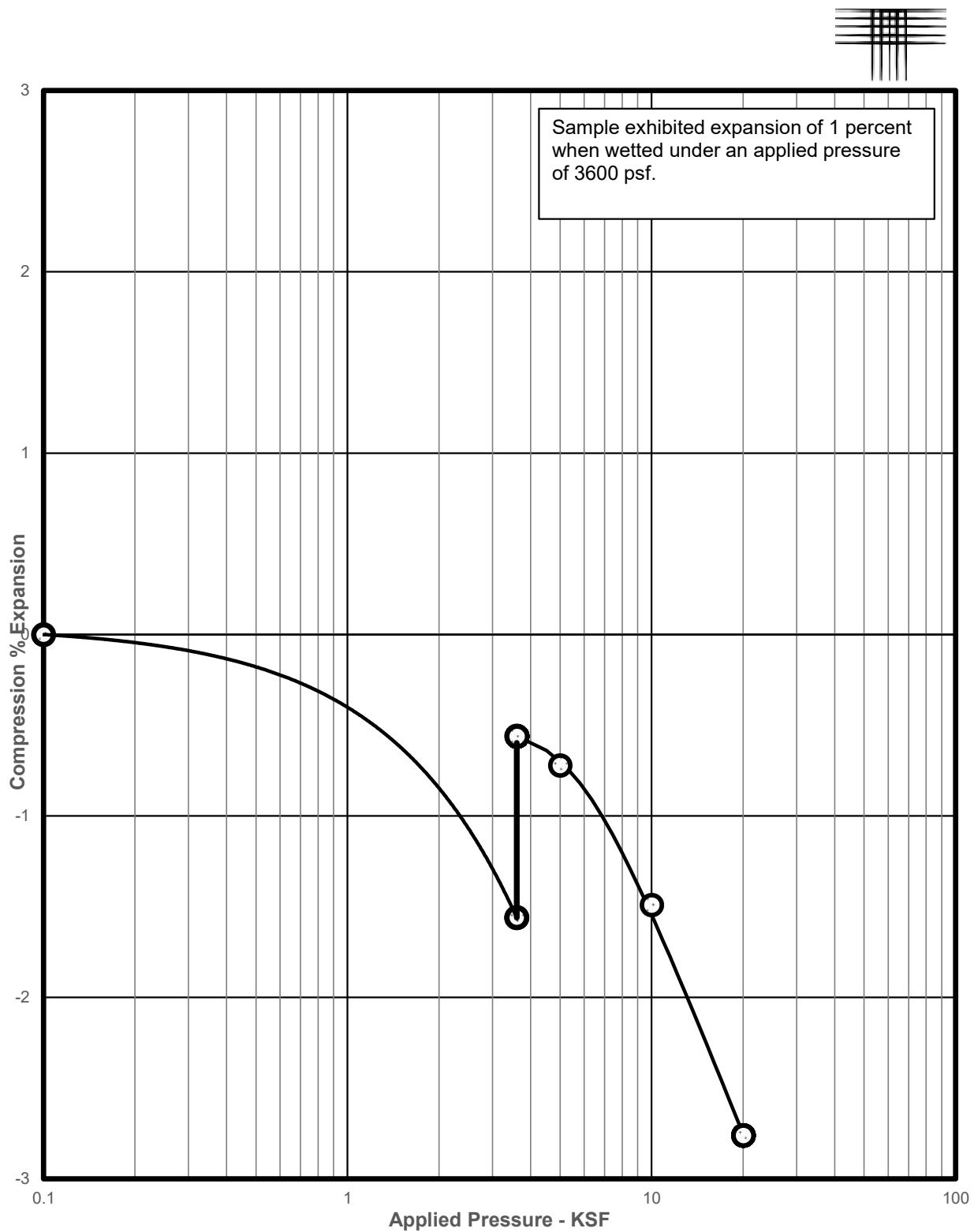


SAMPLE OF: CLAYSTONE
 FROM: TH-15 AT 34 FEET

DRY UNIT WEIGHT: 114 pcf
 MOISTURE CONTENT: 15.1 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 25

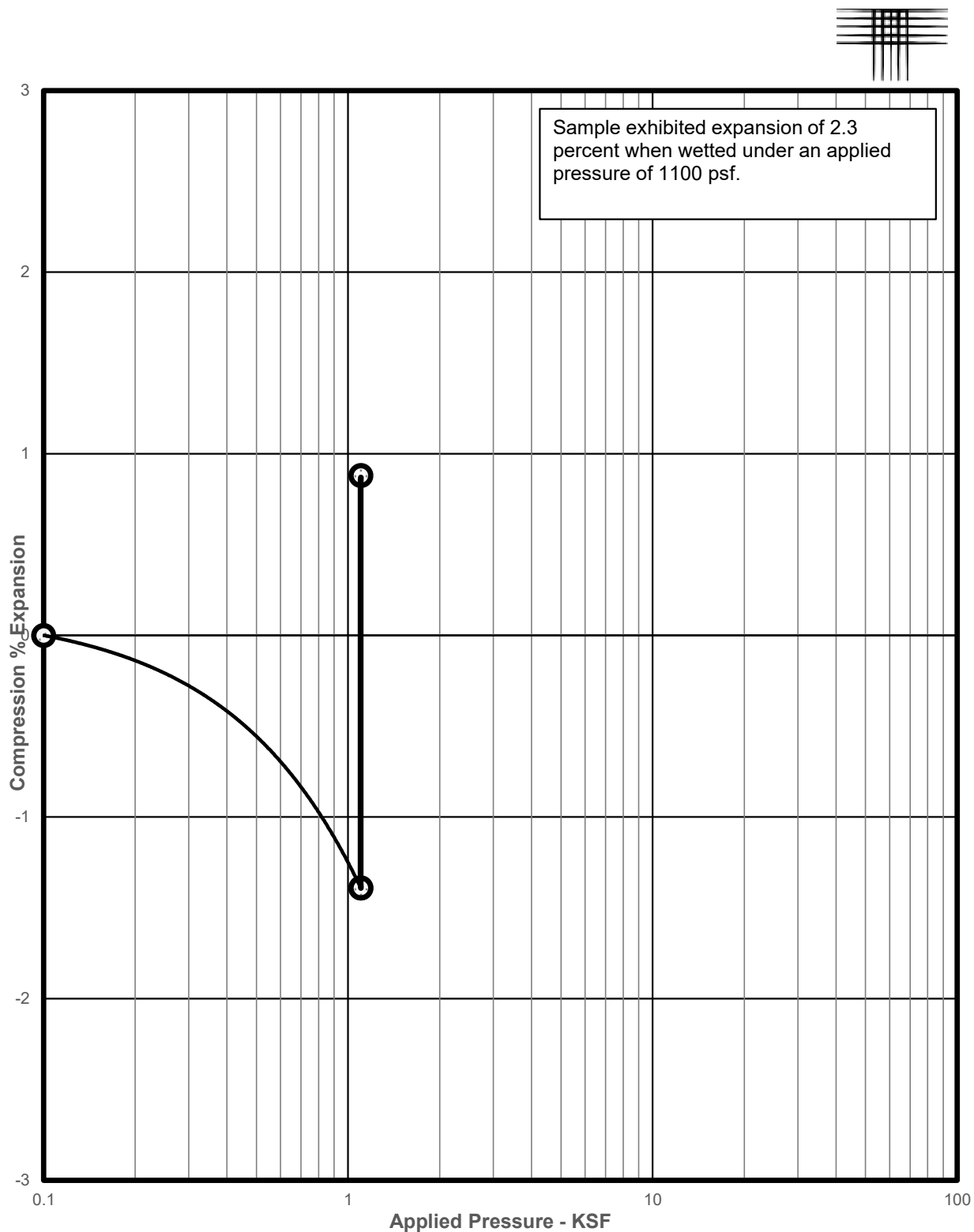


SAMPLE OF: CLAYSTONE
 FROM: TH-15 AT 39 FEET

DRY UNIT WEIGHT: 115 pcf
 MOISTURE CONTENT: 14.7 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 26

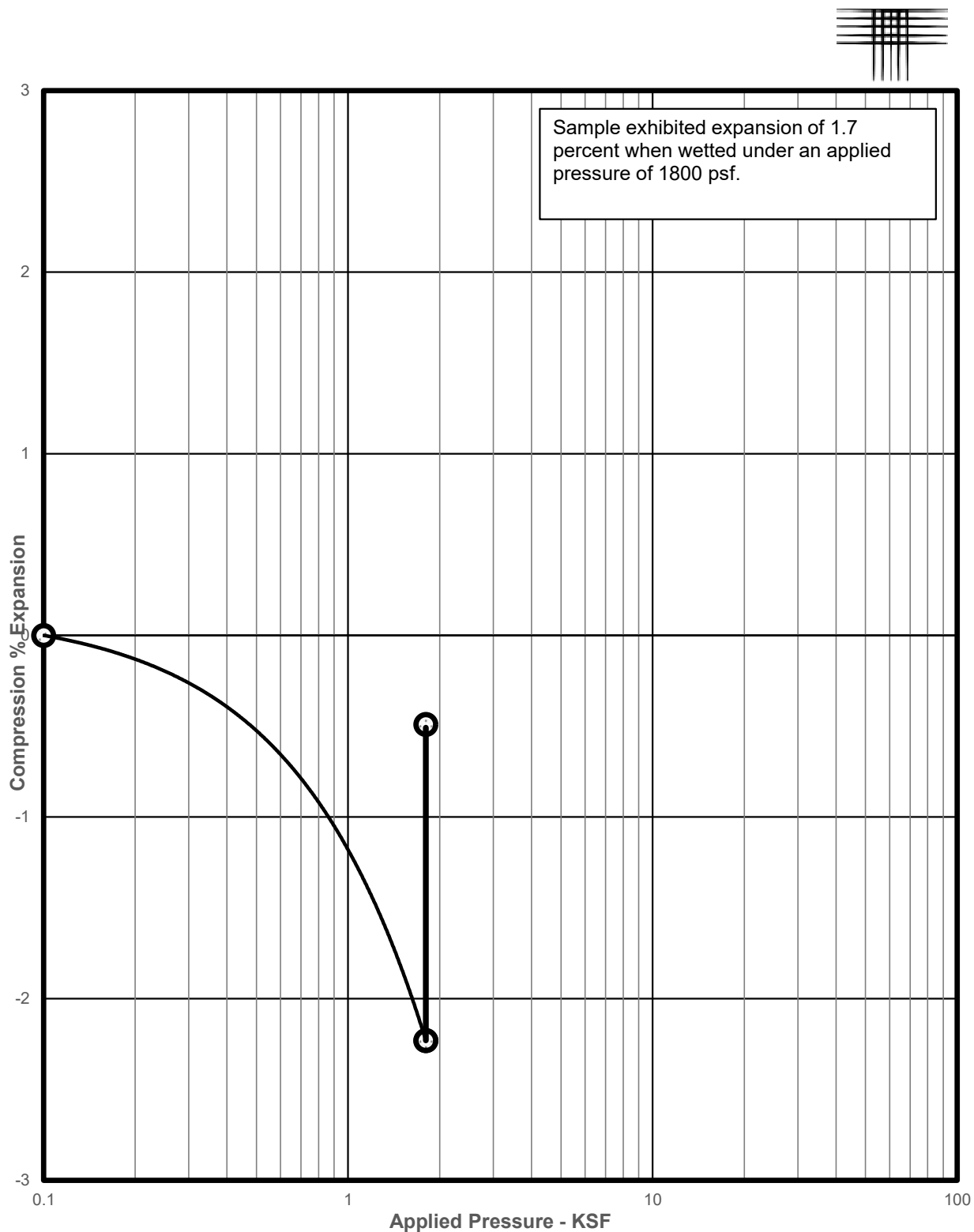


SAMPLE OF: CLAYSTONE
 FROM: TH-16 AT 14 FEET

DRY UNIT WEIGHT: 101 pcf
 MOISTURE CONTENT: 21.1 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 27

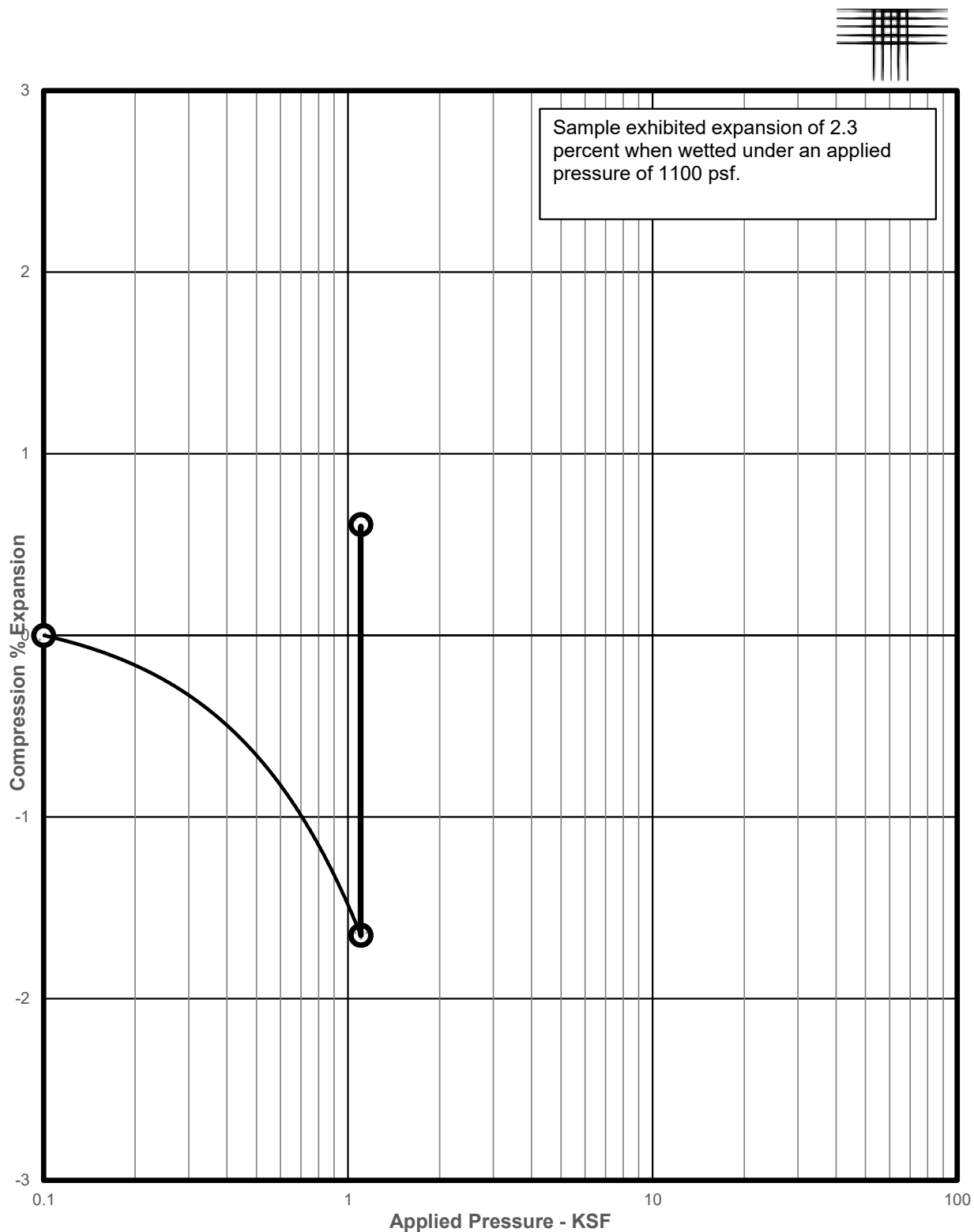


SAMPLE OF: CLAYSTONE
 FROM: TH-16 AT 19 FEET

DRY UNIT WEIGHT: 100 pcf
 MOISTURE CONTENT: 24.4 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 28

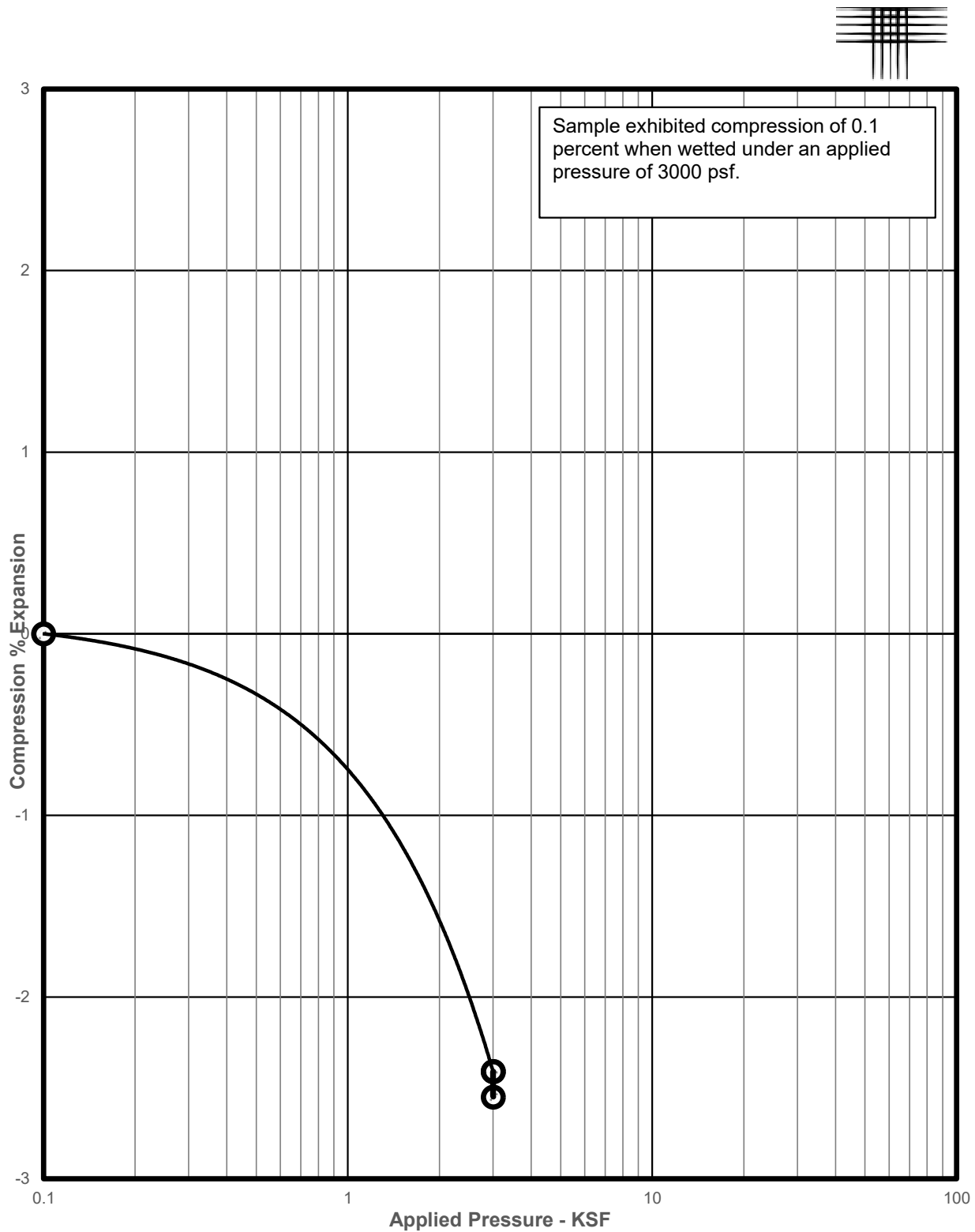


SAMPLE OF: CLAYSTONE
 FROM: TH-19 AT 9 FEET

DRY UNIT WEIGHT: 114 pcf
 MOISTURE CONTENT: 12.2 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 29

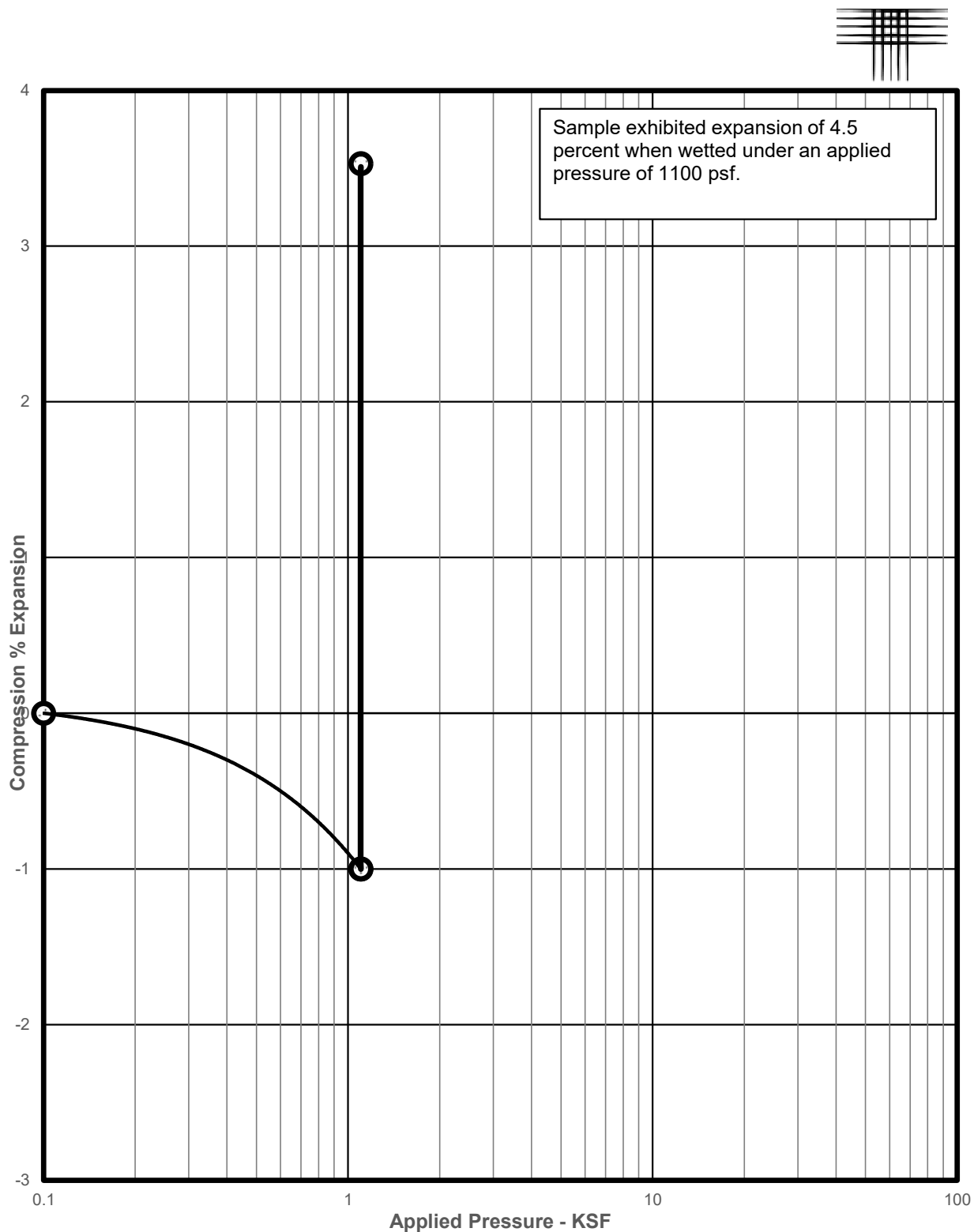


SAMPLE OF: CLAYSTONE
 FROM: TH-21 AT 29 FEET

DRY UNIT WEIGHT: 101 pcf
 MOISTURE CONTENT: 21.4 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 30

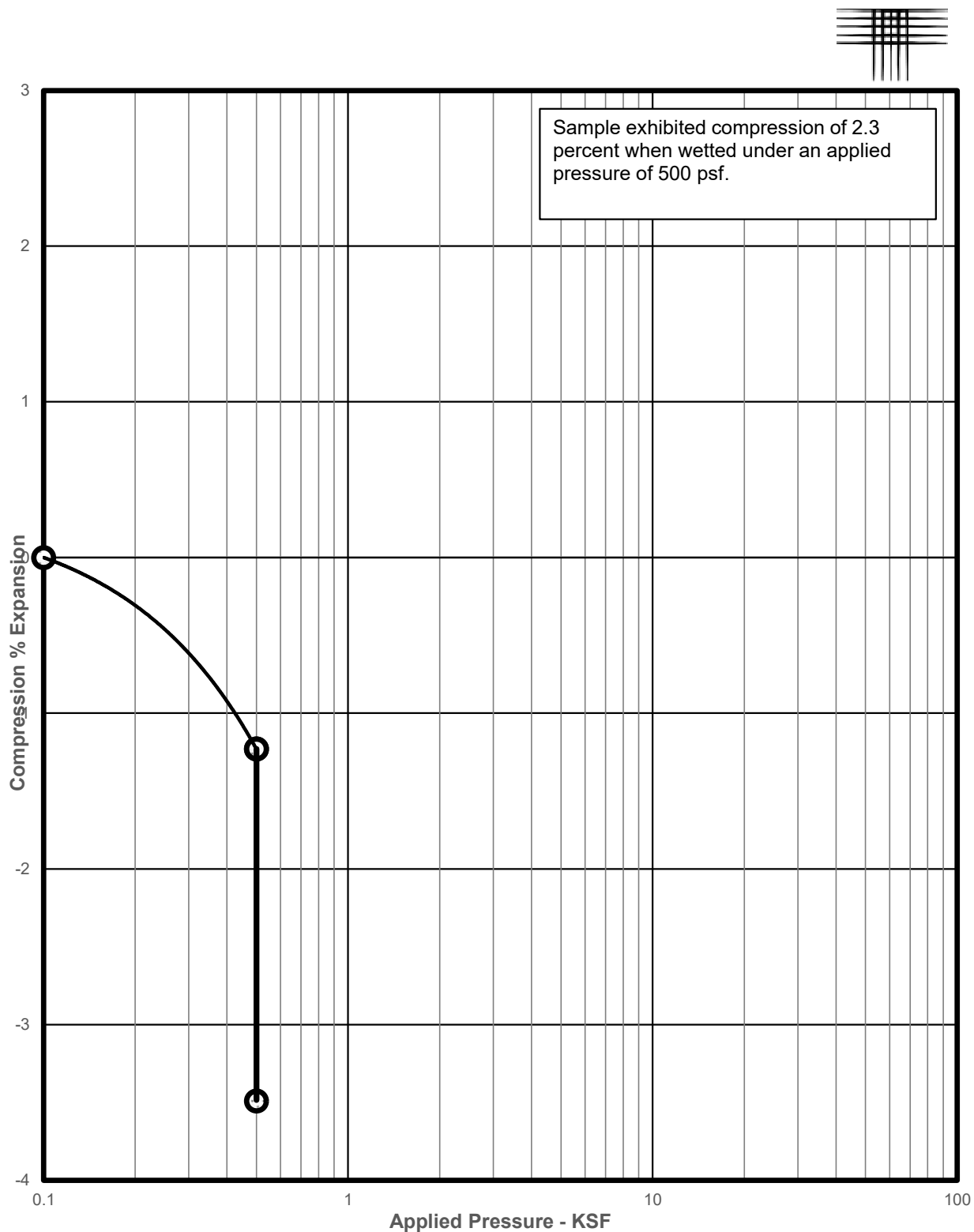


SAMPLE OF: CLAY, SANDY (CL)
 FROM: TH-22 AT 14 FEET

DRY UNIT WEIGHT: 115 pcf
 MOISTURE CONTENT: 11.9 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 31

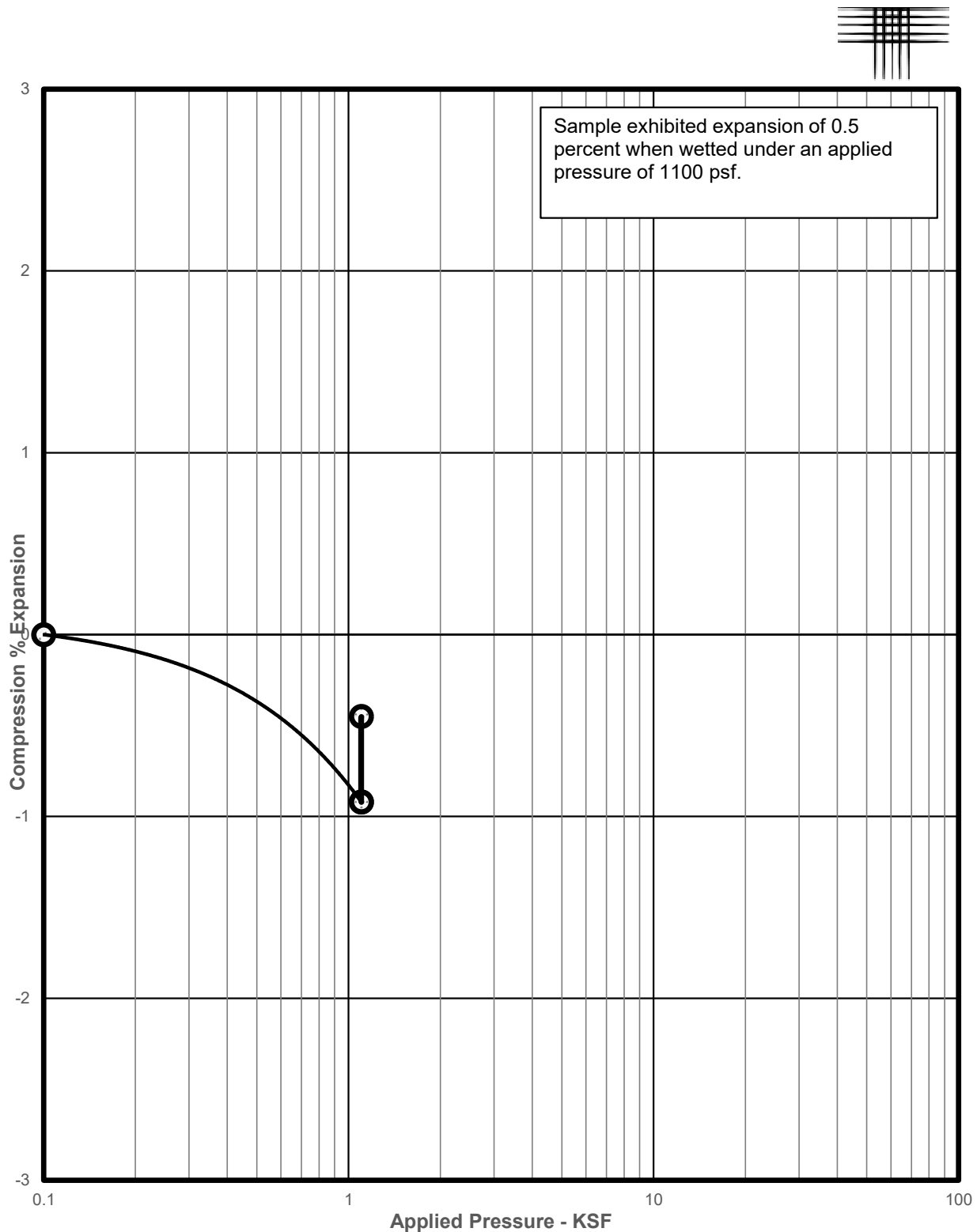


SAMPLE OF: SAND, CLAYEY (SC)
 FROM: TH-23 AT 4 FEET

DRY UNIT WEIGHT: 90 pcf
 MOISTURE CONTENT: 6.7 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 32

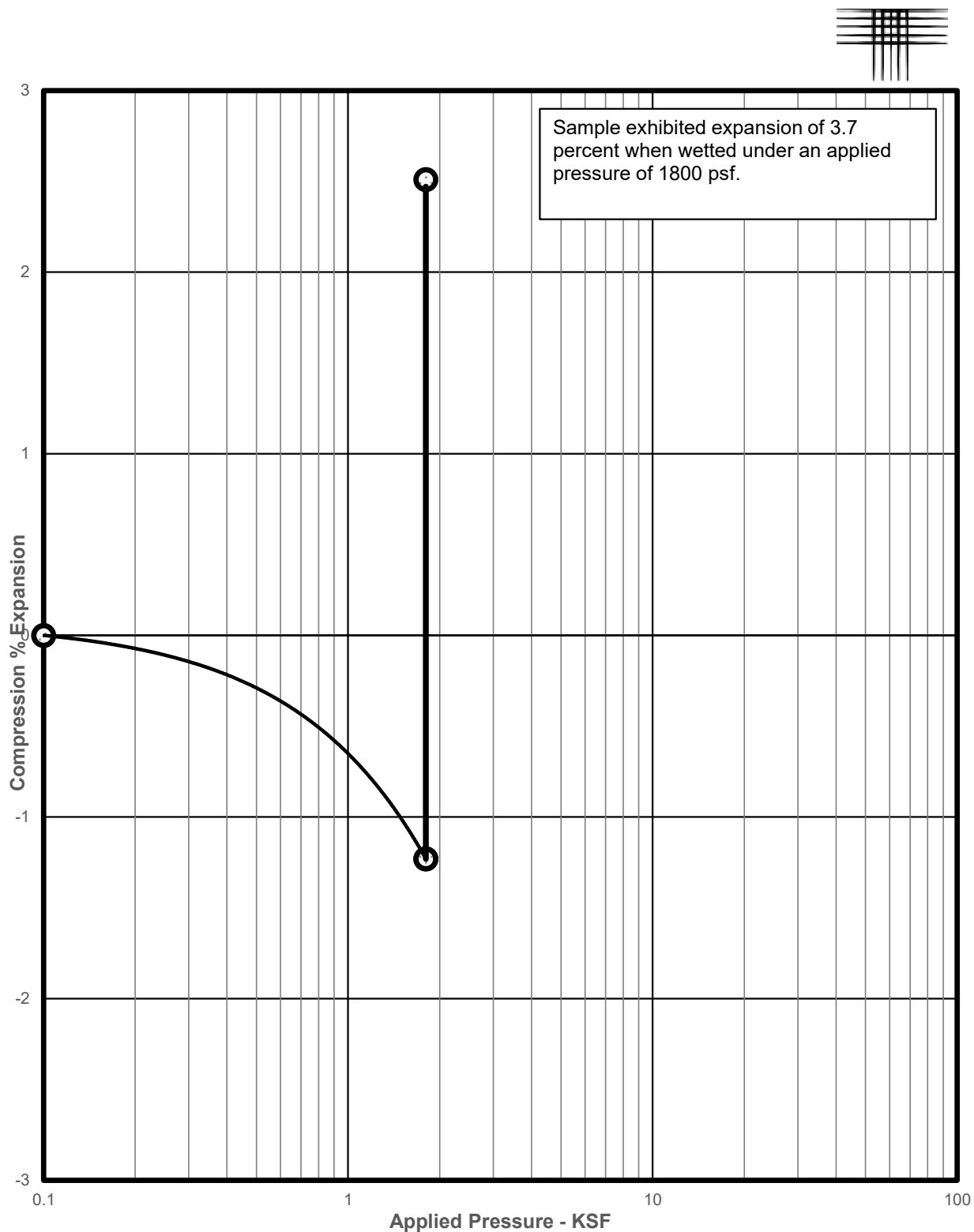


SAMPLE OF: SAND, CLAYEY (SC)
 FROM: TH-23 AT 9 FEET

DRY UNIT WEIGHT: 116 pcf
 MOISTURE CONTENT: 6.8 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 33

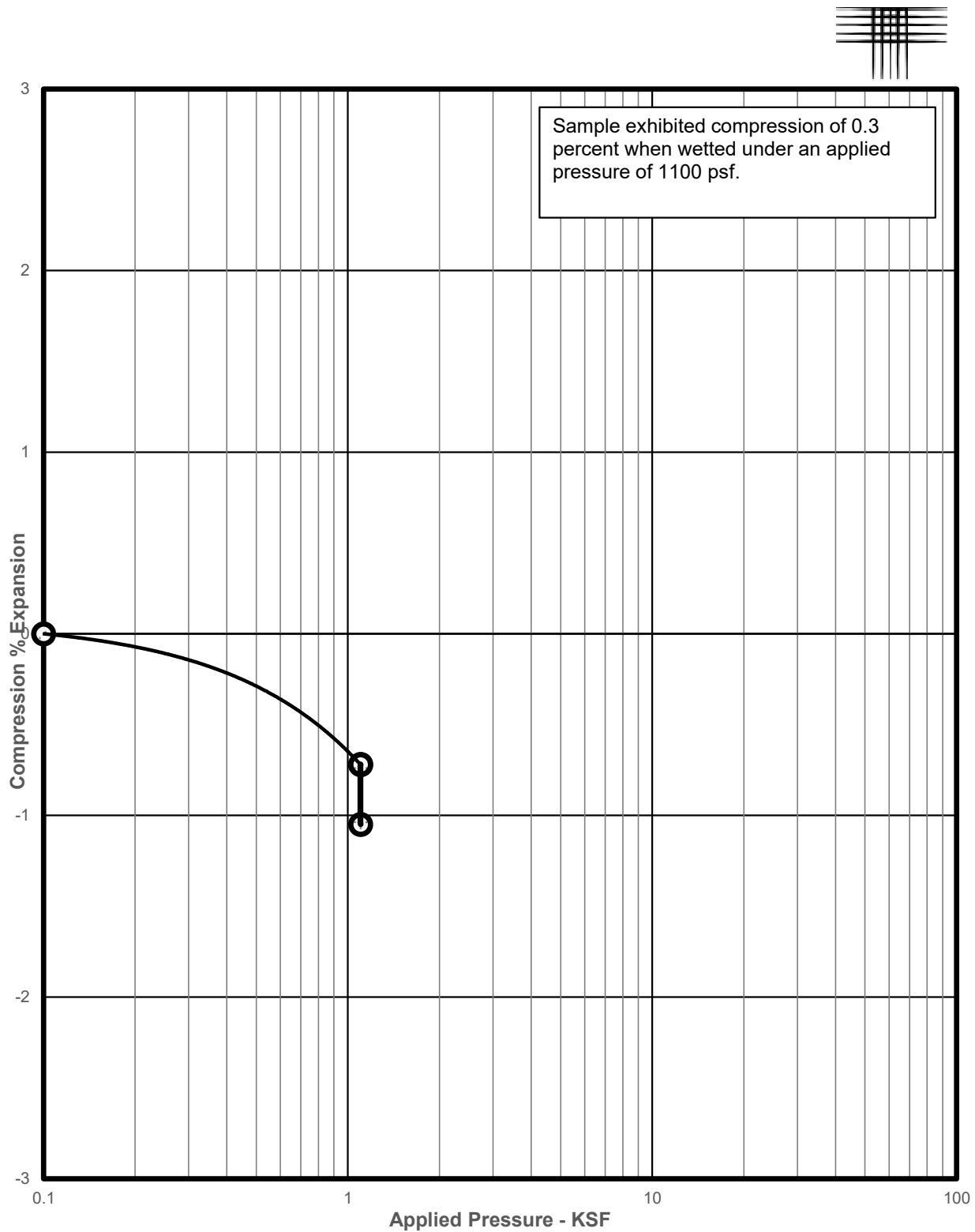


SAMPLE OF: CLAY, SANDY (CL)
 FROM: TH-23 AT 14 FEET

DRY UNIT WEIGHT: 115 pcf
 MOISTURE CONTENT: 14.0 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 34

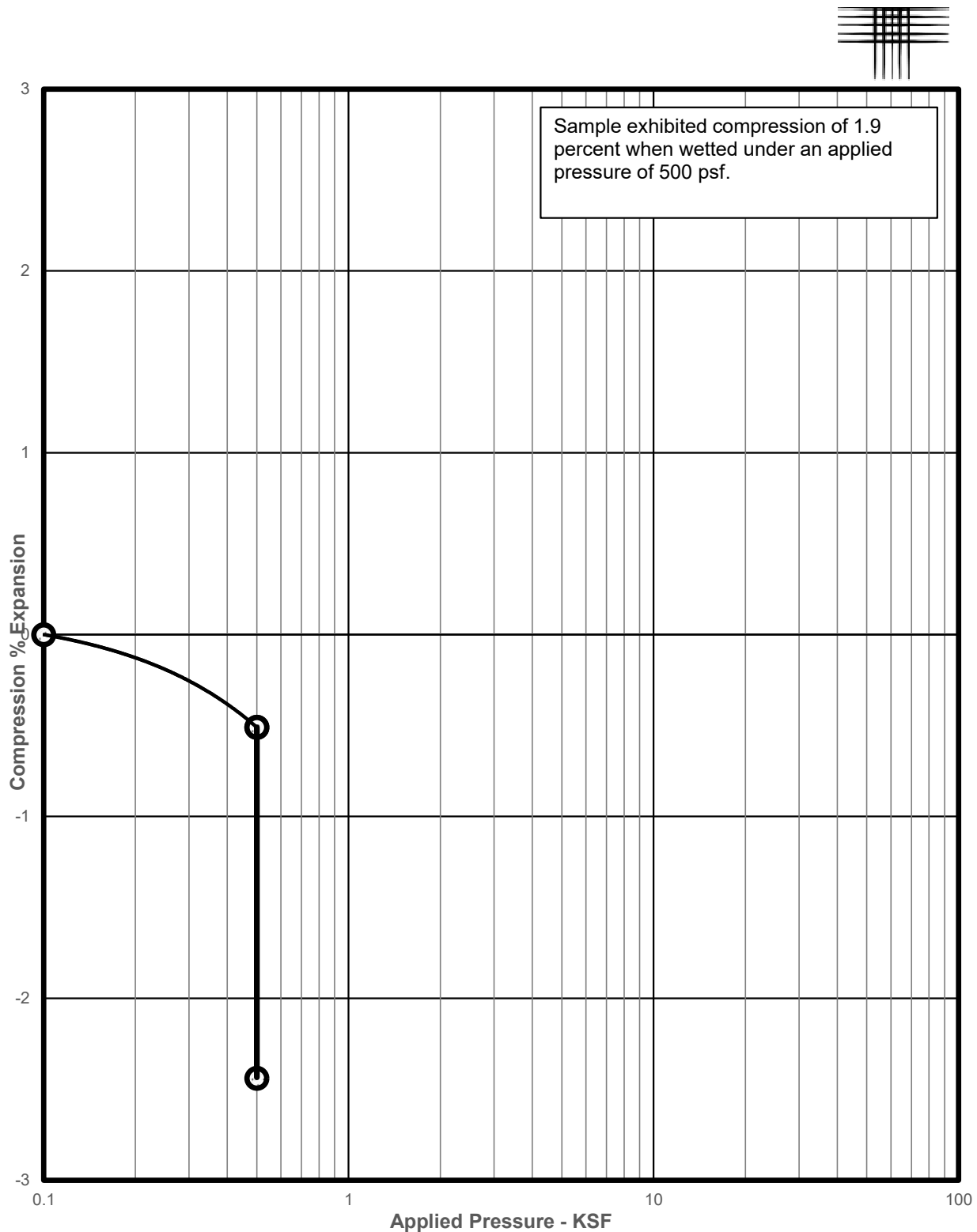


SAMPLE OF: SAND, CLAYEY (SC)
FROM: TH-24 AT 14 FEET

DRY UNIT WEIGHT: 108 pcf
MOISTURE CONTENT: 9.8 %

BMC INVESTMENTS
CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
Test Results** FIG. B- 35

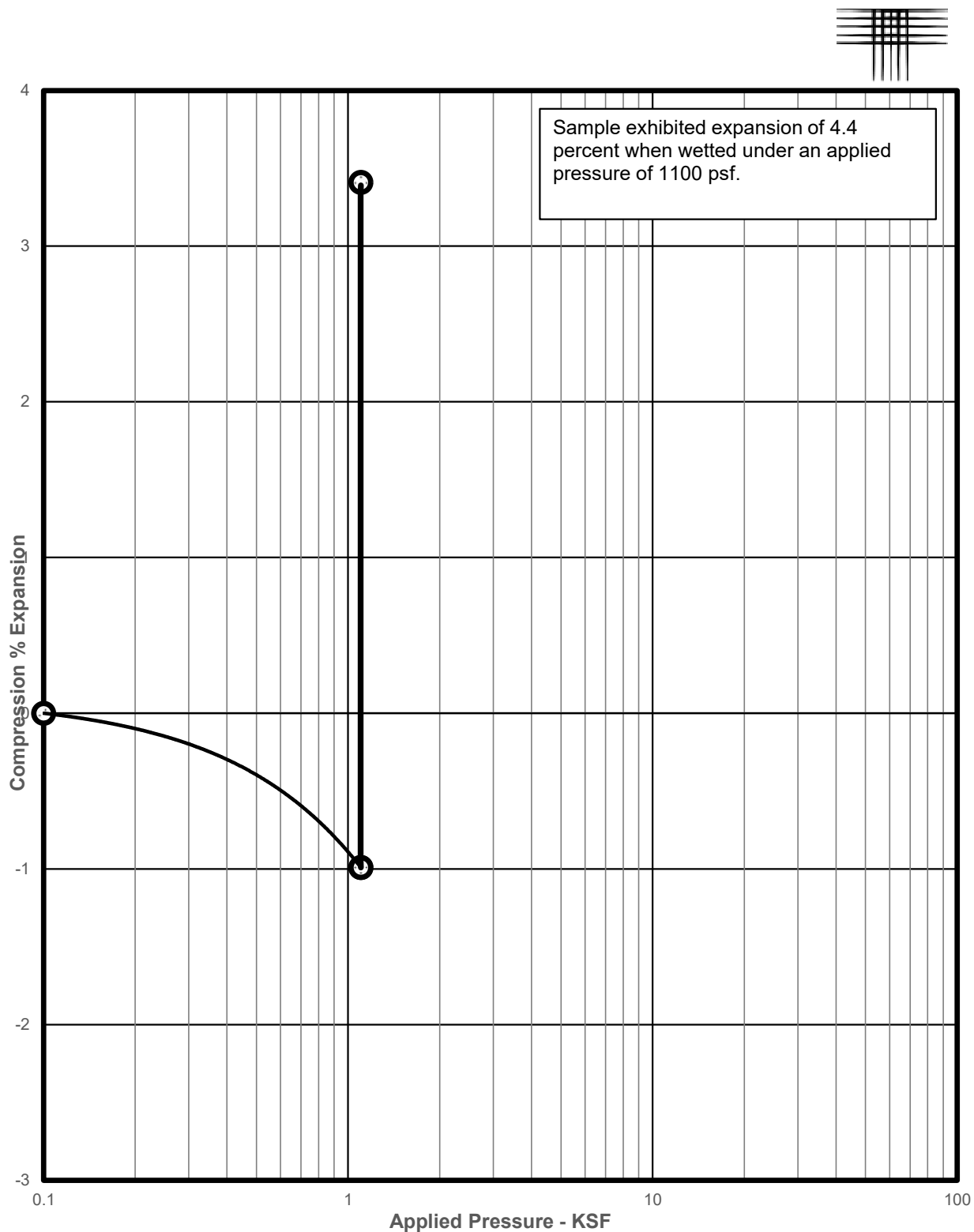


SAMPLE OF: SAND, CLAYEY (SC)
 FROM: TH-25 AT 4 FEET

DRY UNIT WEIGHT: 101 pcf
 MOISTURE CONTENT: 5.0 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 36

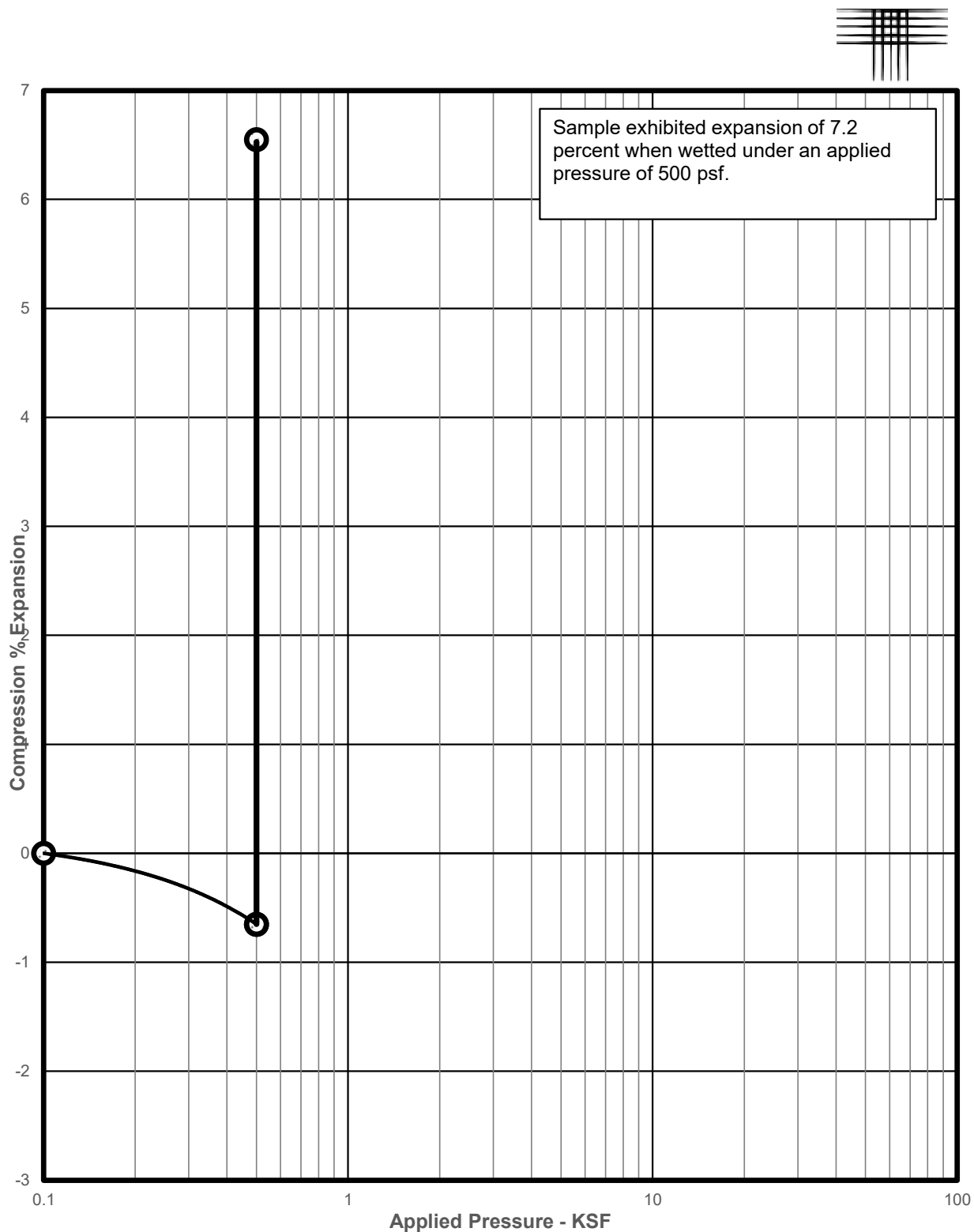


SAMPLE OF: CLAY, SANDY (CL)
FROM: TH-25 AT 9 FEET

DRY UNIT WEIGHT: 120 pcf
MOISTURE CONTENT: 9.2 %

BMC INVESTMENTS
CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
Test Results** FIG. B- 37

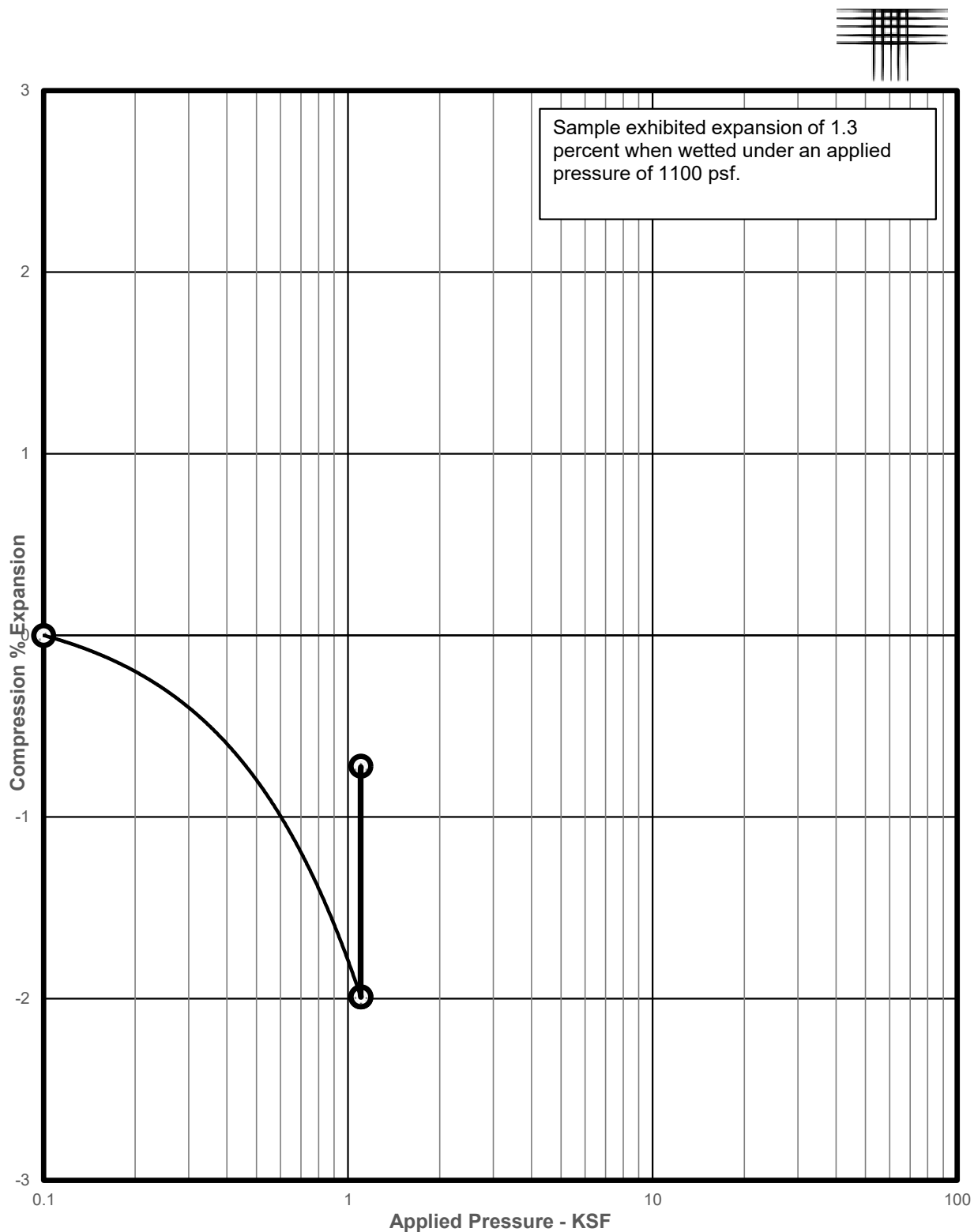


SAMPLE OF: WEATHERED CLAYSTONE
 FROM: TH-26 AT 4 FEET

DRY UNIT WEIGHT: 105 pcf
 MOISTURE CONTENT: 17.7 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 38

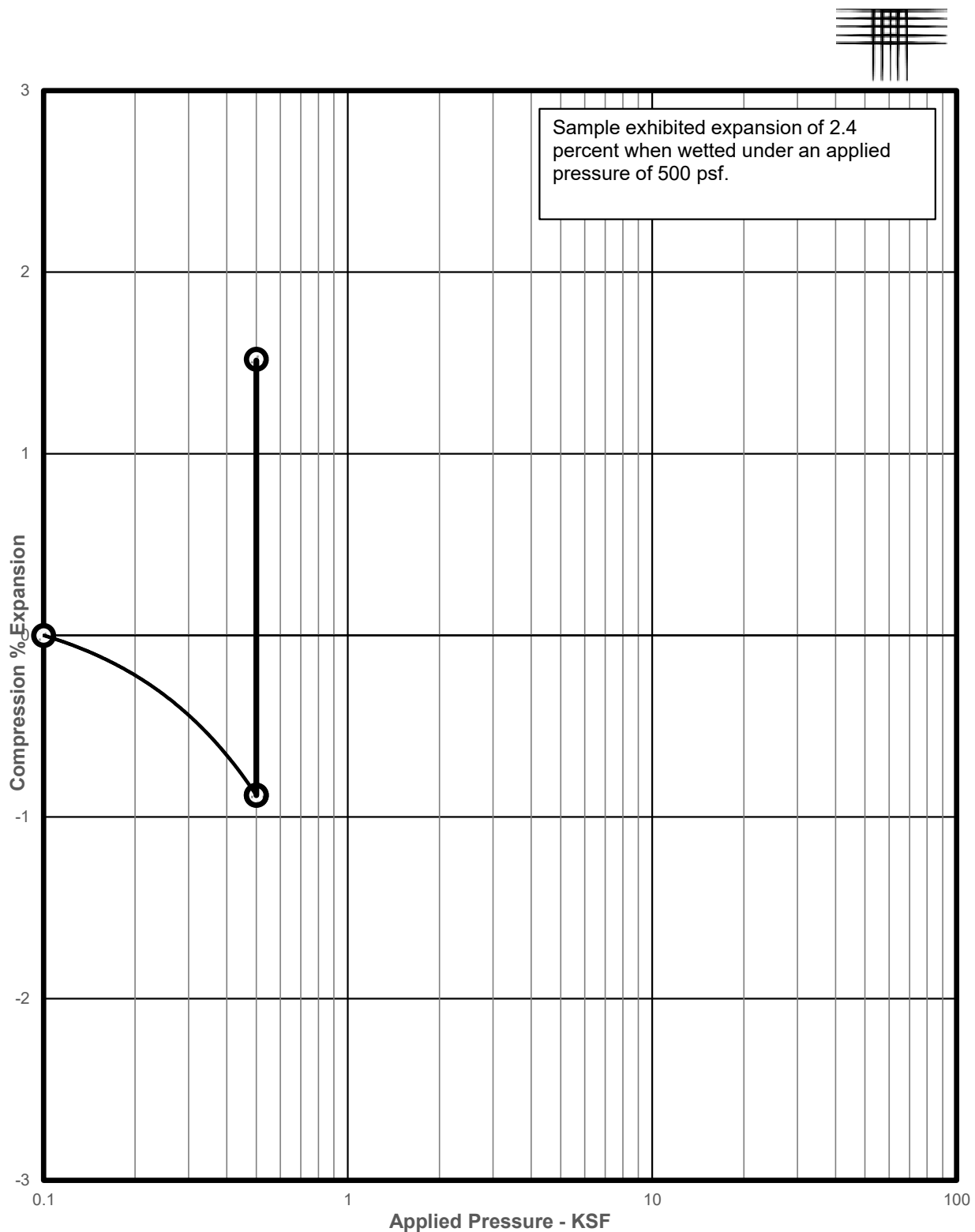


SAMPLE OF: CLAYSTONE
 FROM: TH-26 AT 9 FEET

DRY UNIT WEIGHT: 109 pcf
 MOISTURE CONTENT: 12.1 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 39

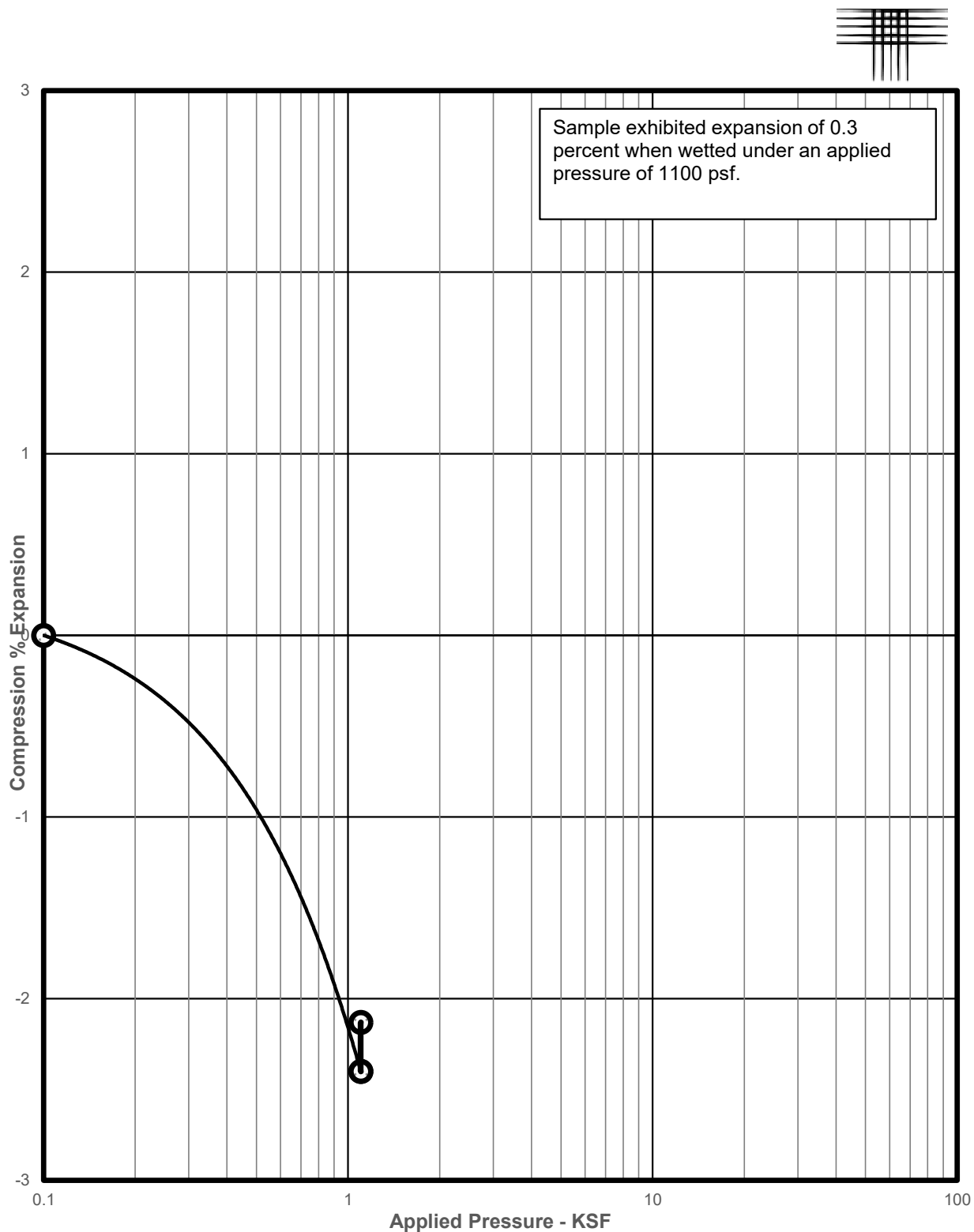


SAMPLE OF: CLAYSTONE
 FROM: TH-27 AT 14 FEET

DRY UNIT WEIGHT: 115 pcf
 MOISTURE CONTENT: 15.5 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 40

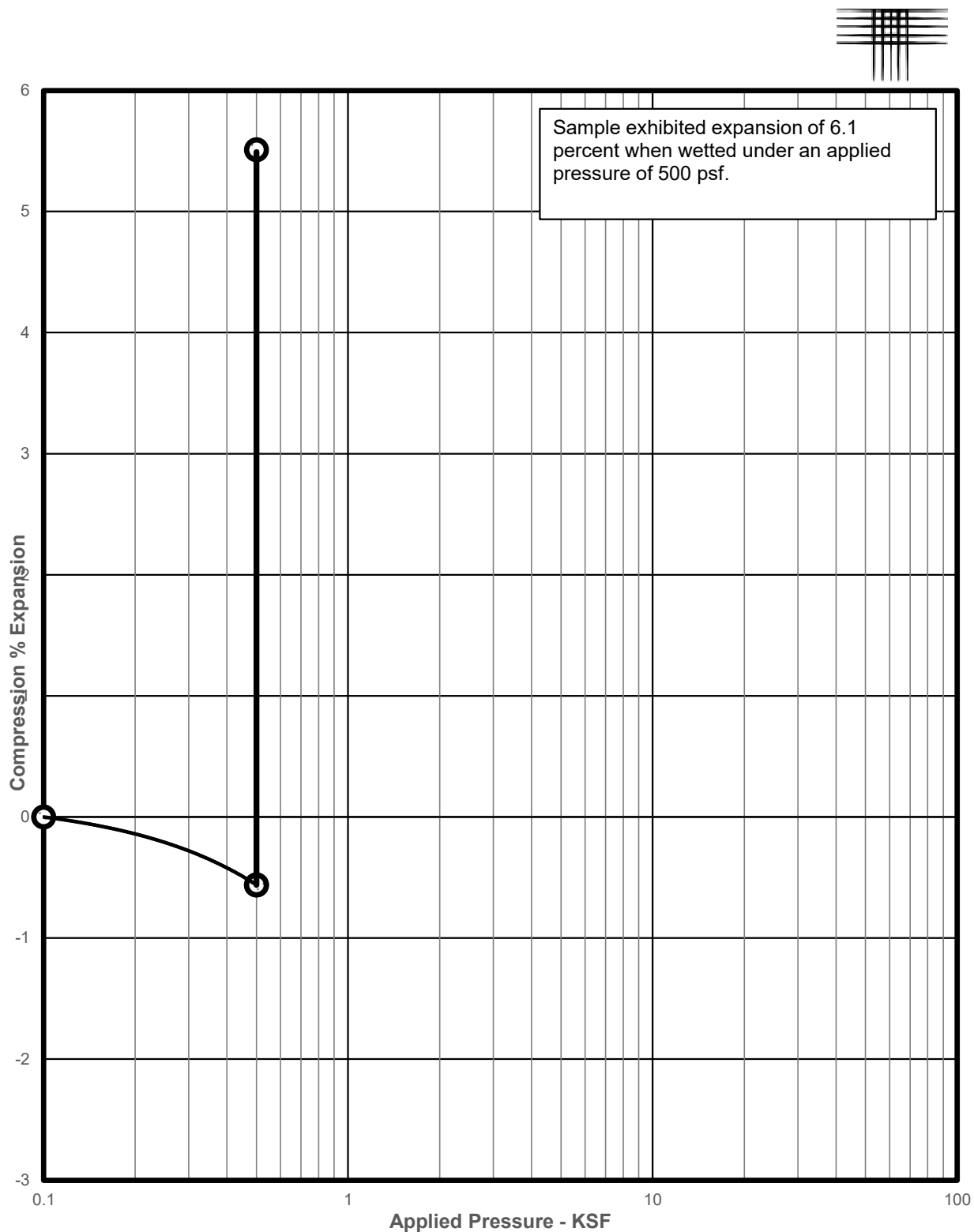


SAMPLE OF: CLAYSTONE
 FROM: TH-27 AT 19 FEET

DRY UNIT WEIGHT: 96 pcf
 MOISTURE CONTENT: 21.3 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 41

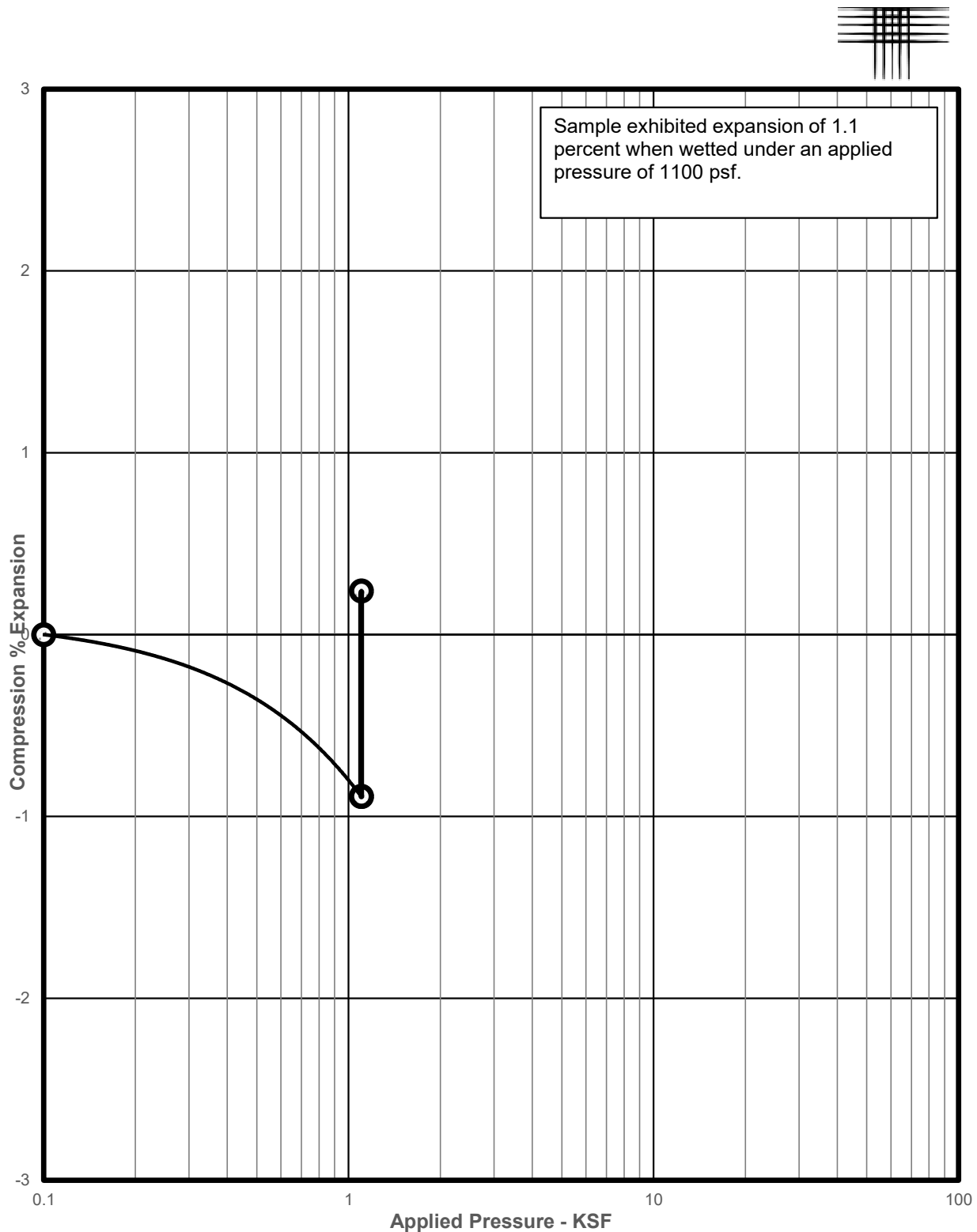


SAMPLE OF: CLAYSTONE
 FROM: TH-28 AT 14 FEET

DRY UNIT WEIGHT: 109 pcf
 MOISTURE CONTENT: 18.2 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 42

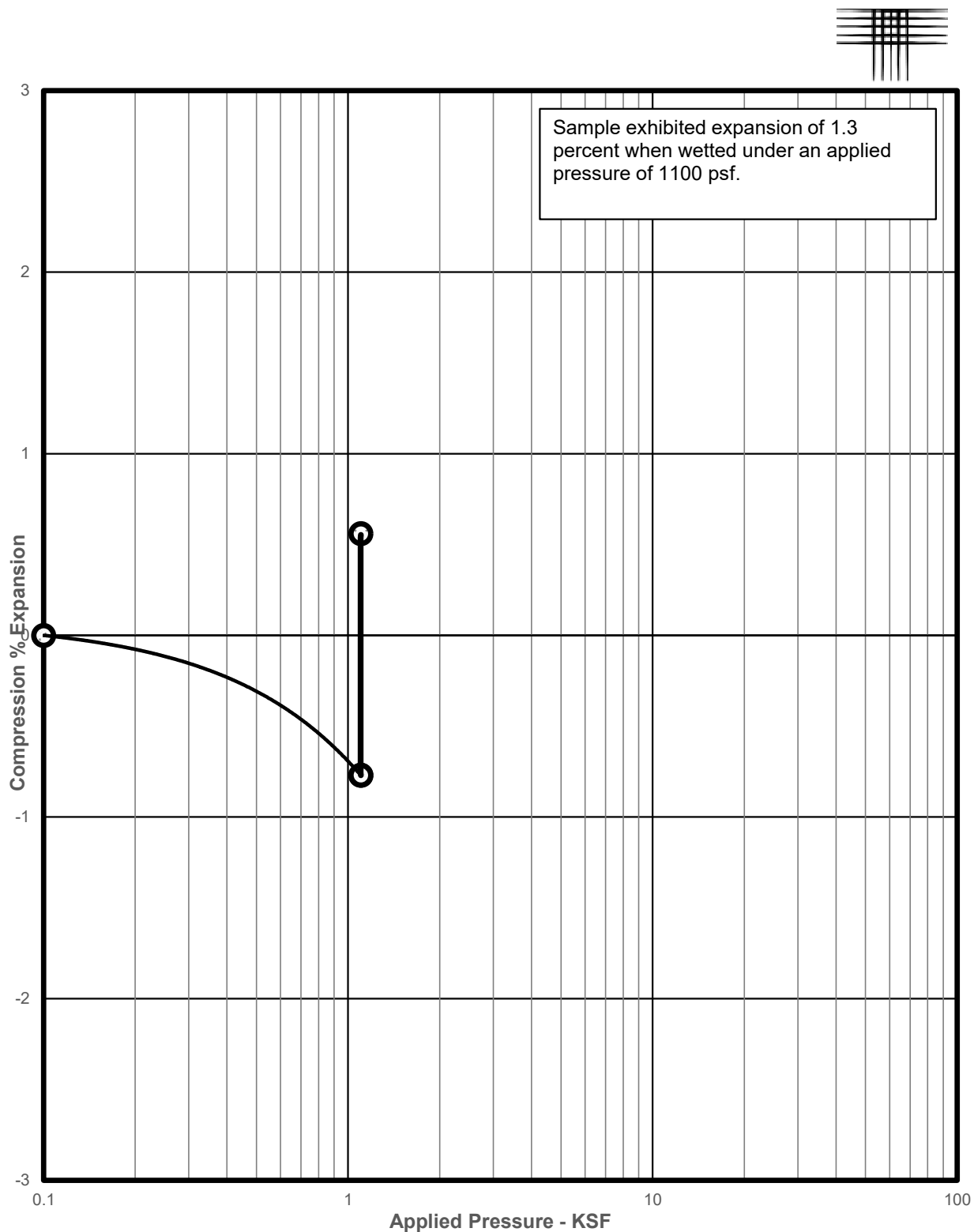


SAMPLE OF: CLAYSTONE
 FROM: TH-28 AT 24 FEET

DRY UNIT WEIGHT: 117 pcf
 MOISTURE CONTENT: 13.7 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 43

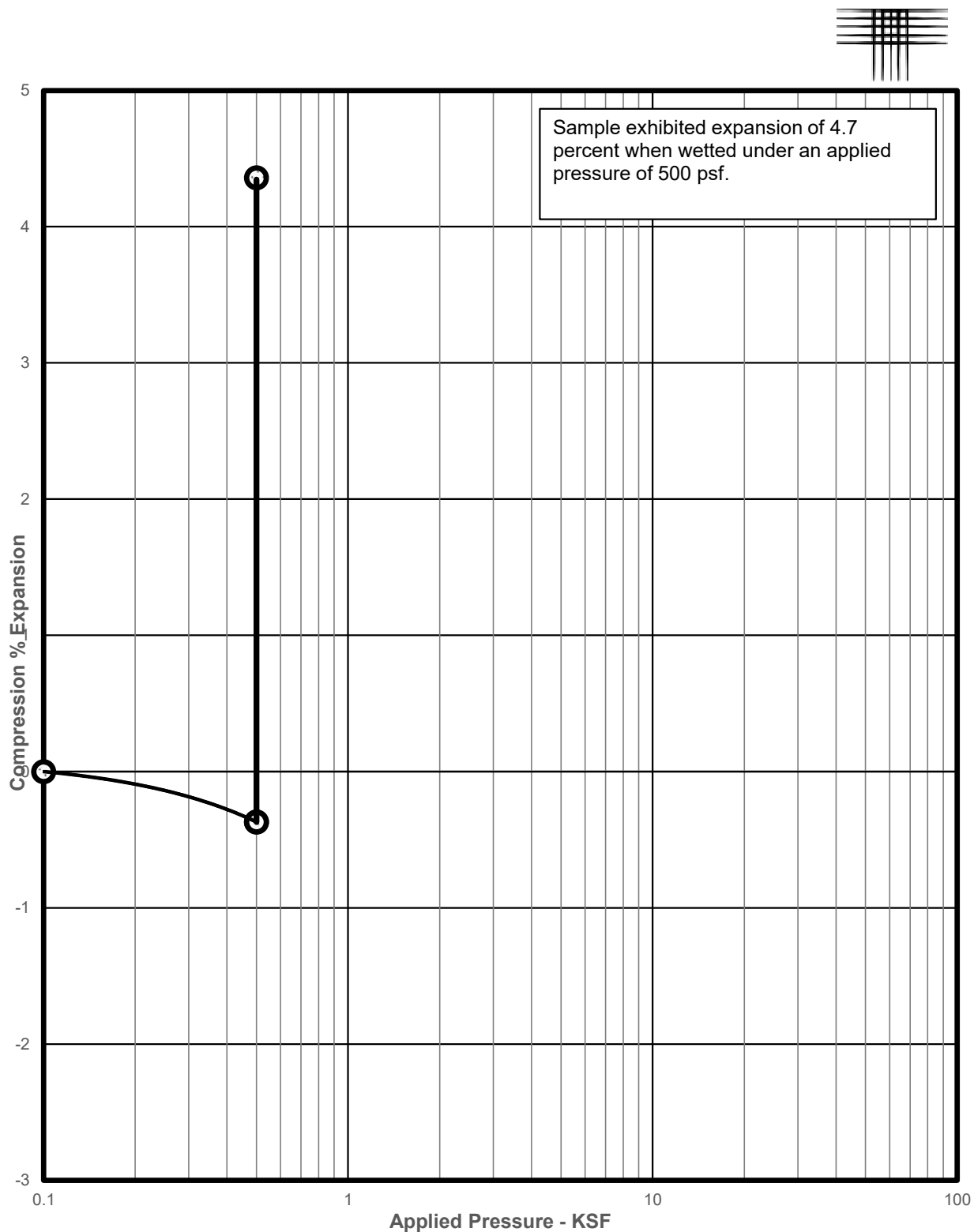


SAMPLE OF: CLAYSTONE
 FROM: TH-30 AT 19 FEET

DRY UNIT WEIGHT: 109 pcf
 MOISTURE CONTENT: 15.6 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 44

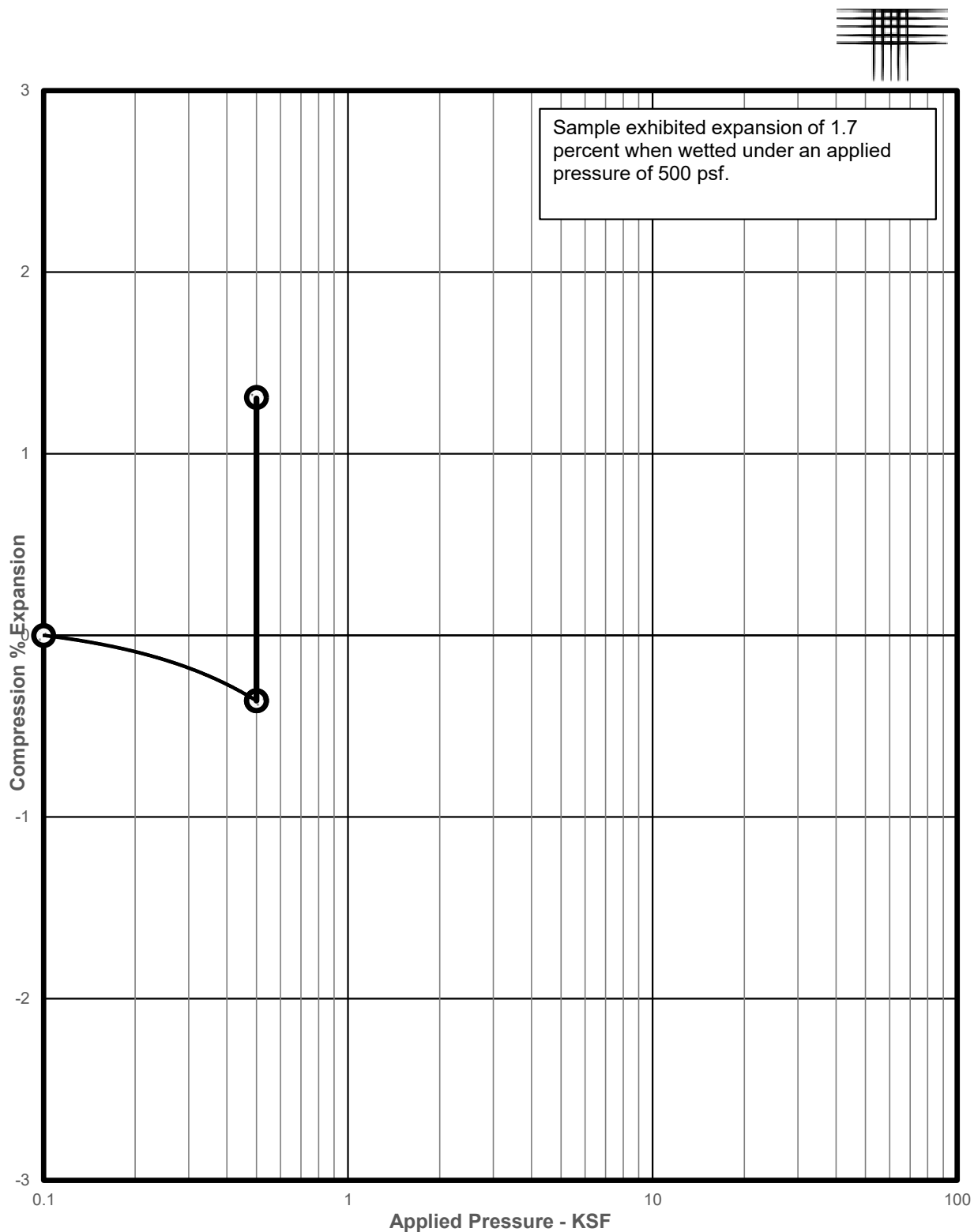


SAMPLE OF: CLAYSTONE
 FROM: TH-31 AT 19 FEET

DRY UNIT WEIGHT: 109 pcf
 MOISTURE CONTENT: 18.6 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 45

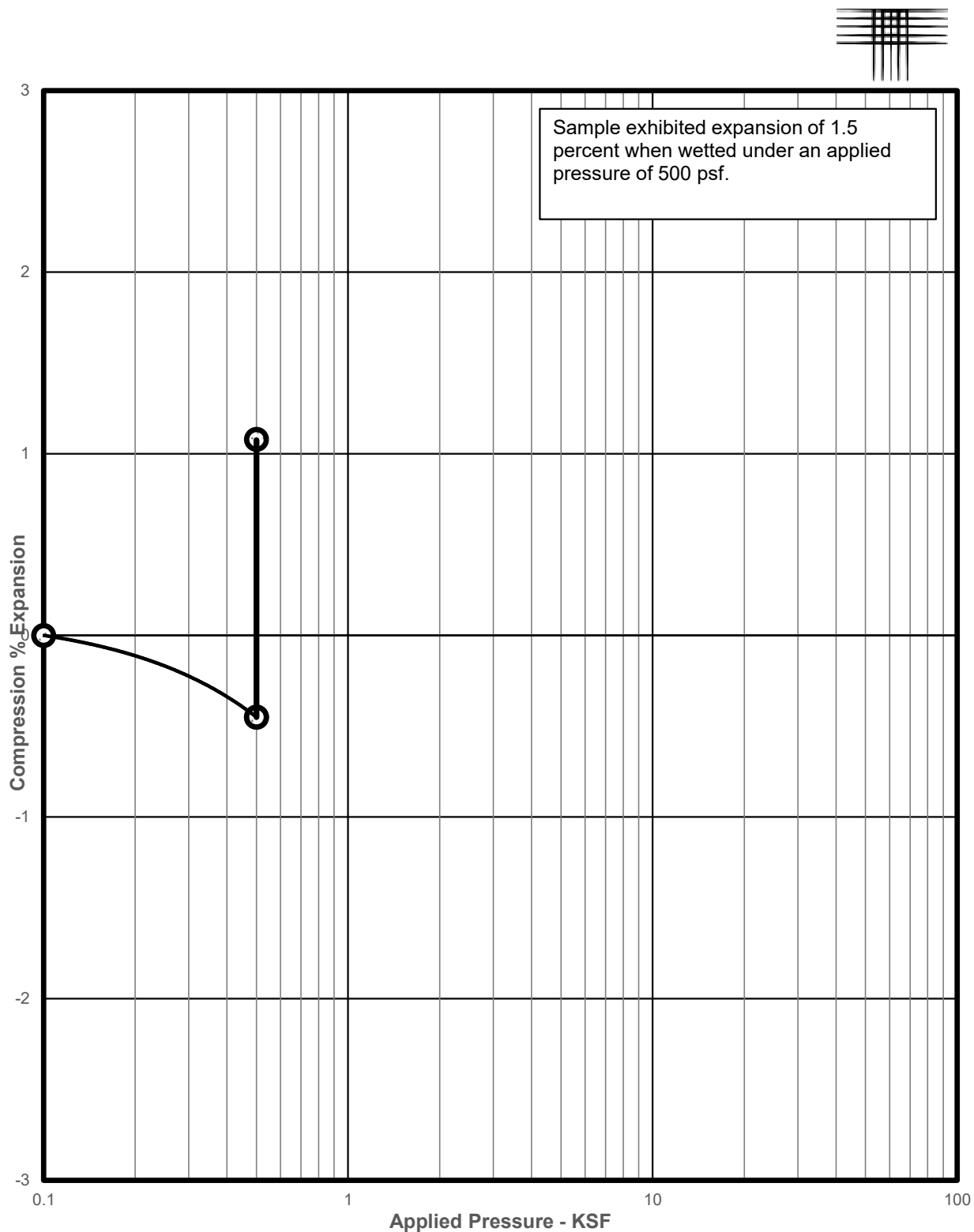


SAMPLE OF: CLAYSTONE
 FROM: TH-31 AT 24 FEET

DRY UNIT WEIGHT: 113 pcf
 MOISTURE CONTENT: 11.9 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 46

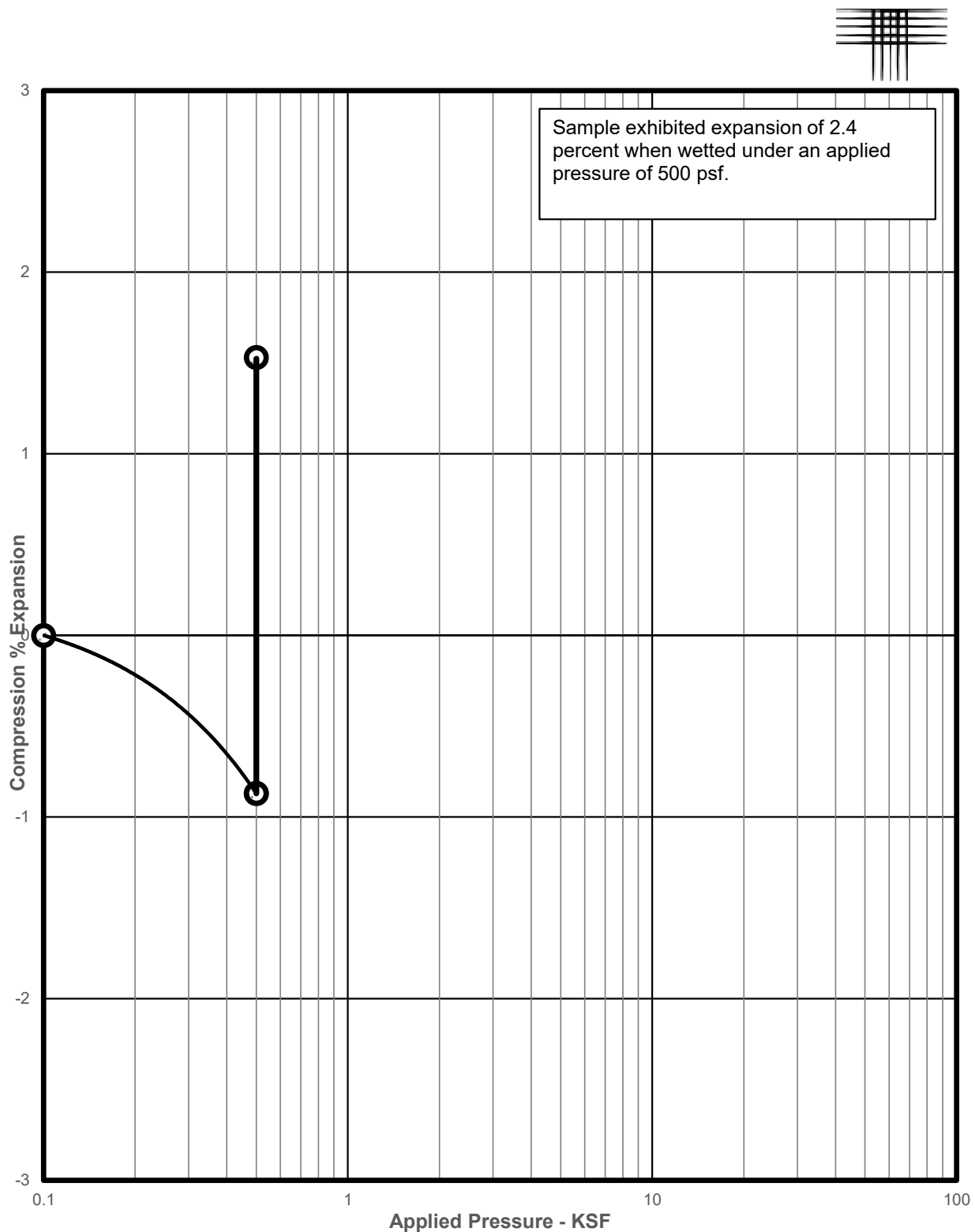


SAMPLE OF: CLAYSTONE
 FROM: TH-32 AT 29 FEET

DRY UNIT WEIGHT: 118 pcf
 MOISTURE CONTENT: 10.4 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 47

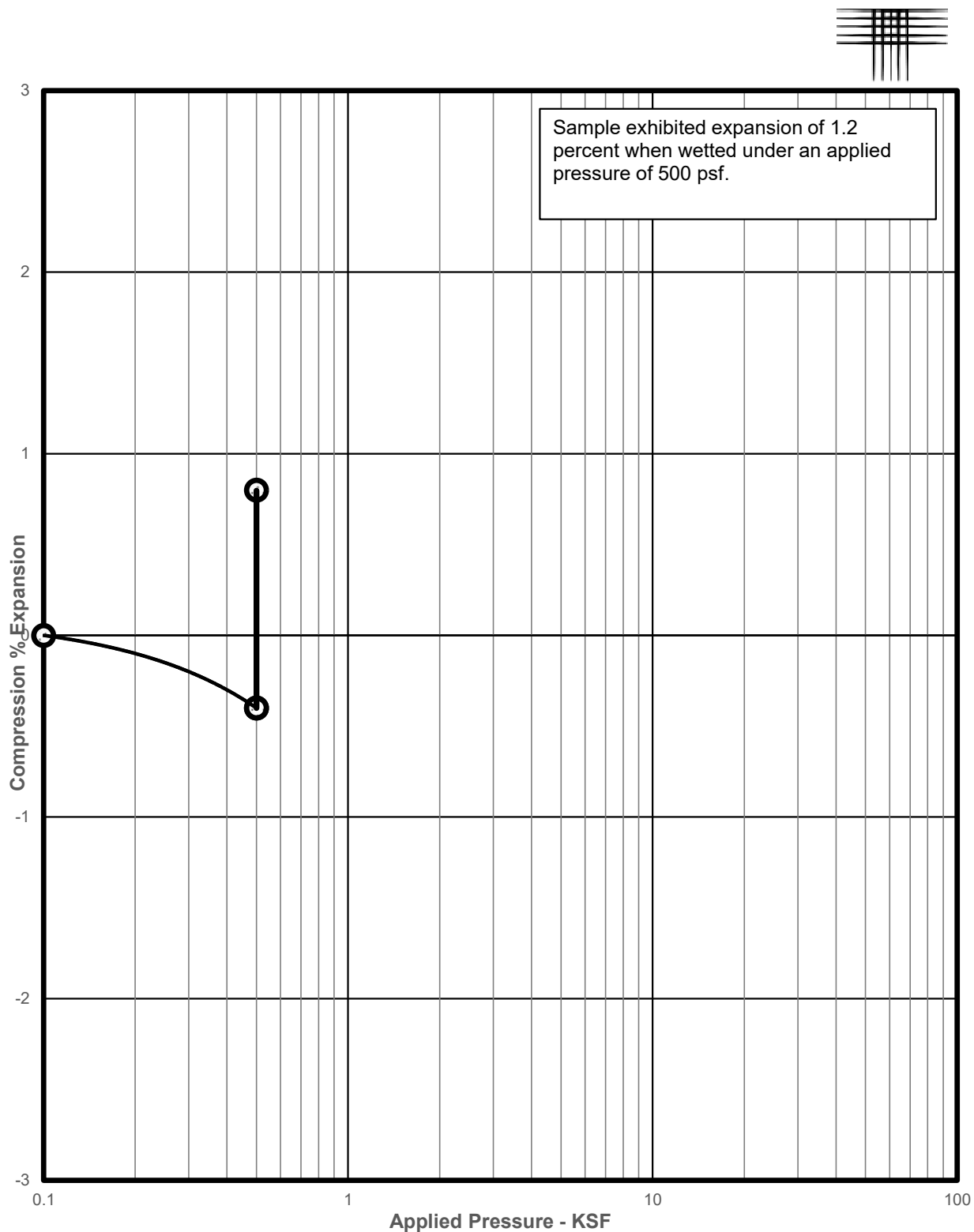


SAMPLE OF: CLAYSTONE
 FROM: TH-33 AT 19 FEET

DRY UNIT WEIGHT: 107 pcf
 MOISTURE CONTENT: 18.3 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 48

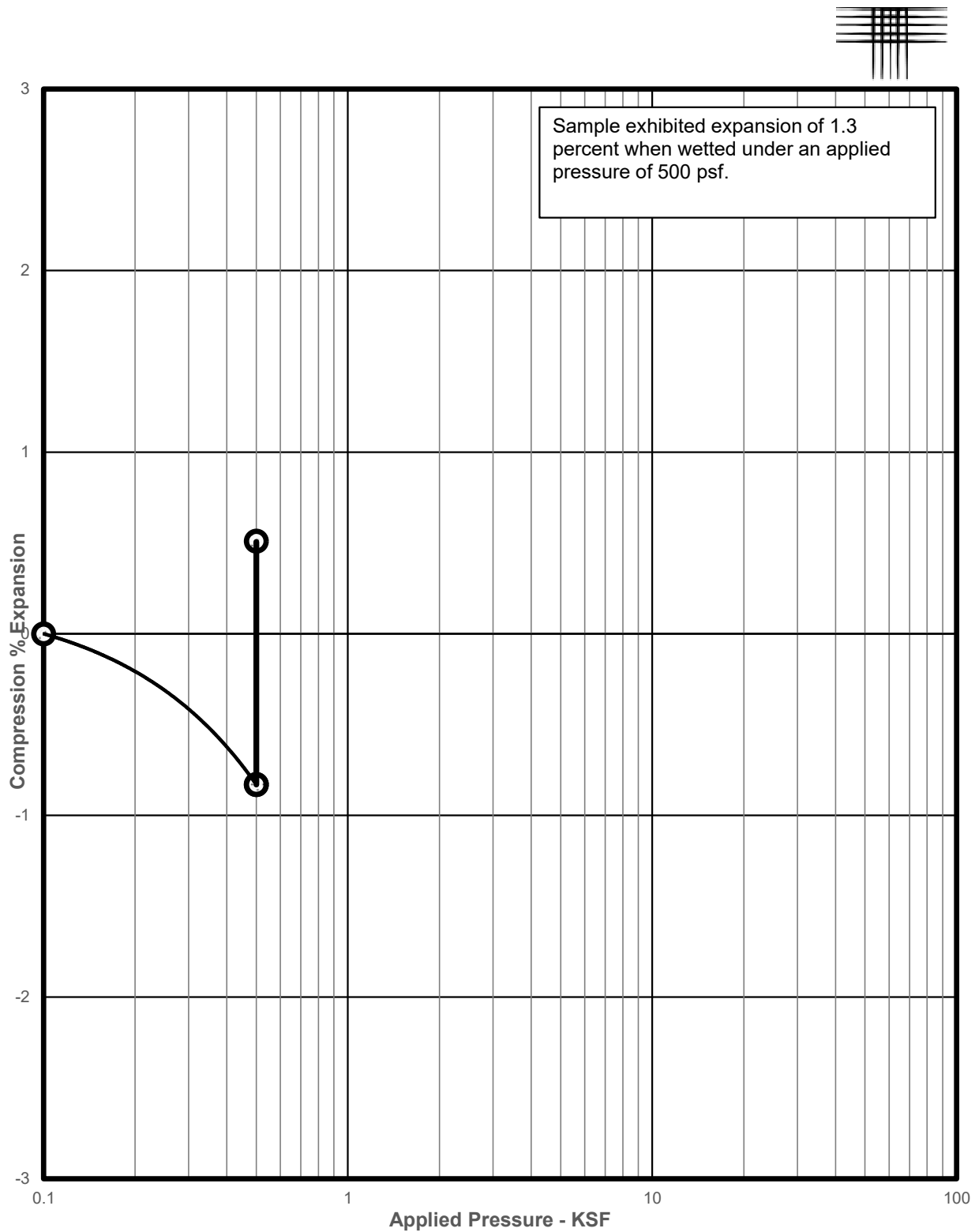


SAMPLE OF: CLAYSTONE
 FROM: TH-33 AT 24 FEET

DRY UNIT WEIGHT: 118 pcf
 MOISTURE CONTENT: 12.1 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 49

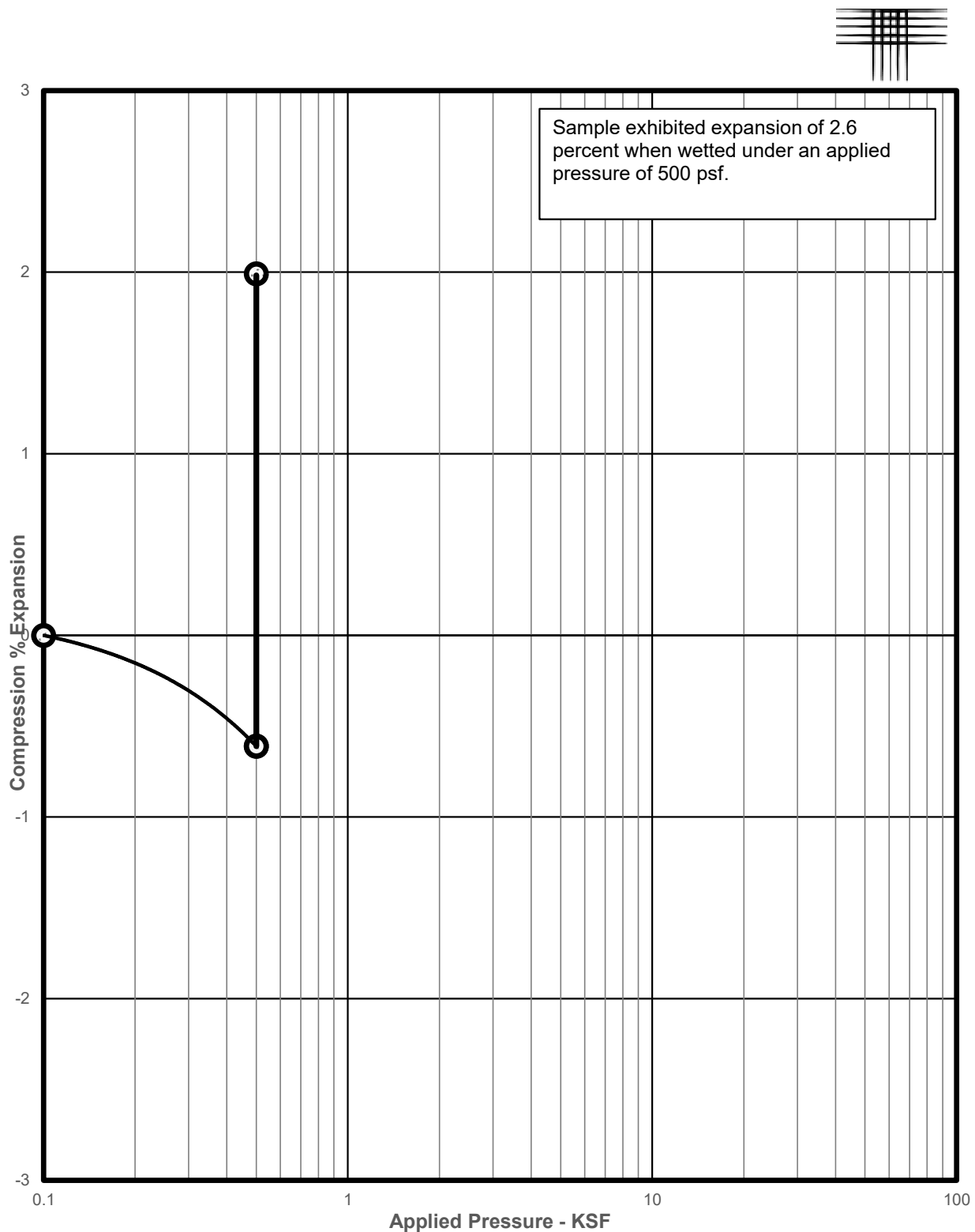


SAMPLE OF: CLAYSTONE
 FROM: TH-34 AT 19 FEET

DRY UNIT WEIGHT: 104 pcf
 MOISTURE CONTENT: 16.7 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 50

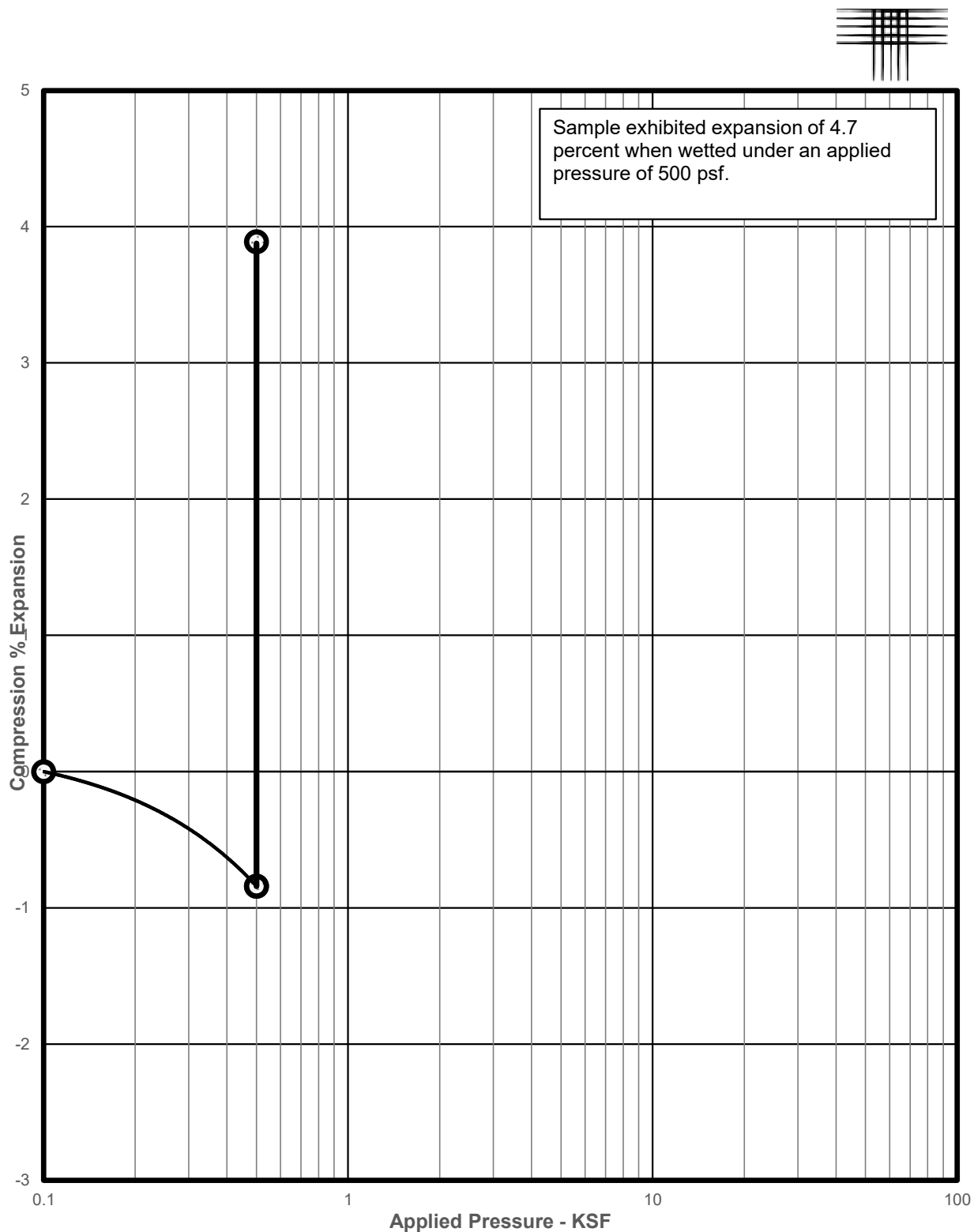


SAMPLE OF: CLAYSTONE
 FROM: TH-36 AT 14 FEET

DRY UNIT WEIGHT: 108 pcf
 MOISTURE CONTENT: 17.0 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 51

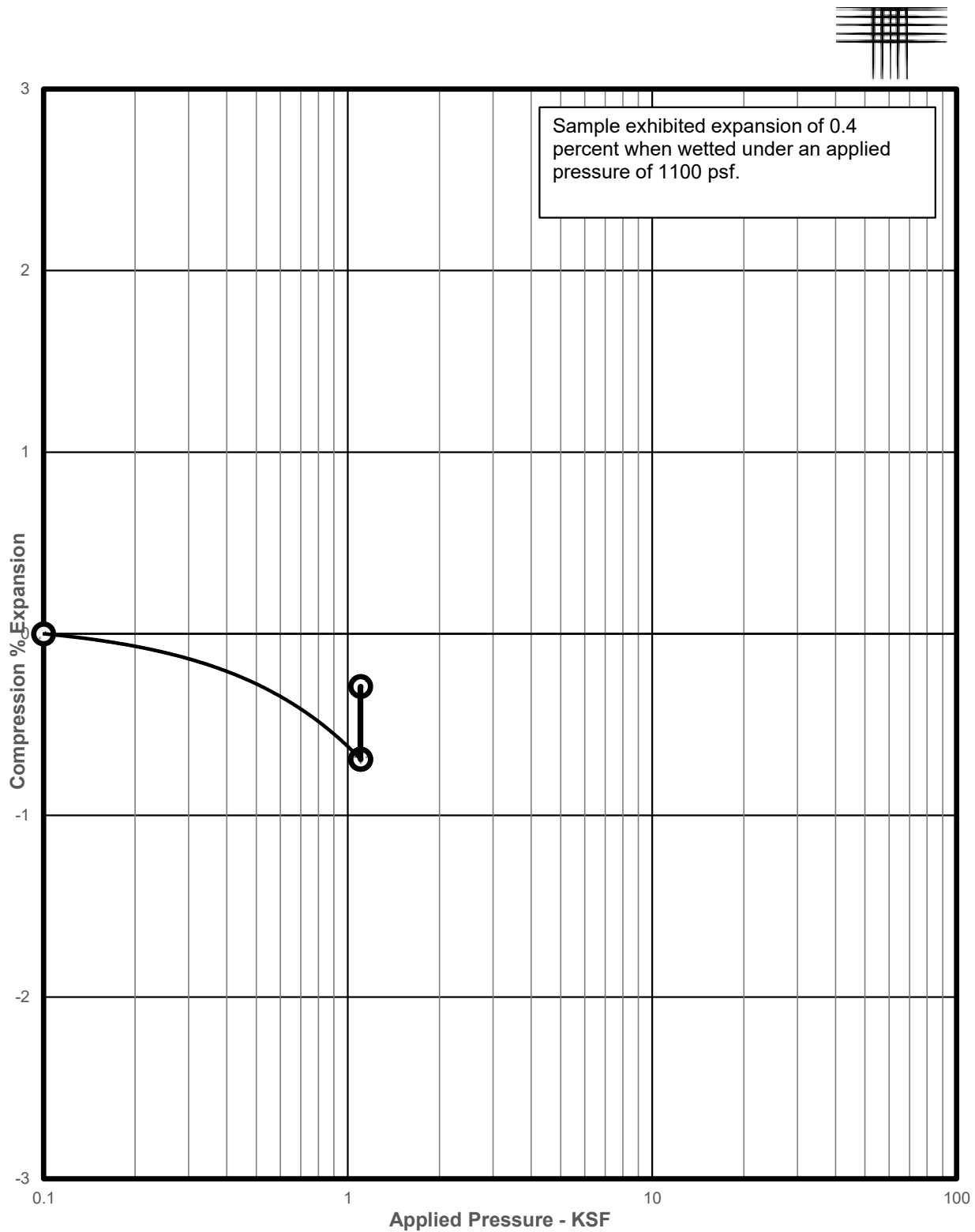


SAMPLE OF: CLAY, SANDY (CL)
 FROM: TH-39 AT 9 FEET

DRY UNIT WEIGHT: 109 pcf
 MOISTURE CONTENT: 13.7 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 52

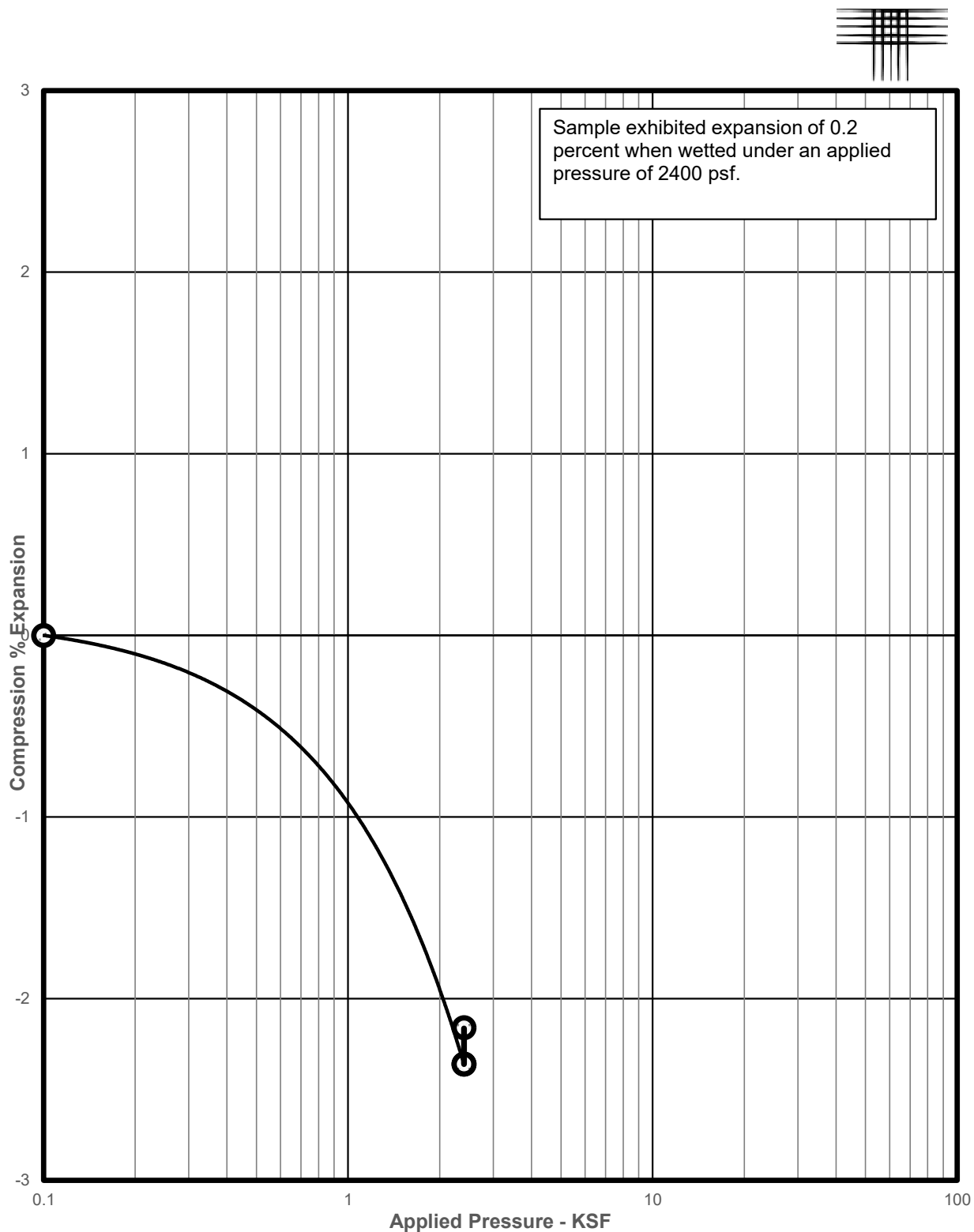


SAMPLE OF: SANDSTONE
 FROM: TH-39 AT 19 FEET

DRY UNIT WEIGHT: 114 pcf
 MOISTURE CONTENT: 11.5 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 53

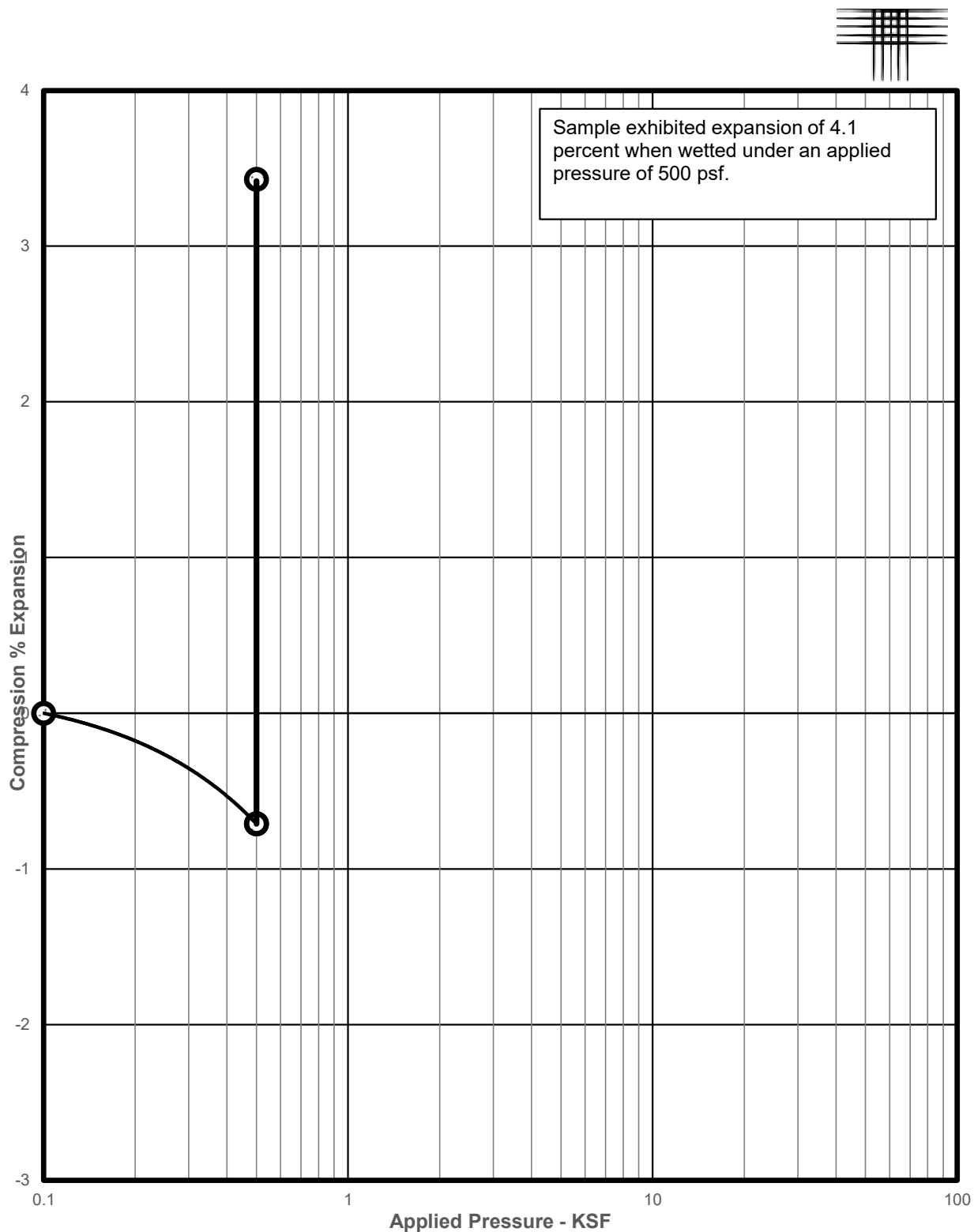


SAMPLE OF: CLAYSTONE
 FROM: TH-41 AT 29 FEET

DRY UNIT WEIGHT: 107 pcf
 MOISTURE CONTENT: 16.2 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 54

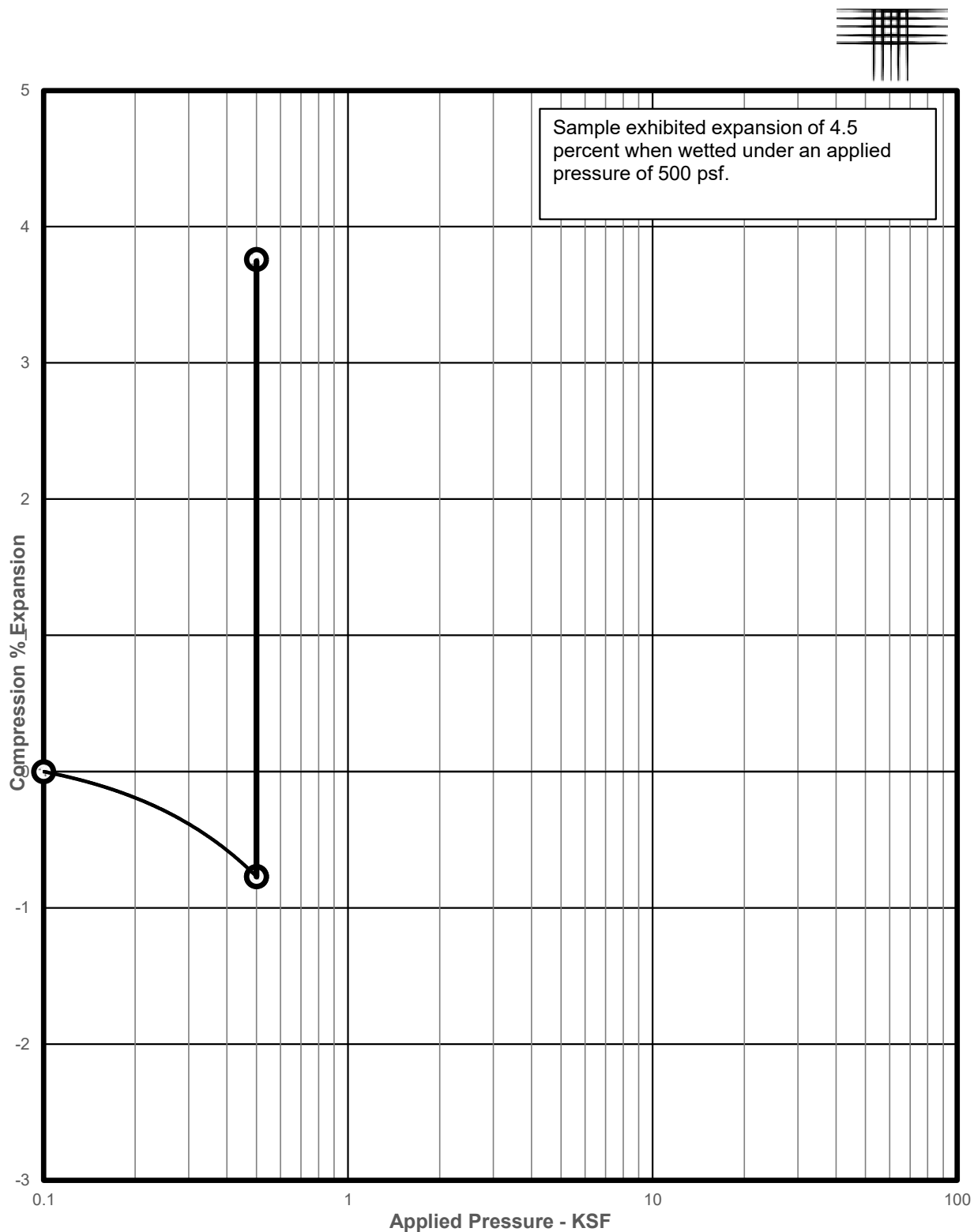


SAMPLE OF: CLAYSTONE
FROM: TH-42 AT 14 FEET

DRY UNIT WEIGHT: 111 pcf
MOISTURE CONTENT: 15.0 %

BMC INVESTMENTS
CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
Test Results** FIG. B- 55

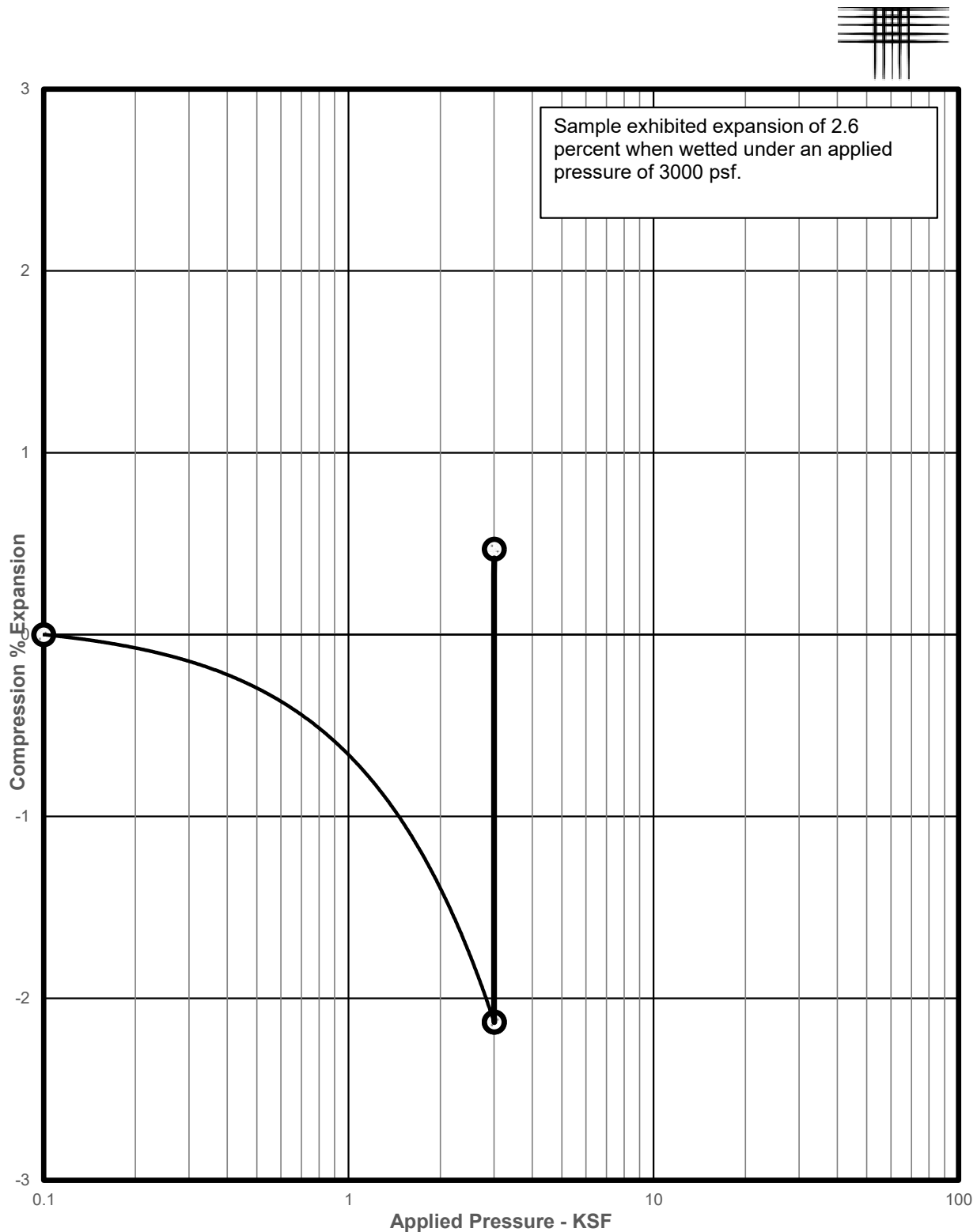


SAMPLE OF: CLAYSTONE
 FROM: TH-43 AT 14 FEET

DRY UNIT WEIGHT: 109 pcf
 MOISTURE CONTENT: 18.2 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 56

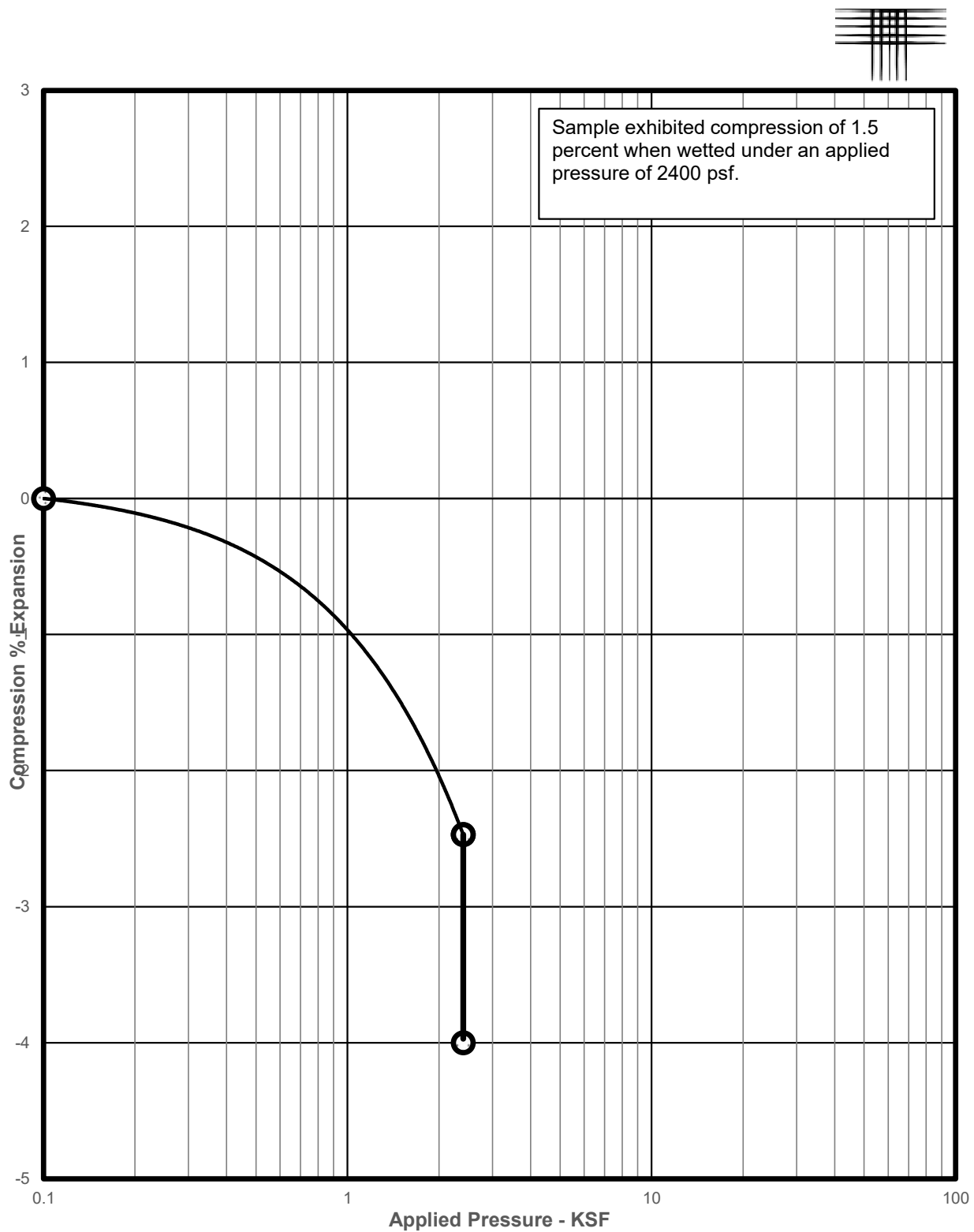


SAMPLE OF: CLAYSTONE
 FROM: TH-43 AT 34 FEET

DRY UNIT WEIGHT: 101 pcf
 MOISTURE CONTENT: 24.9 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 57

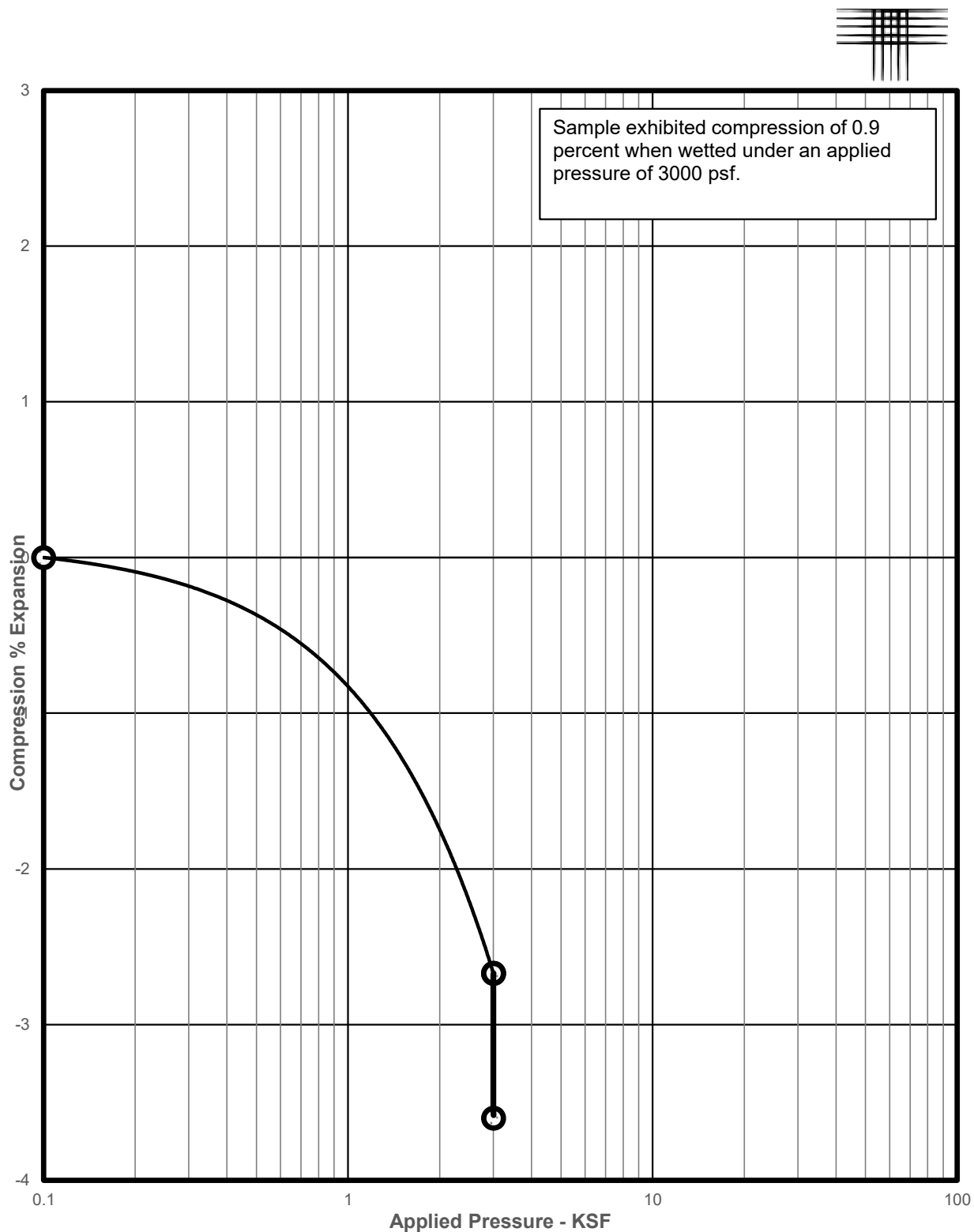


SAMPLE OF: SAND, CLAYEY (SC)
FROM: TH-45 AT 4 FEET

DRY UNIT WEIGHT: 110 pcf
MOISTURE CONTENT: 8.8 %

BMC INVESTMENTS
CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
Test Results** FIG. B- 58

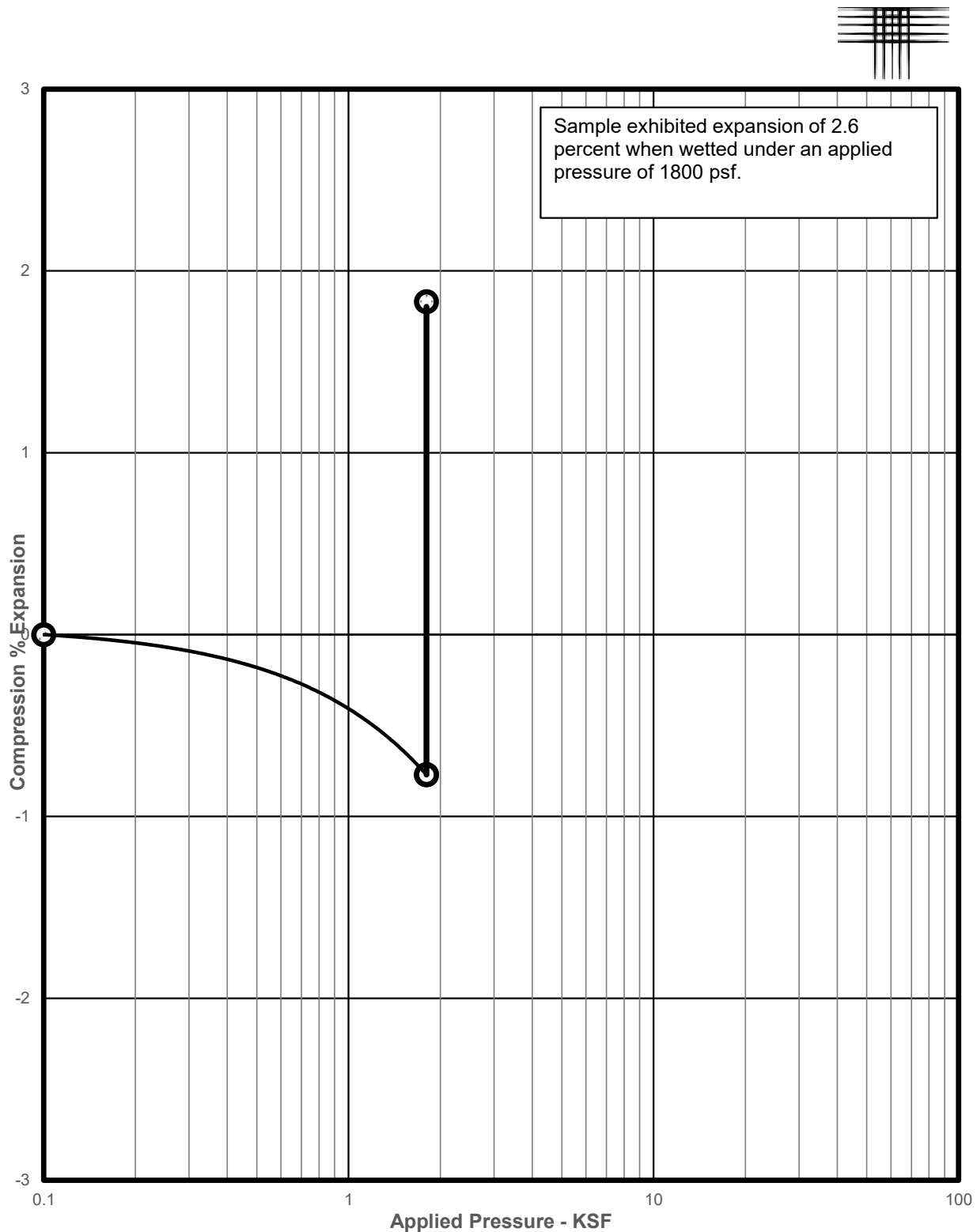


SAMPLE OF: CLAYSTONE
 FROM: TH-45 AT 9 FEET

DRY UNIT WEIGHT: 98 pcf
 MOISTURE CONTENT: 14.9 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 59

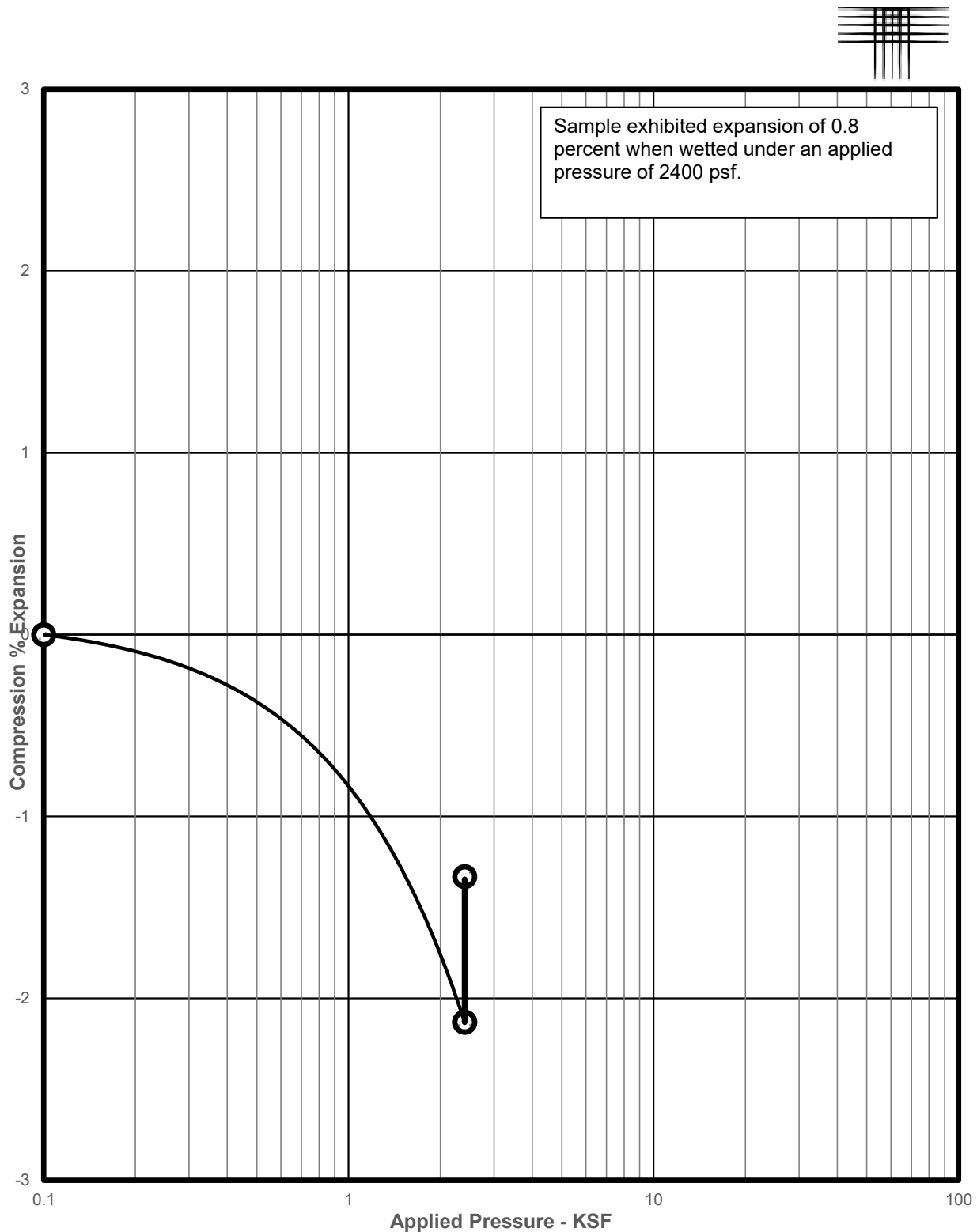


SAMPLE OF: CLAYSTONE
 FROM: TH-46 AT 4 FEET

DRY UNIT WEIGHT: 120 pcf
 MOISTURE CONTENT: 9.6 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 60

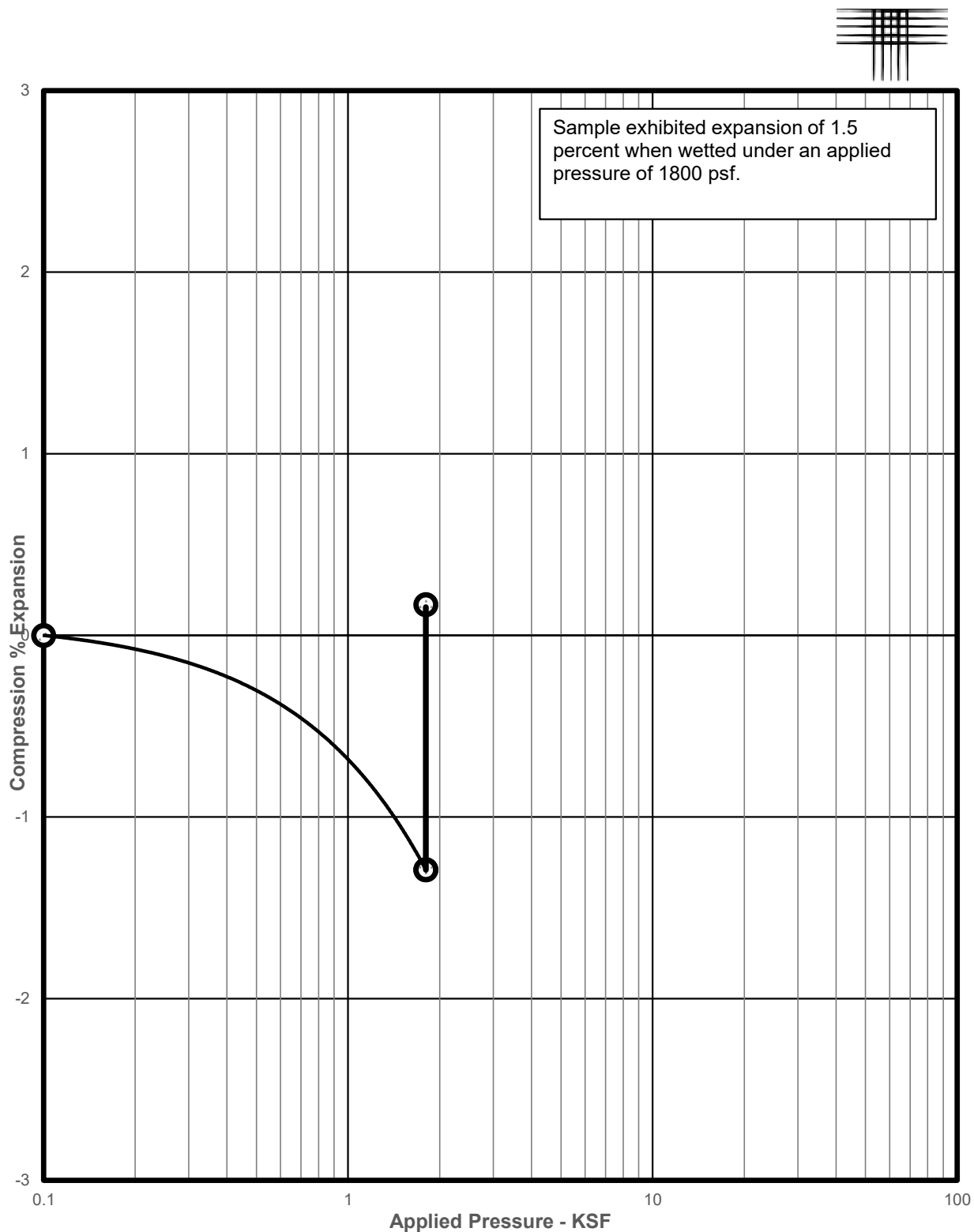


SAMPLE OF: CLAYSTONE
 FROM: TH-46 AT 9 FEET

DRY UNIT WEIGHT: 104 pcf
 MOISTURE CONTENT: 17.1 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 61

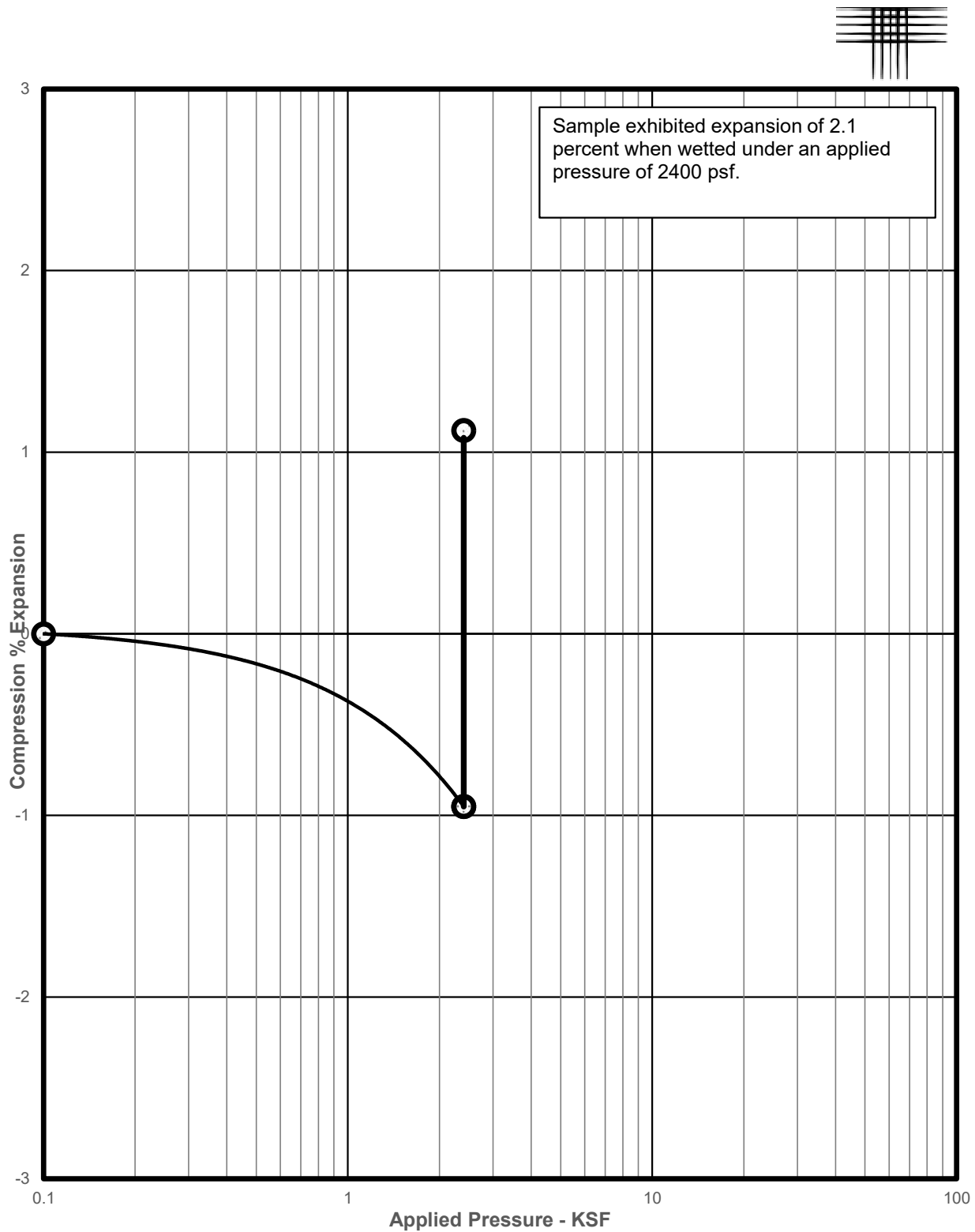


SAMPLE OF: CLAYSTONE
 FROM: TH-47 AT 14 FEET

DRY UNIT WEIGHT: 118 pcf
 MOISTURE CONTENT: 12.0 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 62

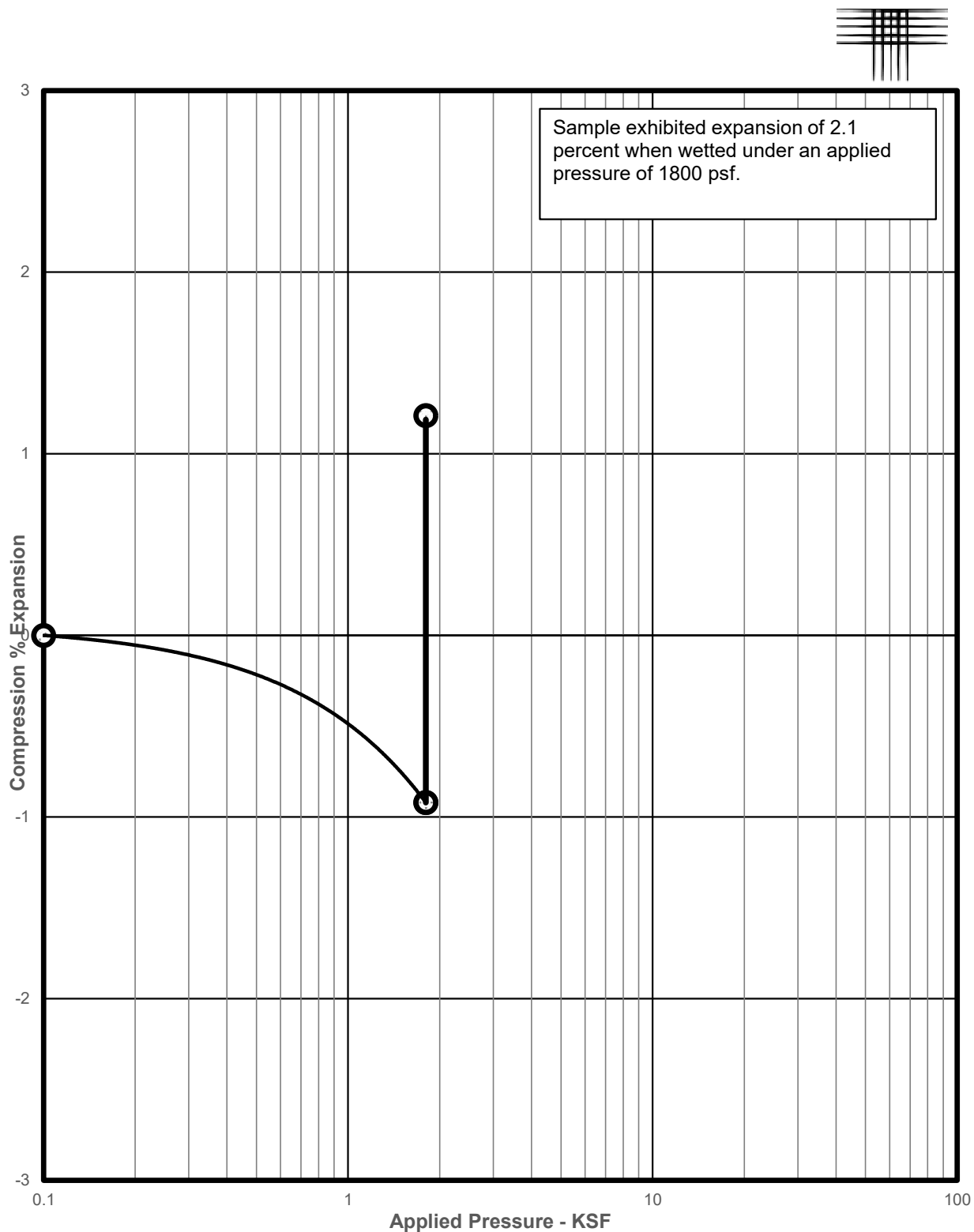


SAMPLE OF: CLAYSTONE
 FROM: TH-47 AT 19 FEET

DRY UNIT WEIGHT: 107 pcf
 MOISTURE CONTENT: 20.6 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 63

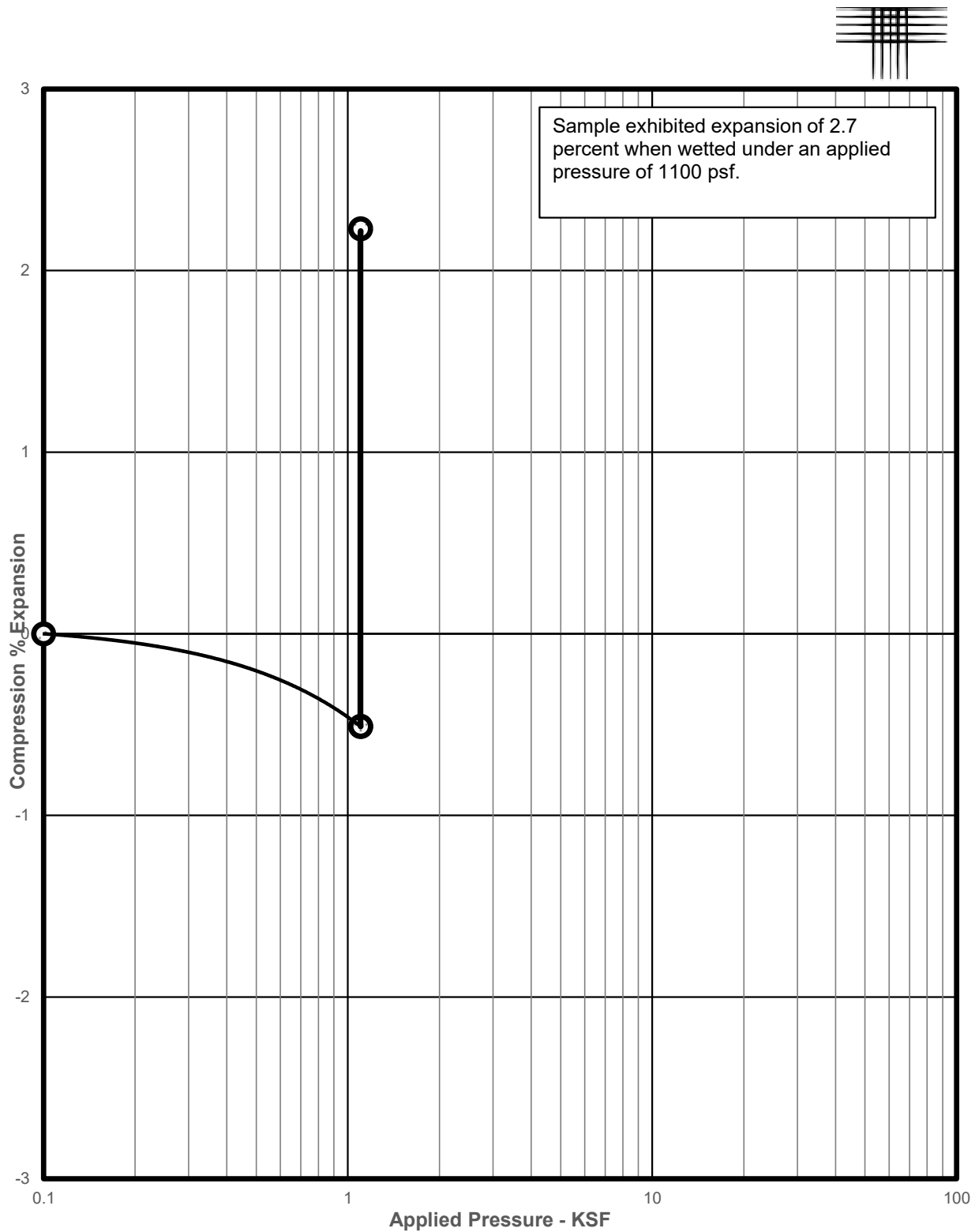


SAMPLE OF: CLAYSTONE
 FROM: TH-48 AT 14 FEET

DRY UNIT WEIGHT: 98 pcf
 MOISTURE CONTENT: 21.9 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 64

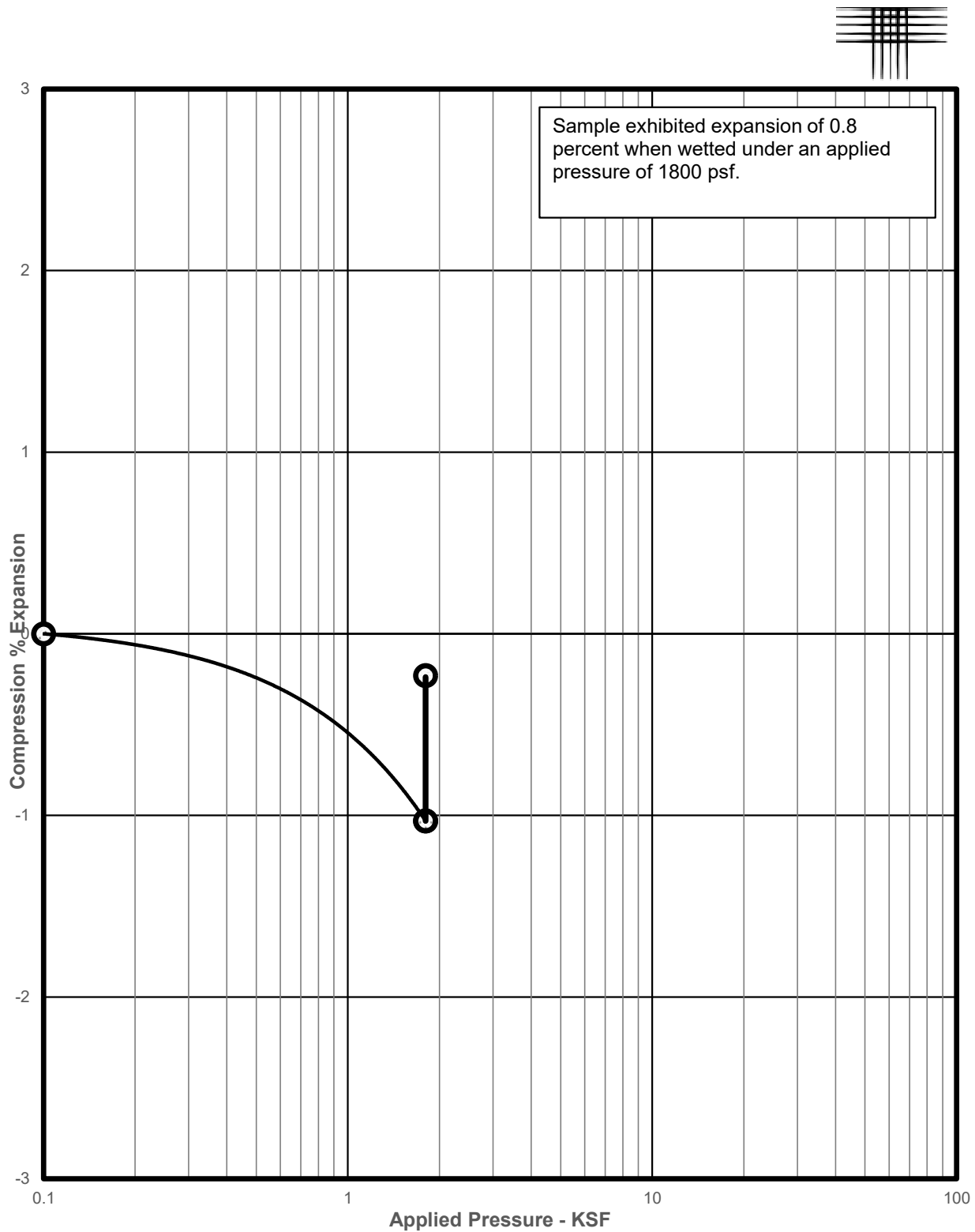


SAMPLE OF: CLAYSTONE
FROM: TH-50 AT 9 FEET

DRY UNIT WEIGHT: 114 pcf
MOISTURE CONTENT: 10.3 %

BMC INVESTMENTS
CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
Test Results** FIG. B- 65

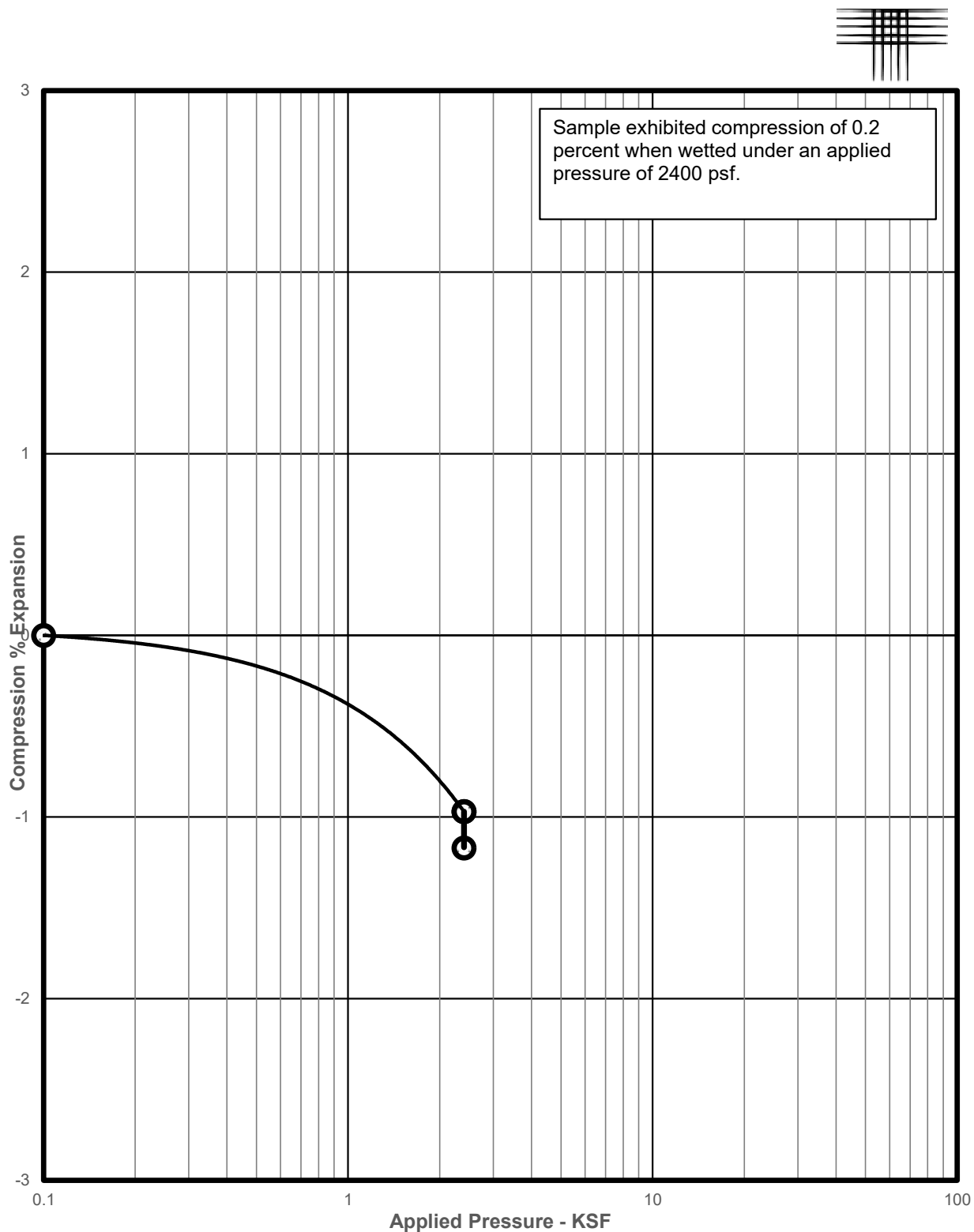


SAMPLE OF: CLAYSTONE
 FROM: TH-50 AT 14 FEET

DRY UNIT WEIGHT: 109 pcf
 MOISTURE CONTENT: 17.0 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 66

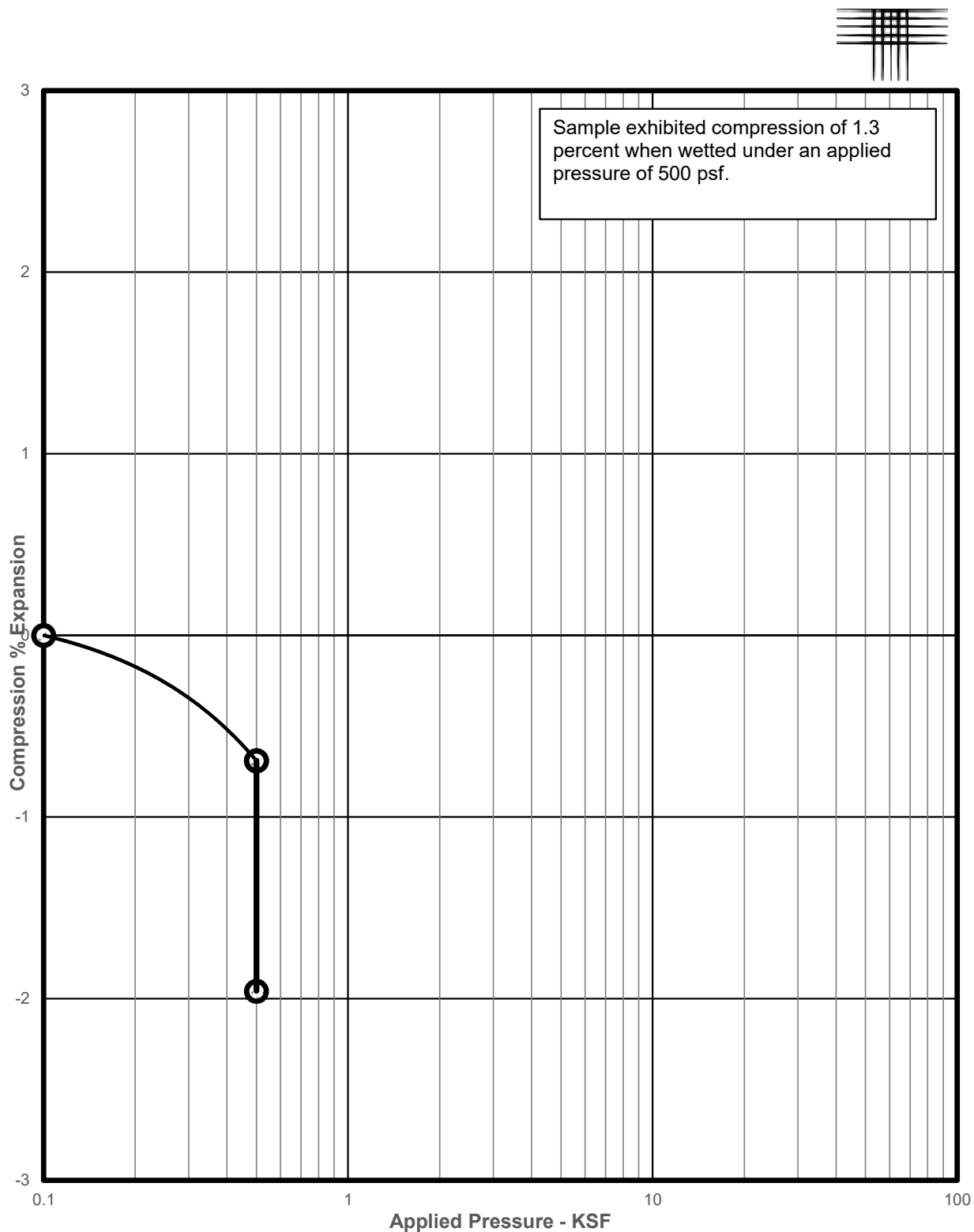


SAMPLE OF: CLAYSTONE
FROM: TH-50 AT 19 FEET

DRY UNIT WEIGHT: 111 pcf
MOISTURE CONTENT: 15.3 %

BMC INVESTMENTS
CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
Test Results** FIG. B- 67

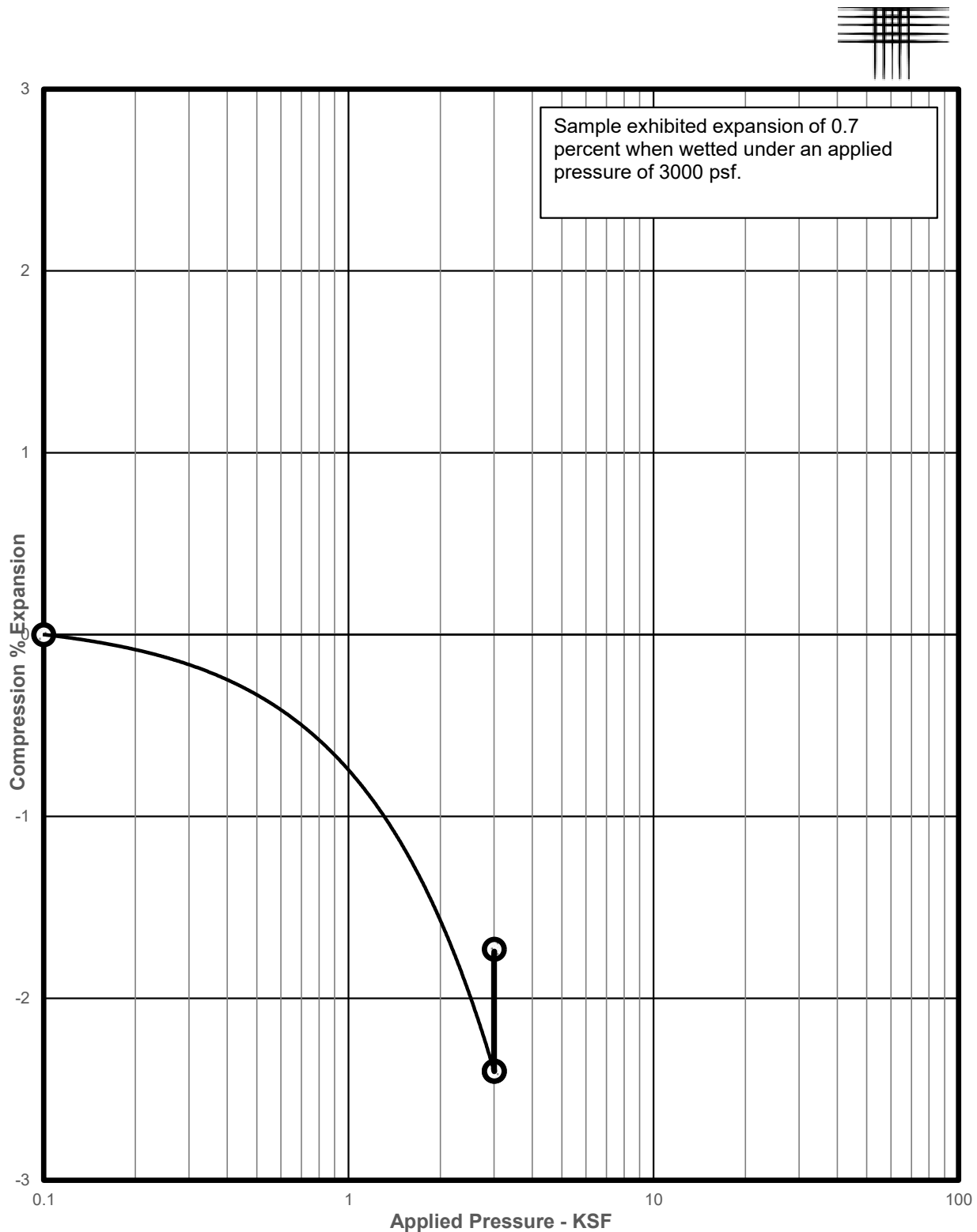


SAMPLE OF: SAND, CLAYEY (SC)
 FROM: TH-51 AT 4 FEET

DRY UNIT WEIGHT: 89 pcf
 MOISTURE CONTENT: 7.8 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 68

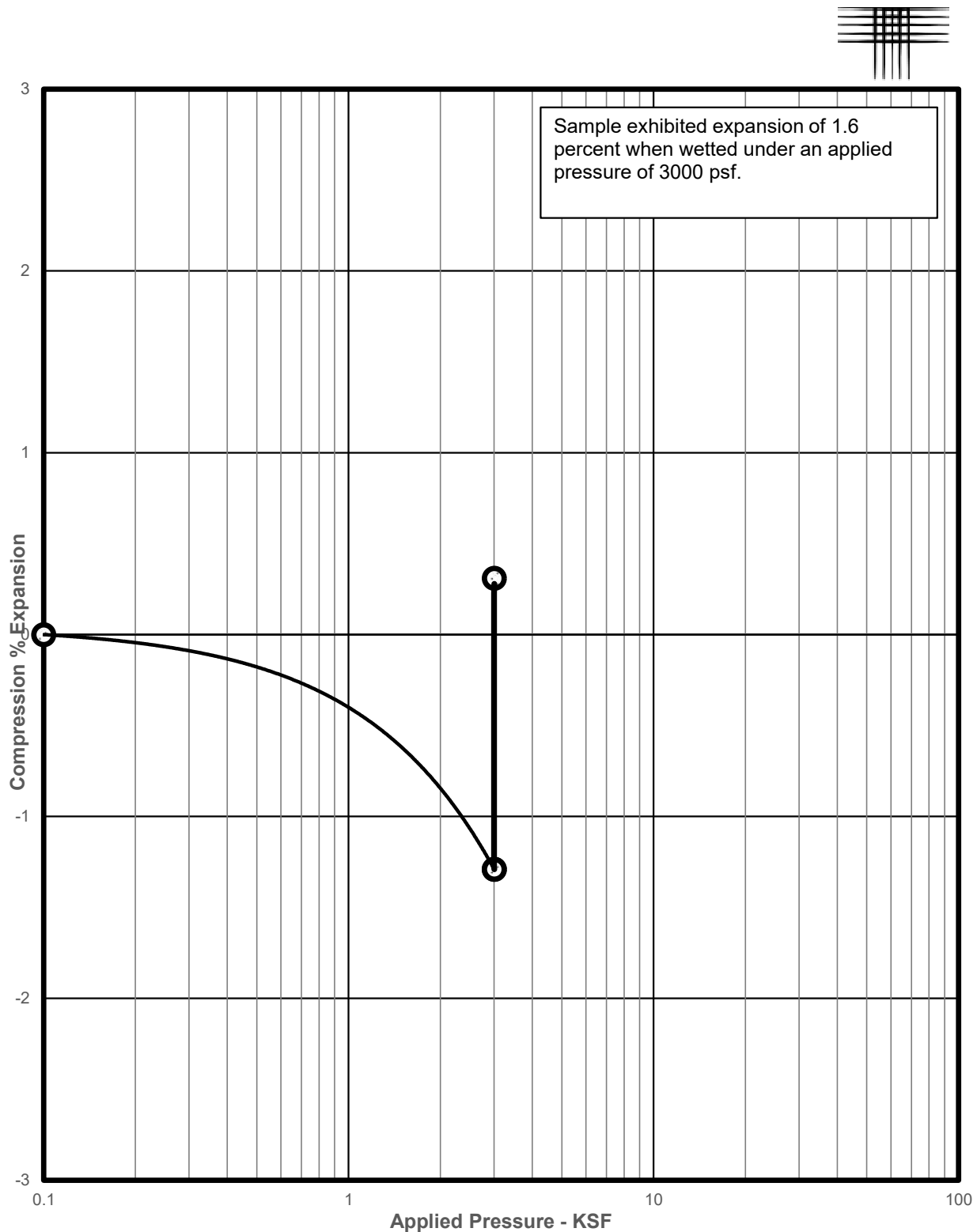


SAMPLE OF: CLAYSTONE
 FROM: TH-51 AT 24 FEET

DRY UNIT WEIGHT: 104 pcf
 MOISTURE CONTENT: 19.8 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 69

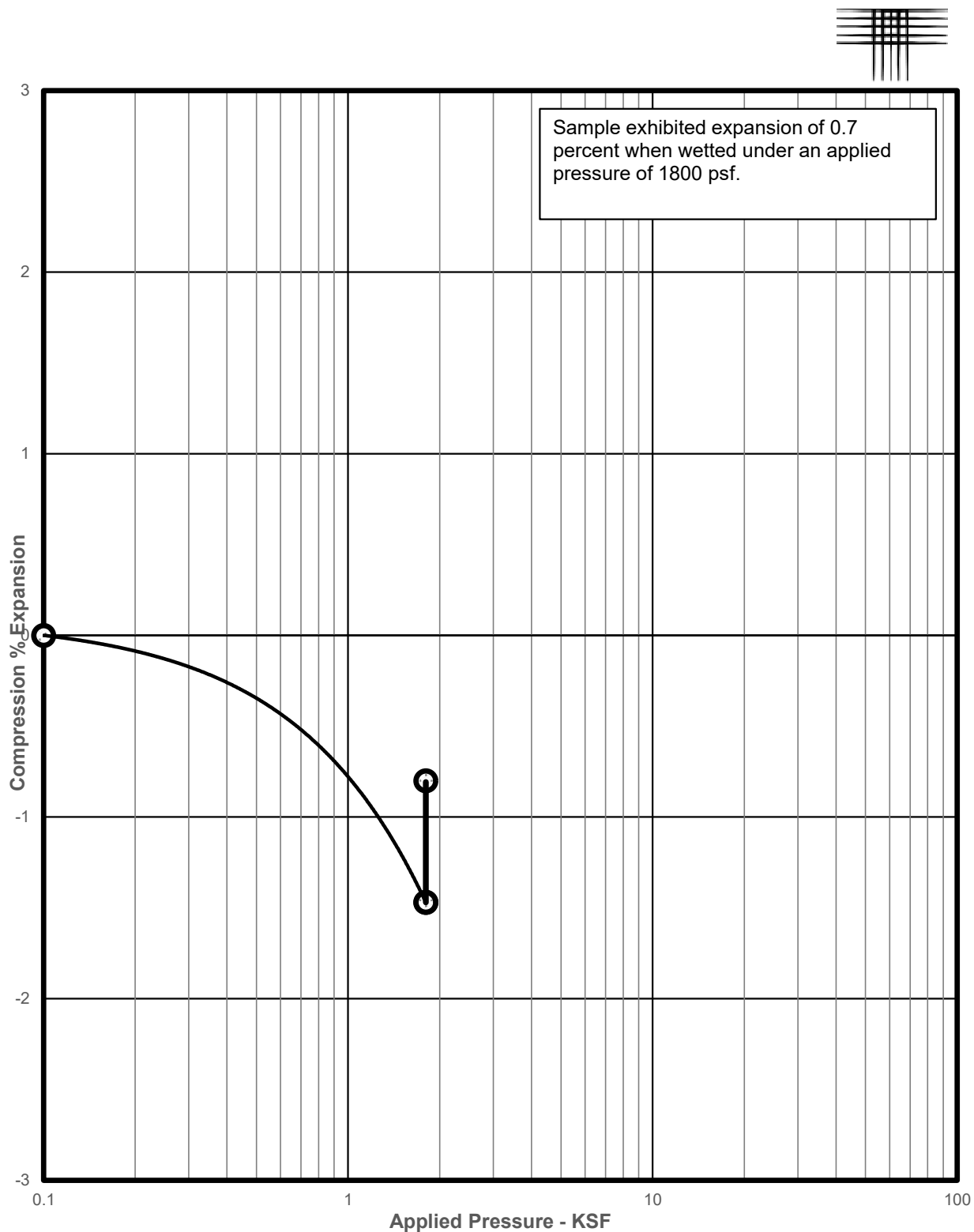


SAMPLE OF: CLAYSTONE
 FROM: TH-52 AT 19 FEET

DRY UNIT WEIGHT: 112 pcf
 MOISTURE CONTENT: 18.4 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 70

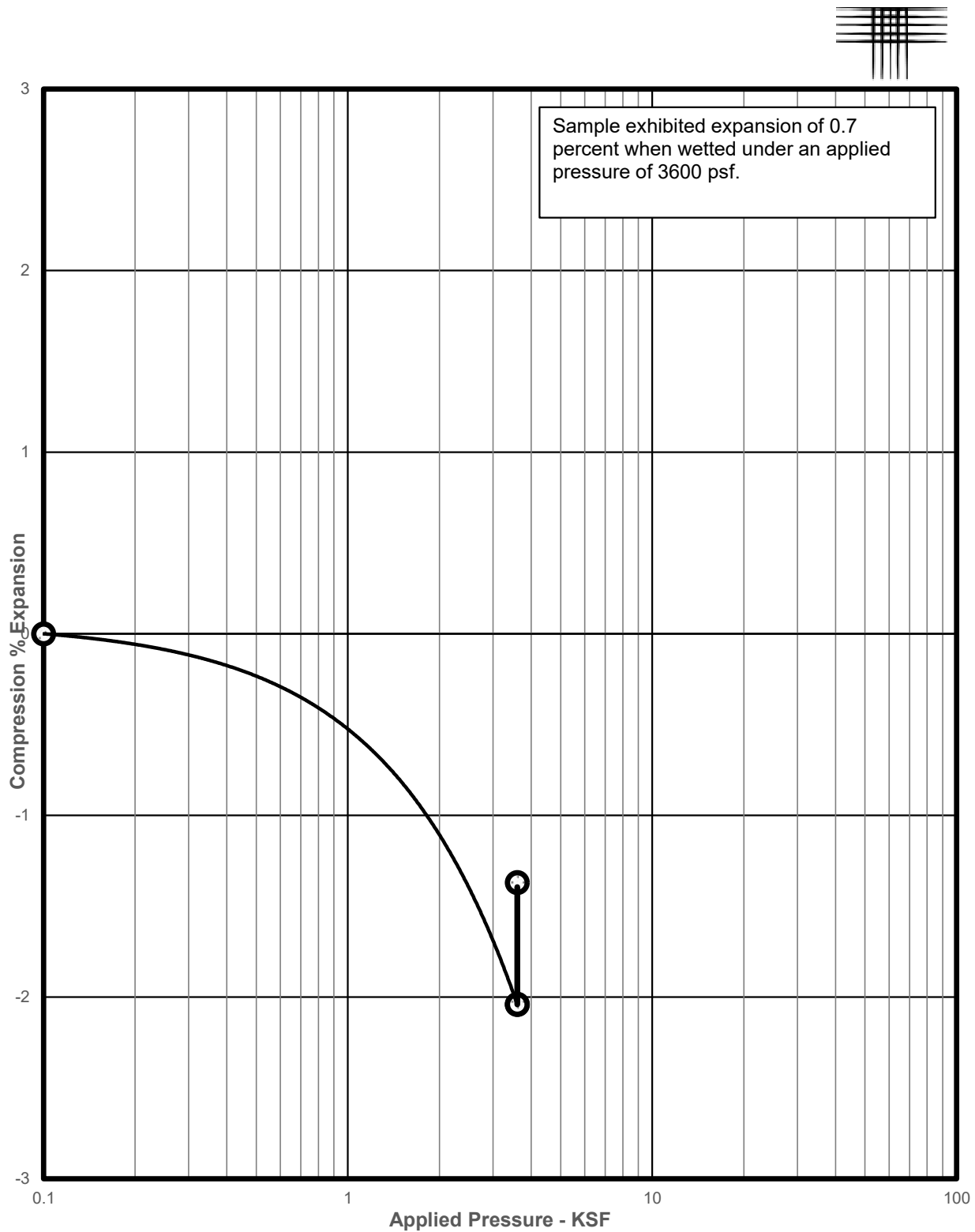


SAMPLE OF: CLAY, SANDY (CL)
 FROM: TH-53 AT 4 FEET

DRY UNIT WEIGHT: 103 pcf
 MOISTURE CONTENT: 12.0 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 71

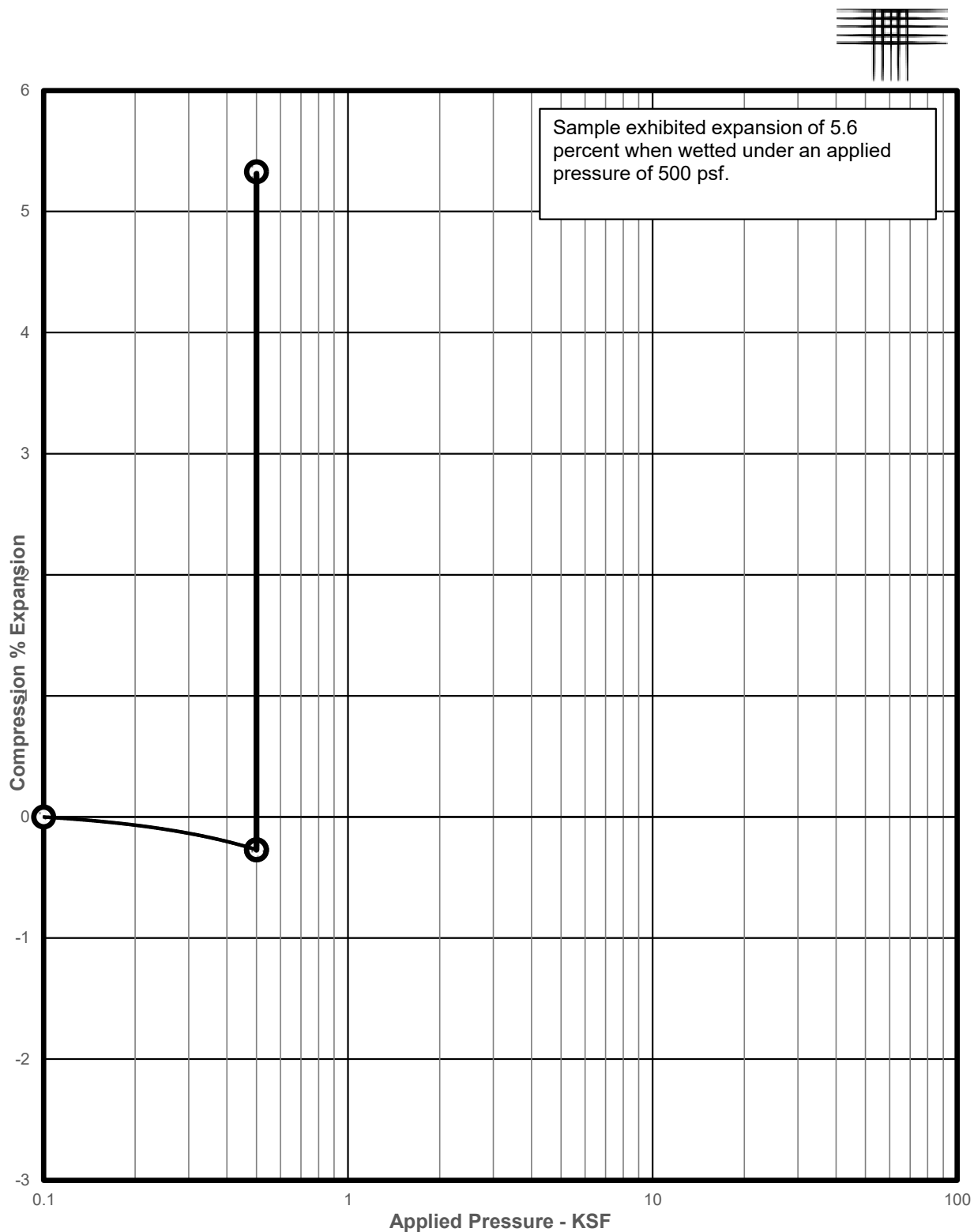


SAMPLE OF: CLAYSTONE
 FROM: TH-53 AT 19 FEET

DRY UNIT WEIGHT: 109 pcf
 MOISTURE CONTENT: 15.1 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 72

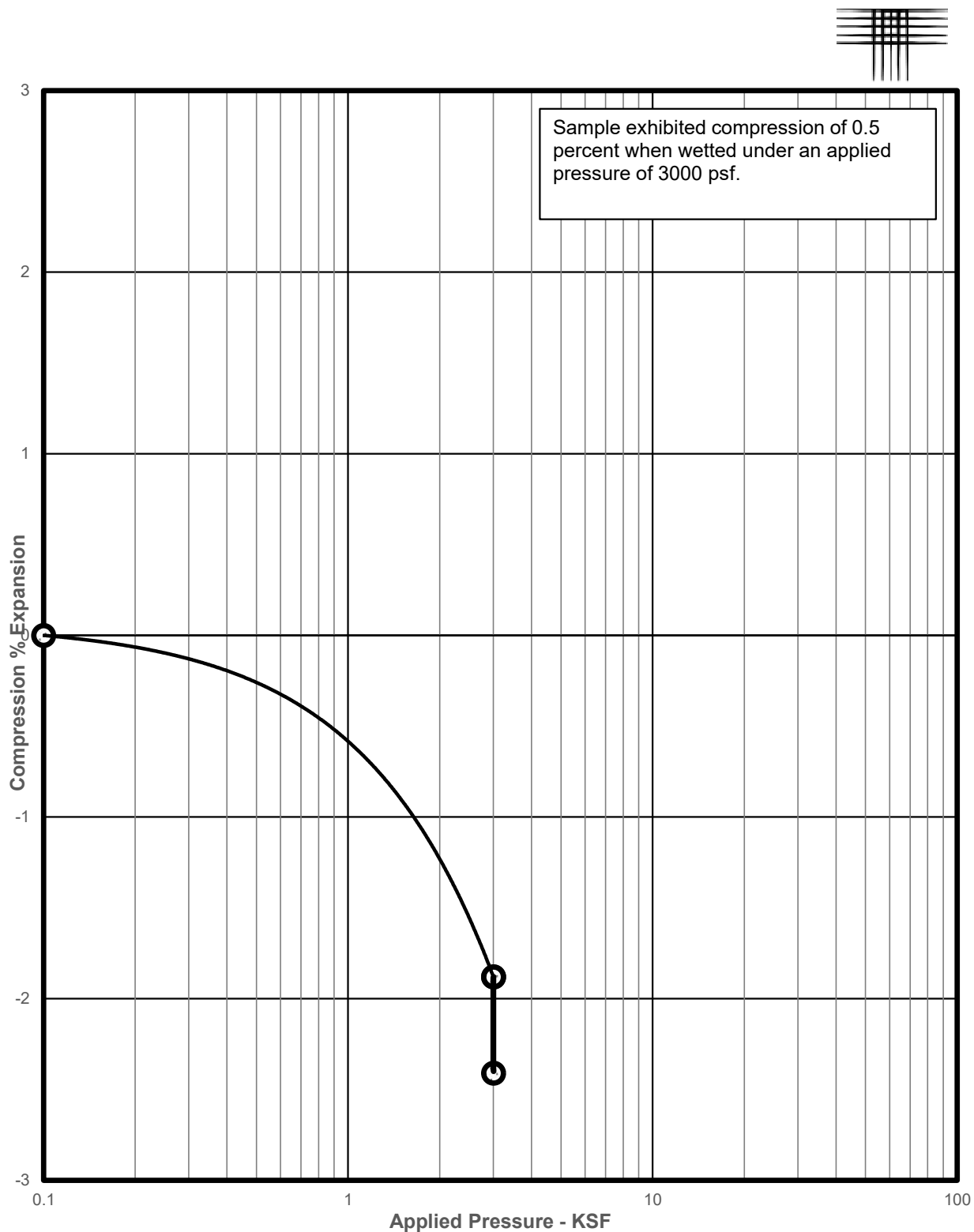


SAMPLE OF: CLAY, VERY SANDY (CL)
 FROM: TH-55 AT 4 FEET

DRY UNIT WEIGHT: 103 pcf
 MOISTURE CONTENT: 9.6 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 73

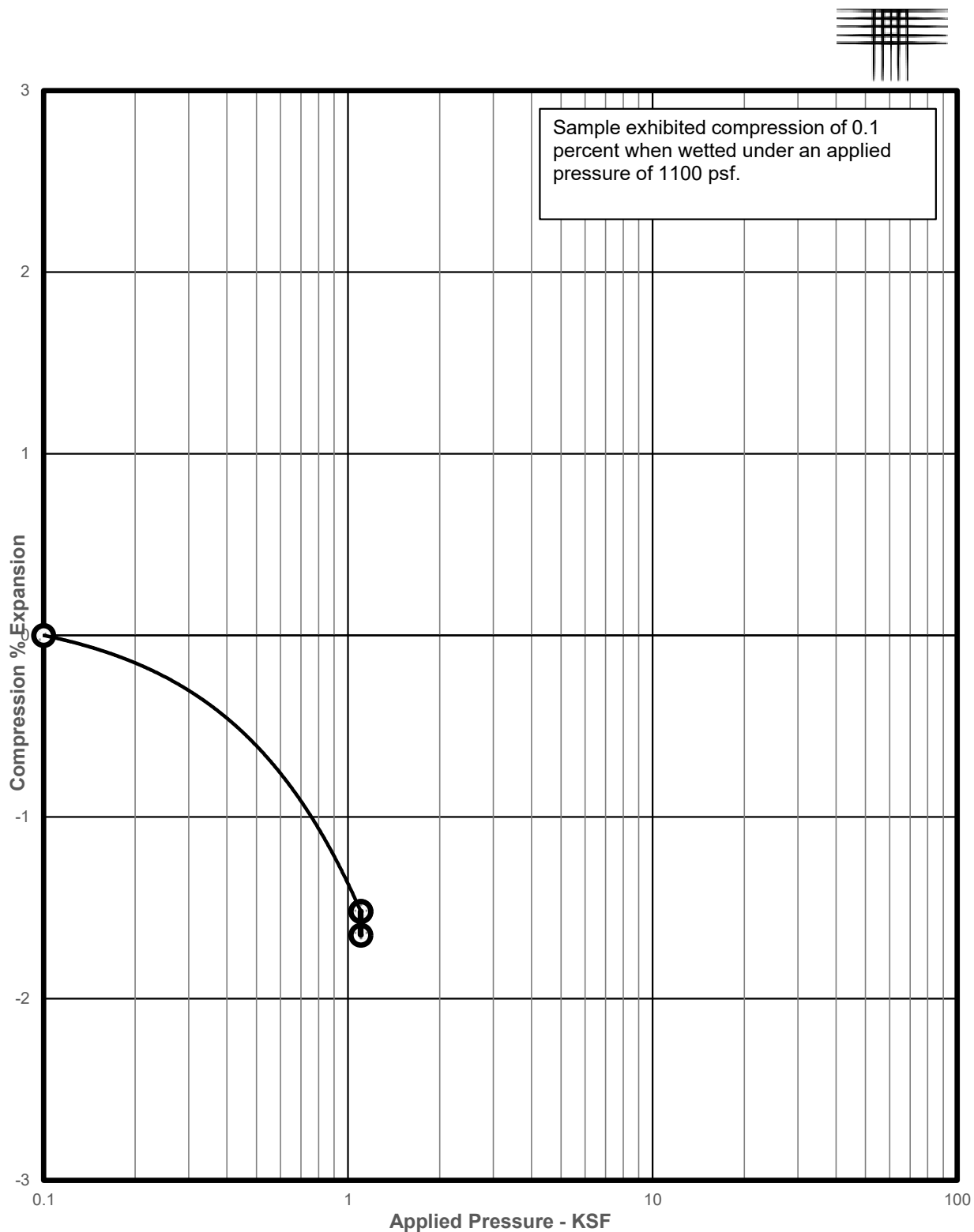


SAMPLE OF: CLAYSTONE
 FROM: TH-55 AT 24 FEET

DRY UNIT WEIGHT: 110 pcf
 MOISTURE CONTENT: 13.7 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 74

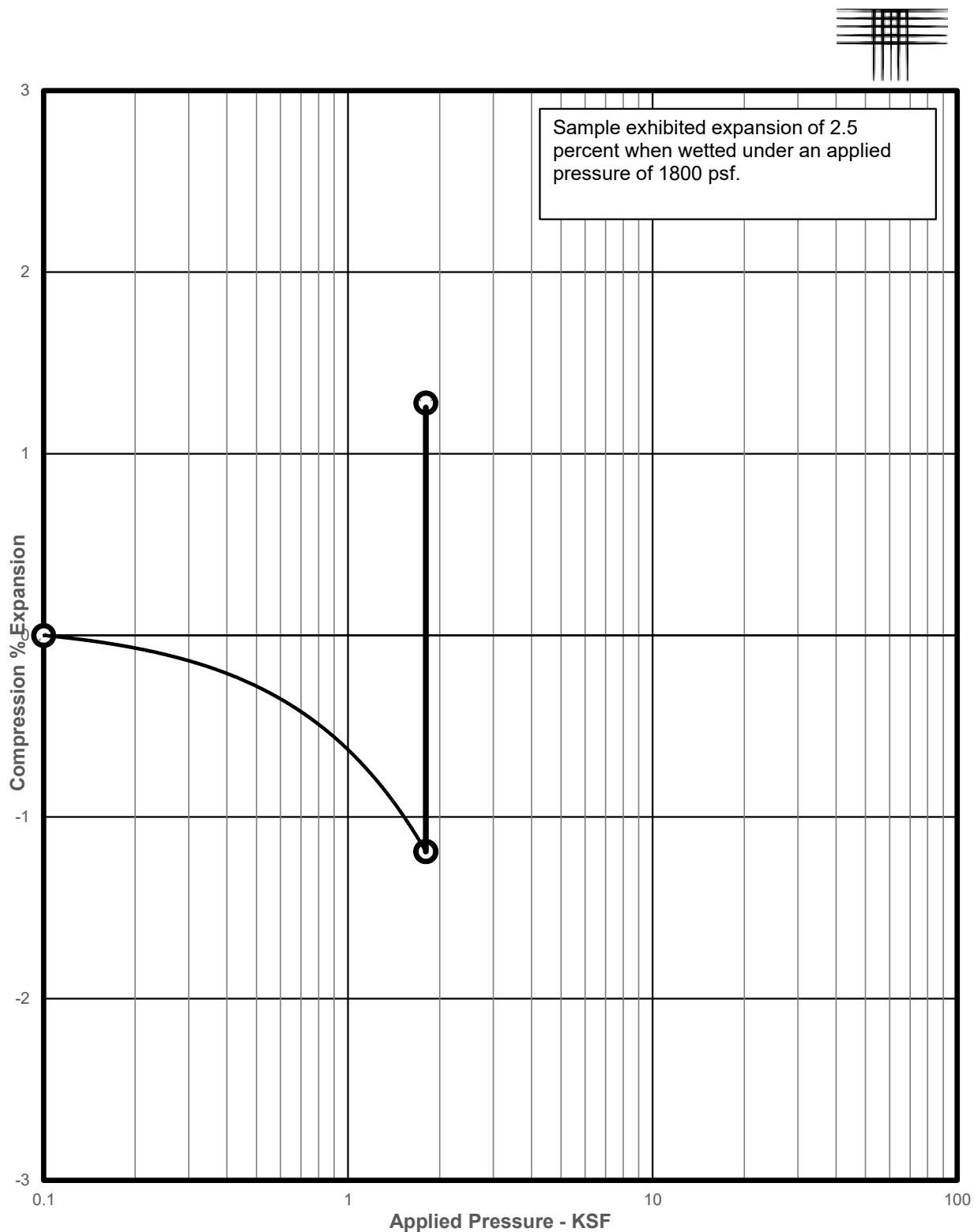


SAMPLE OF: SAND, CLAYEY (SC)
FROM: TH-56 AT 4 FEET

DRY UNIT WEIGHT: 108 pcf
MOISTURE CONTENT: 6.2 %

BMC INVESTMENTS
CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
Test Results** FIG. B- 75

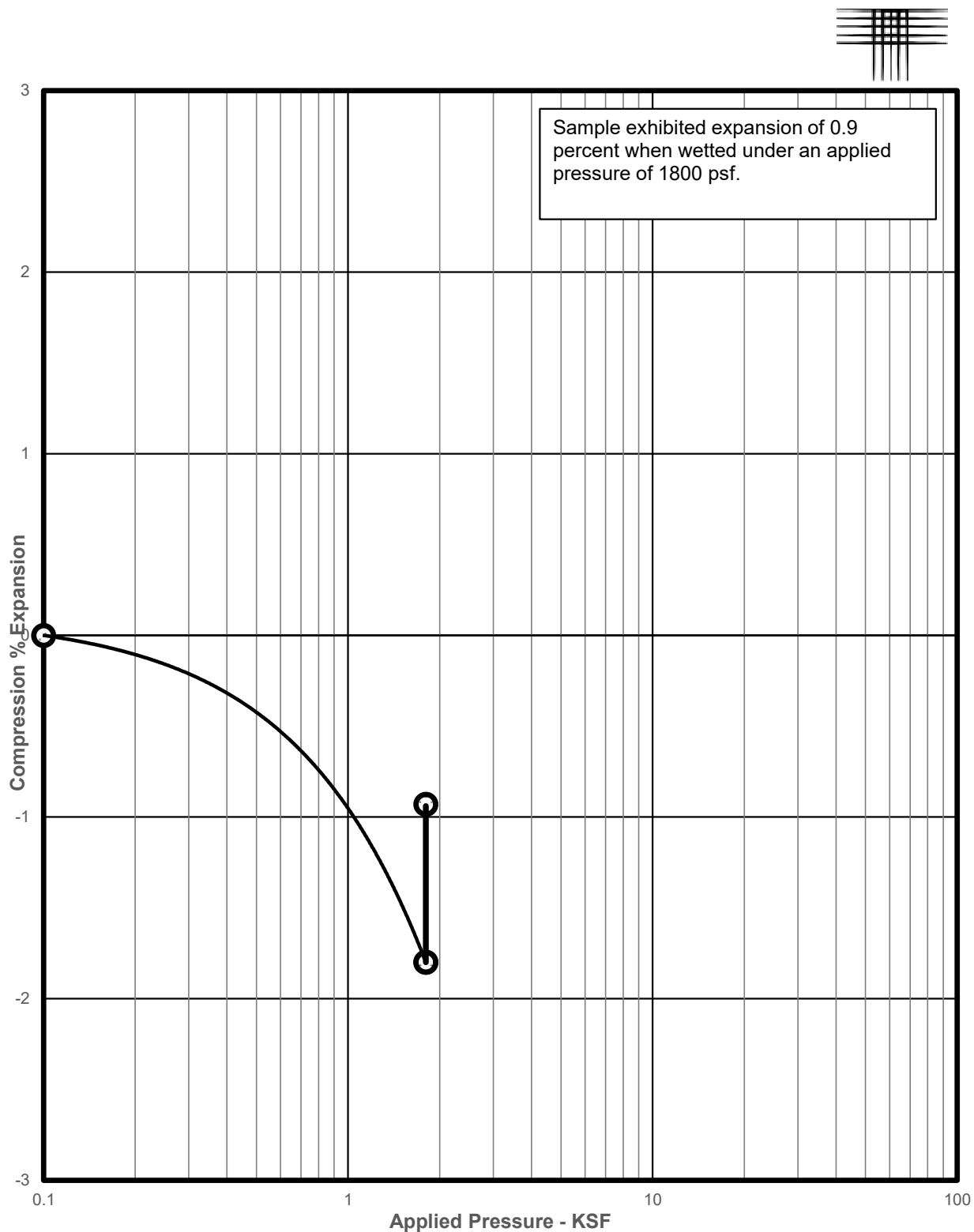


SAMPLE OF: CLAY, VERY SANDY (CL)
FROM: TH-56 AT 9 FEET

DRY UNIT WEIGHT: 110 pcf
MOISTURE CONTENT: 12.0 %

BMC INVESTMENTS
CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
Test Results** FIG. B- 76

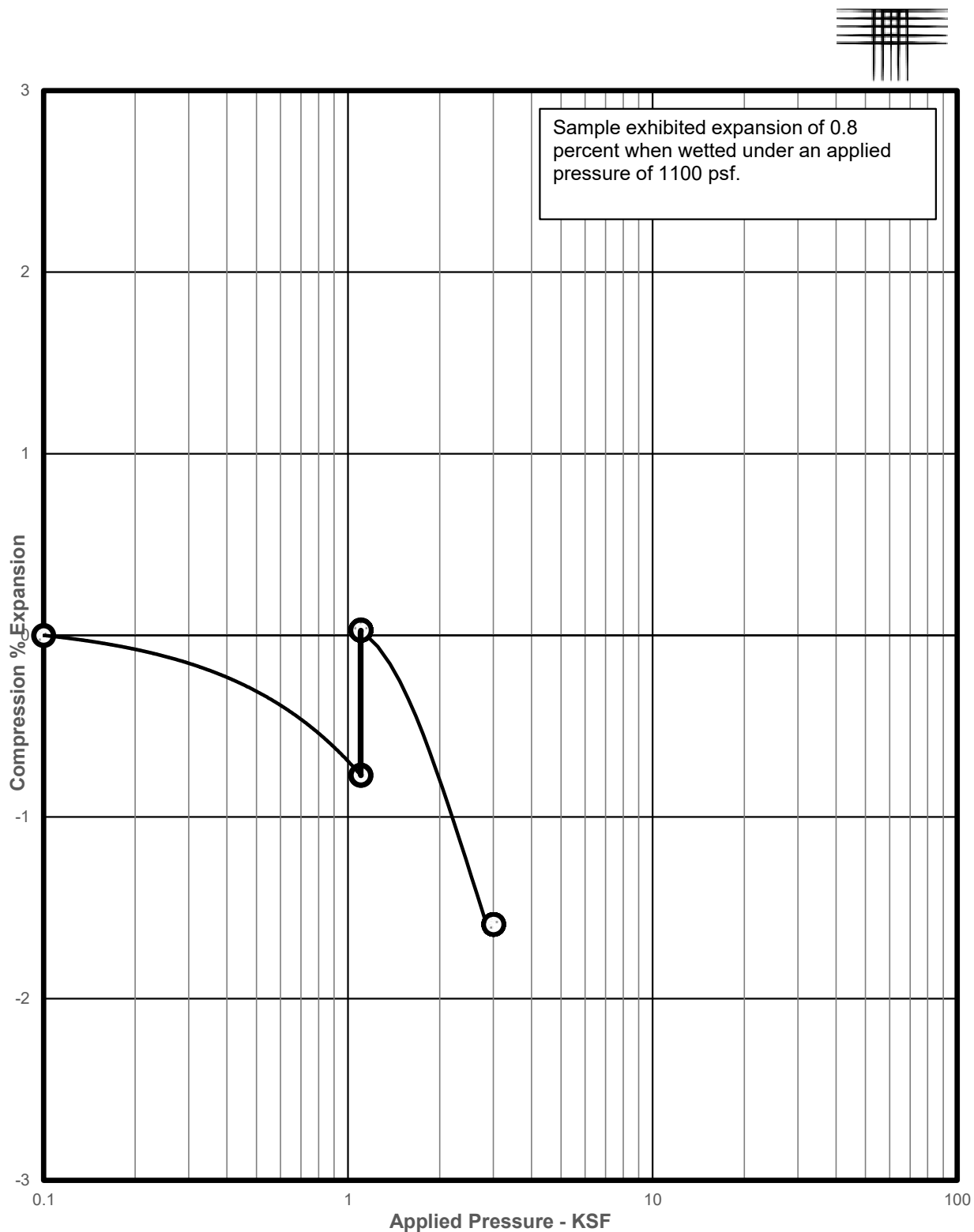


SAMPLE OF: WEATHERED CLAYSTONE
 FROM: TH-60 AT 14 FEET

DRY UNIT WEIGHT: 91 pcf
 MOISTURE CONTENT: 27.8 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 77

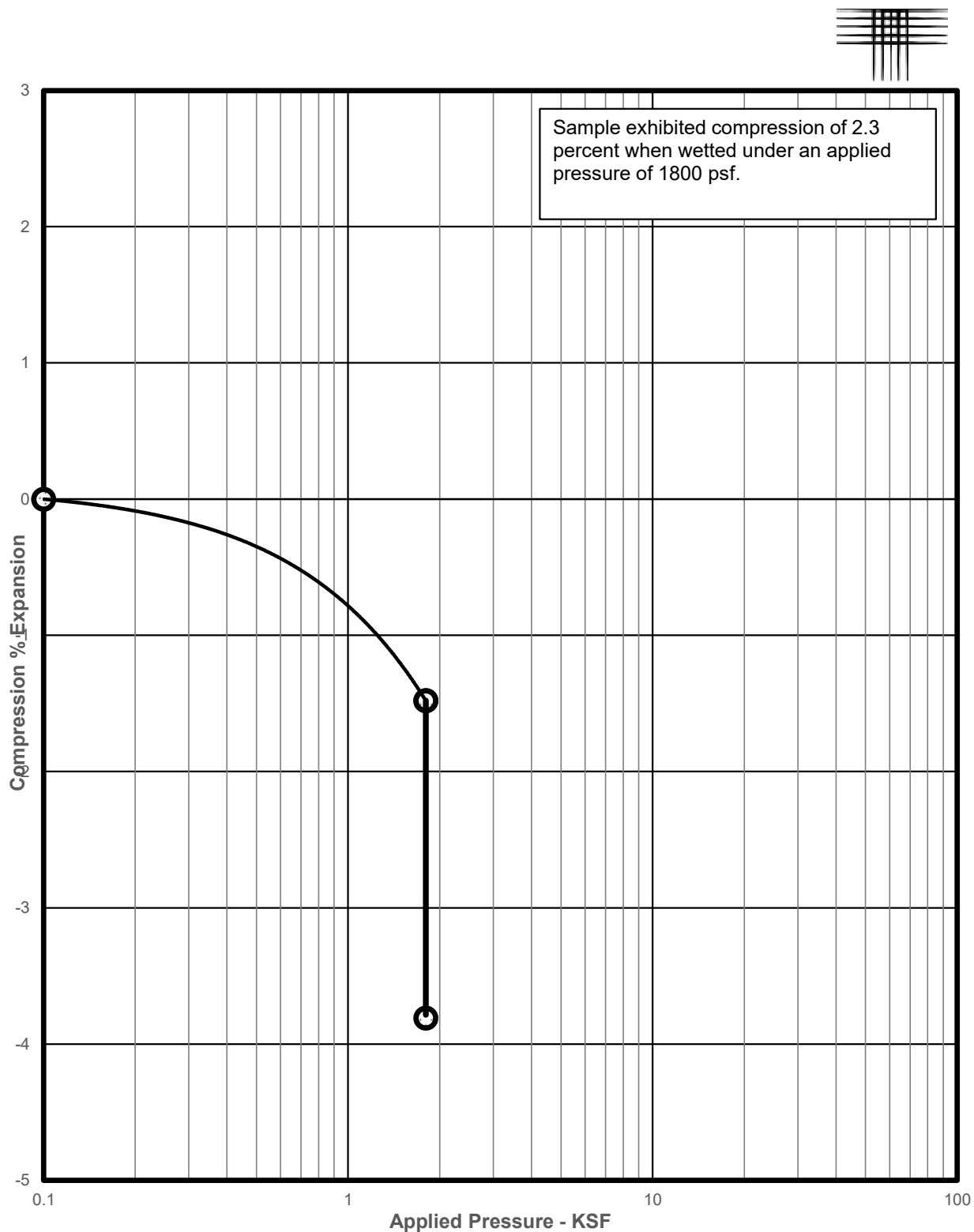


SAMPLE OF: CLAY, VERY SANDY (CL)
 FROM: TH-61 AT 4 FEET

DRY UNIT WEIGHT: 114 pcf
 MOISTURE CONTENT: 9.7 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 78

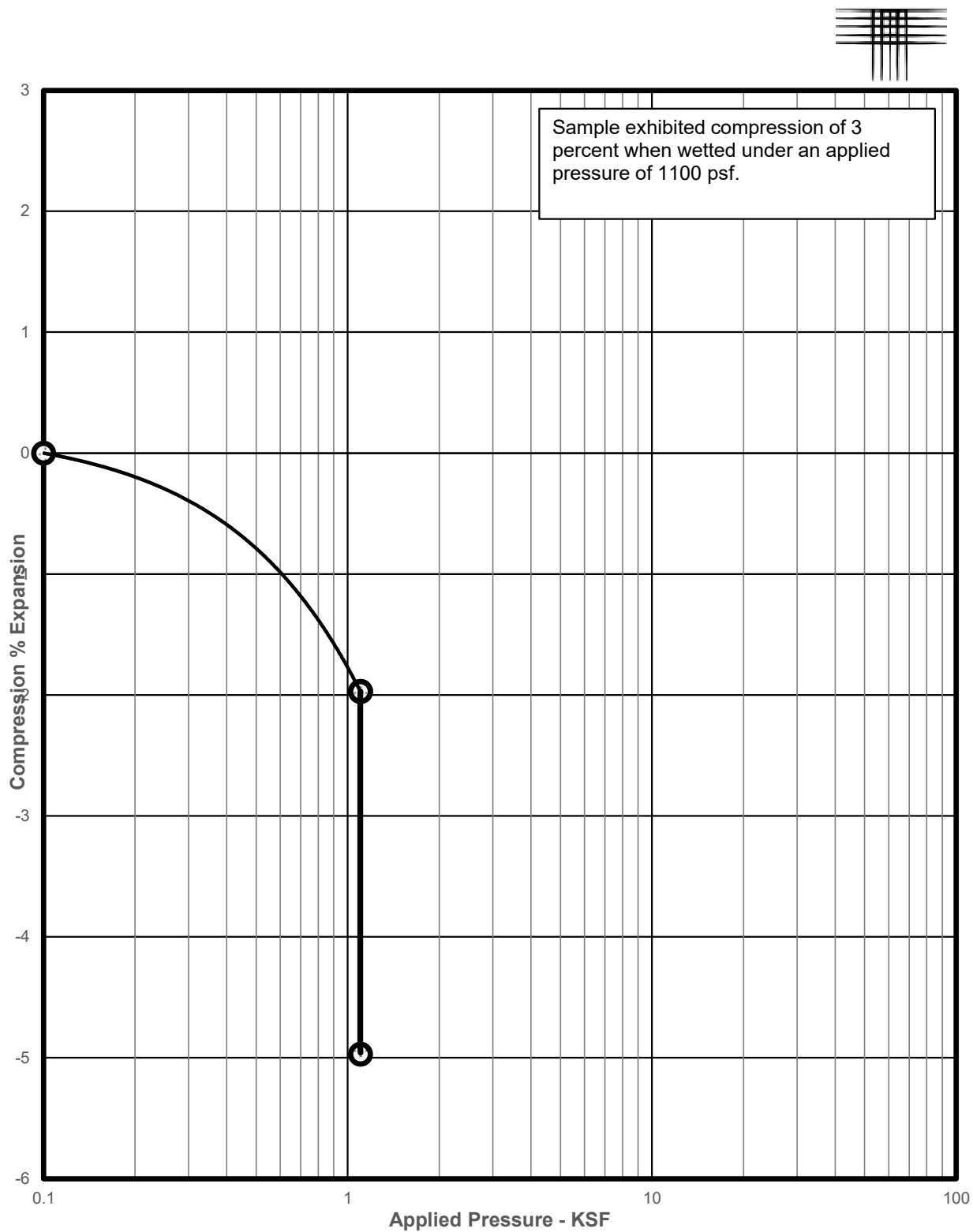


SAMPLE OF: SAND, CLAYEY (SC)
 FROM: TH-61 AT 9 FEET

DRY UNIT WEIGHT: 104 pcf
 MOISTURE CONTENT: 6.3 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 79

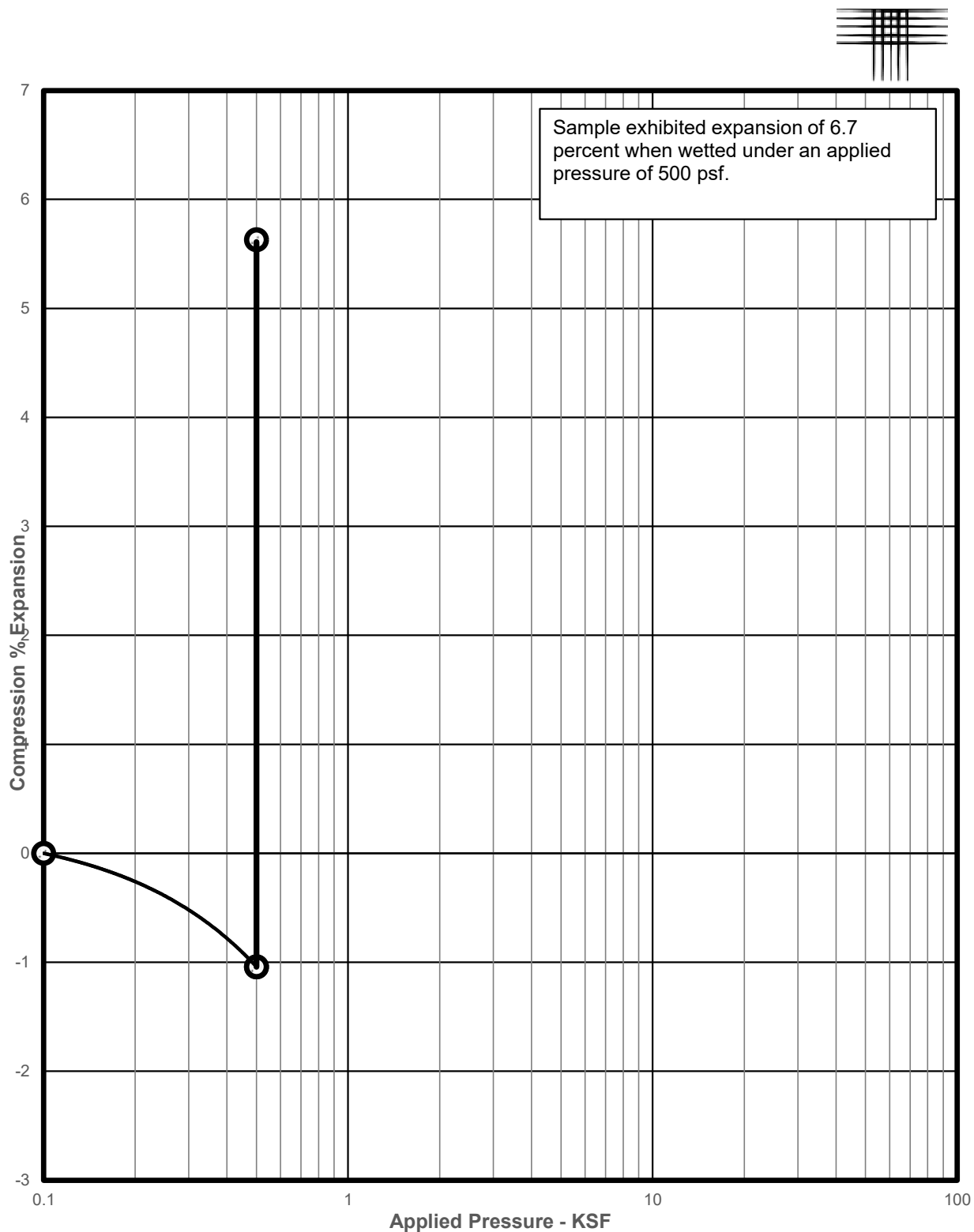


SAMPLE OF: SAND, CLAYEY (SC)
 FROM: TH-61 AT 14 FEET

DRY UNIT WEIGHT: 111 pcf
 MOISTURE CONTENT: 4.8 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 80

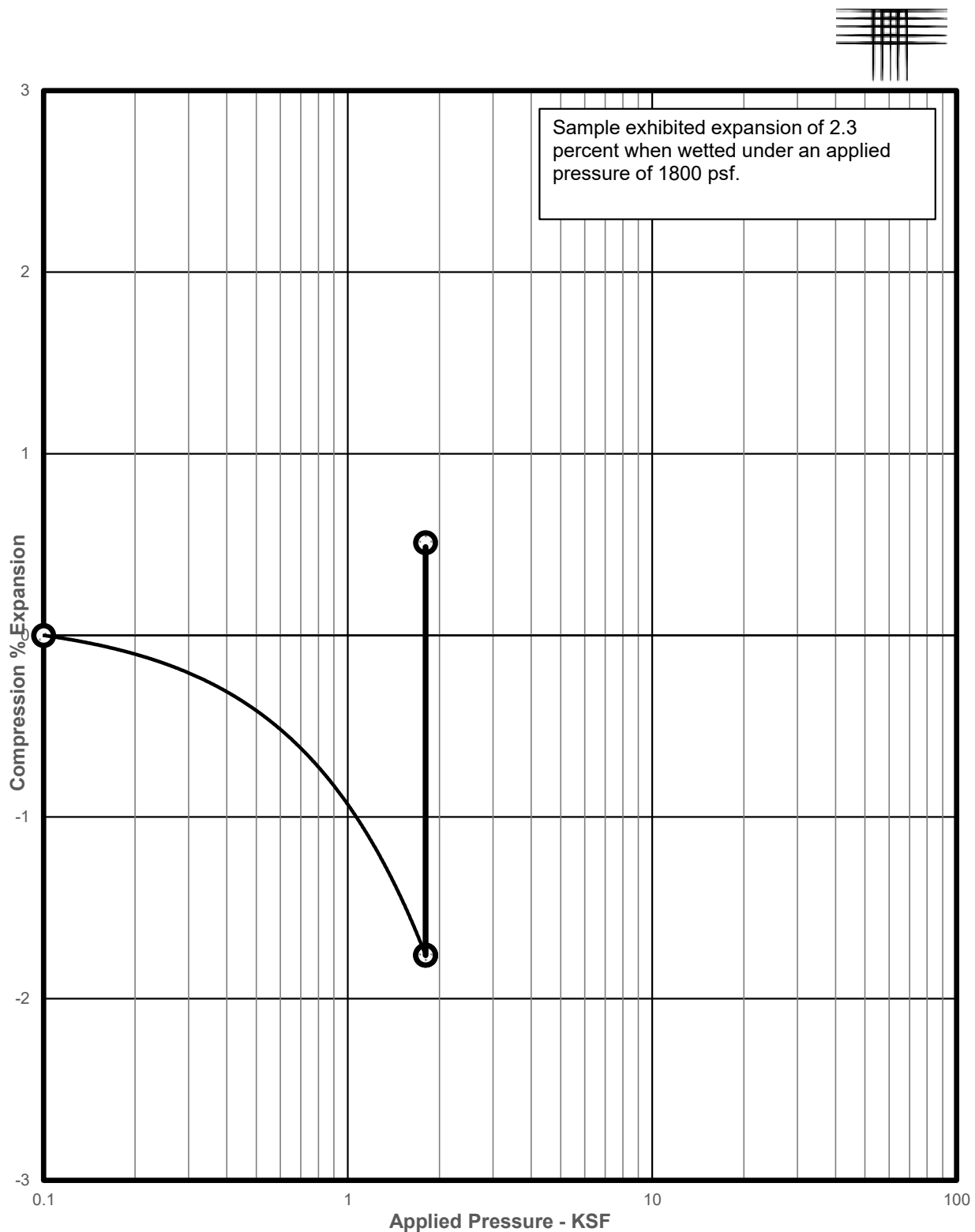


SAMPLE OF: CLAYSTONE
 FROM: TH-63 AT 9 FEET

DRY UNIT WEIGHT: 105 pcf
 MOISTURE CONTENT: 19.6 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 81

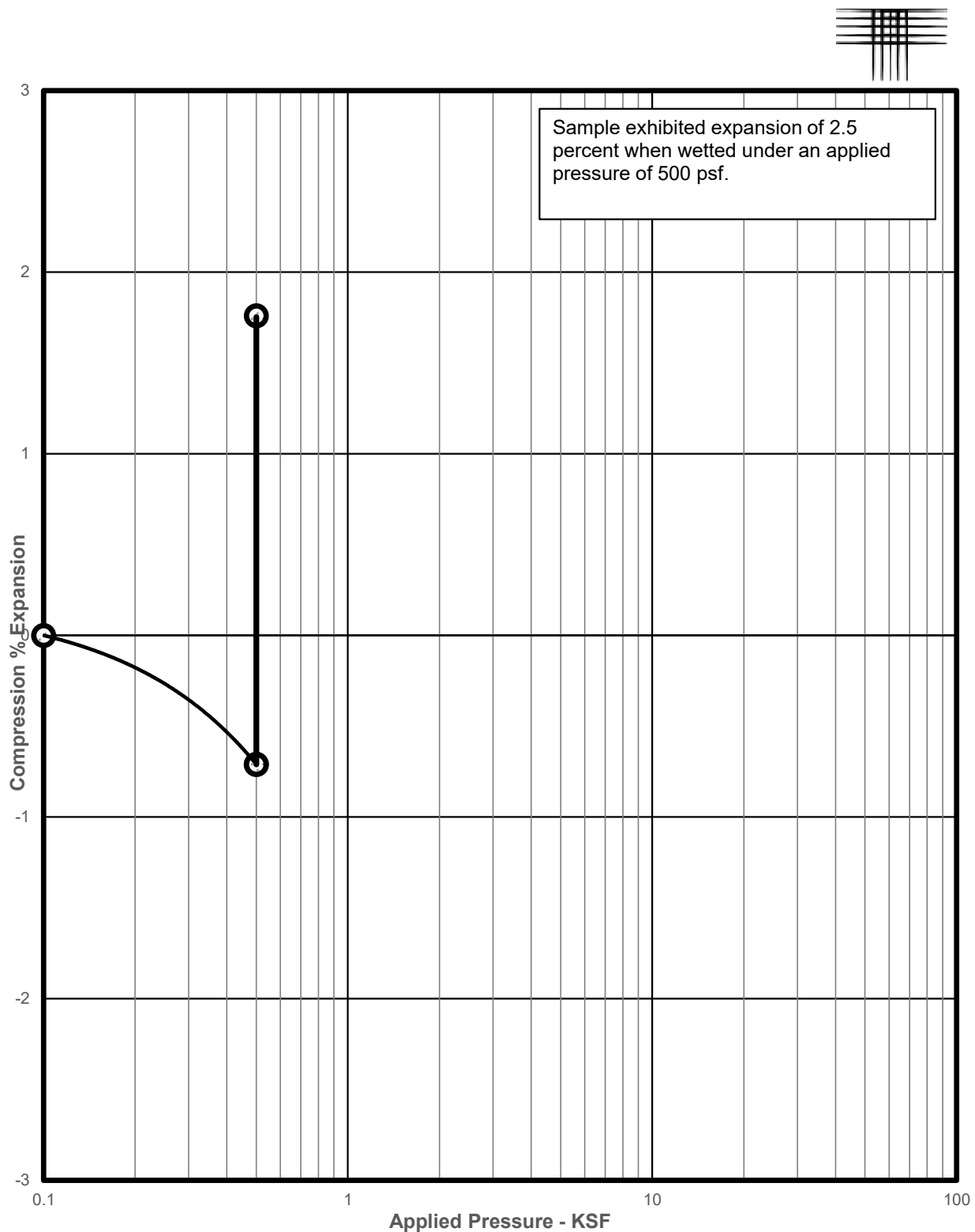


SAMPLE OF: CLAYSTONE
 FROM: TH-63 AT 24 FEET

DRY UNIT WEIGHT: 105 pcf
 MOISTURE CONTENT: 18.6 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 82

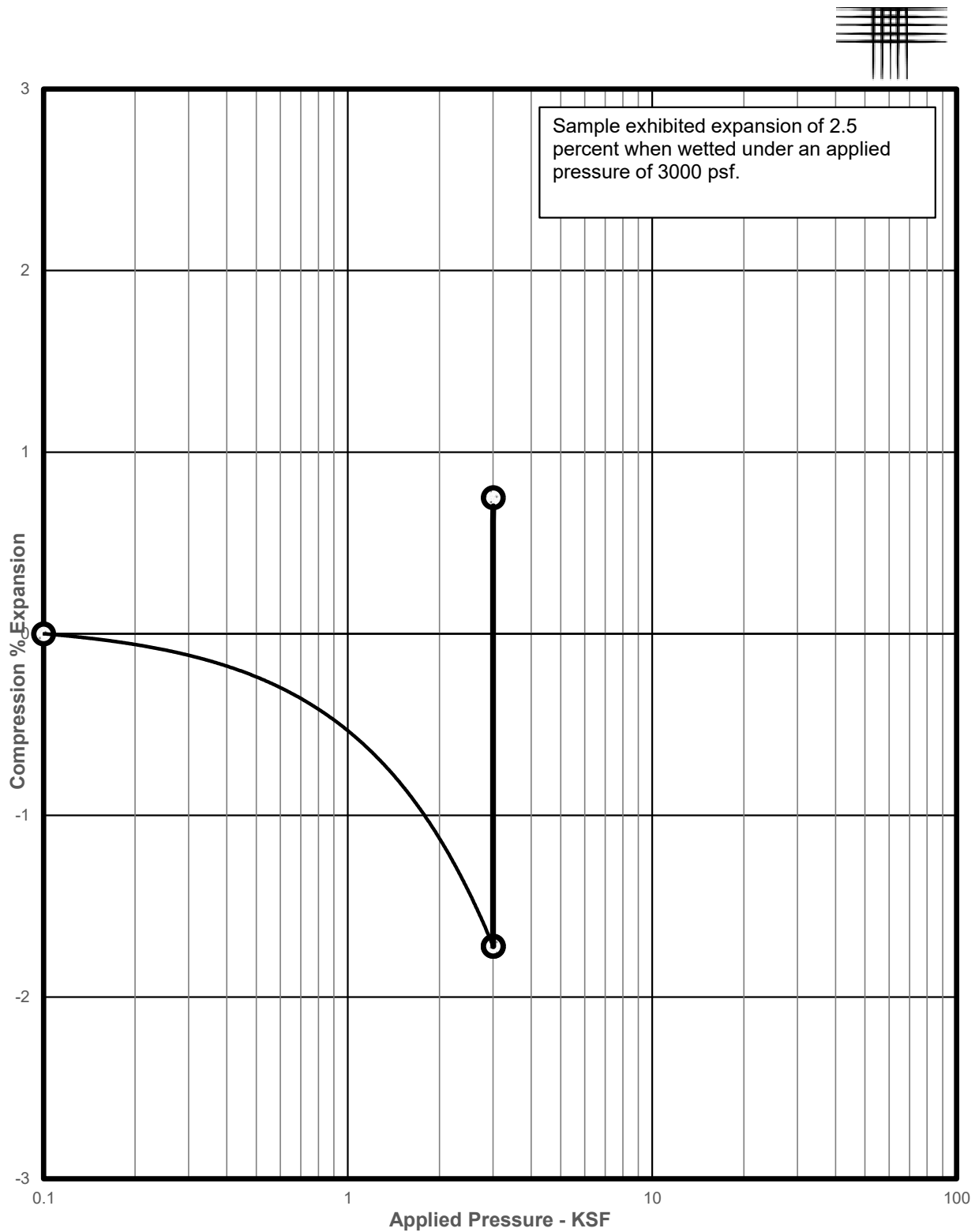


SAMPLE OF: CLAYSTONE
 FROM: TH-64 AT 14 FEET

DRY UNIT WEIGHT: 119 pcf
 MOISTURE CONTENT: 13.6 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 83

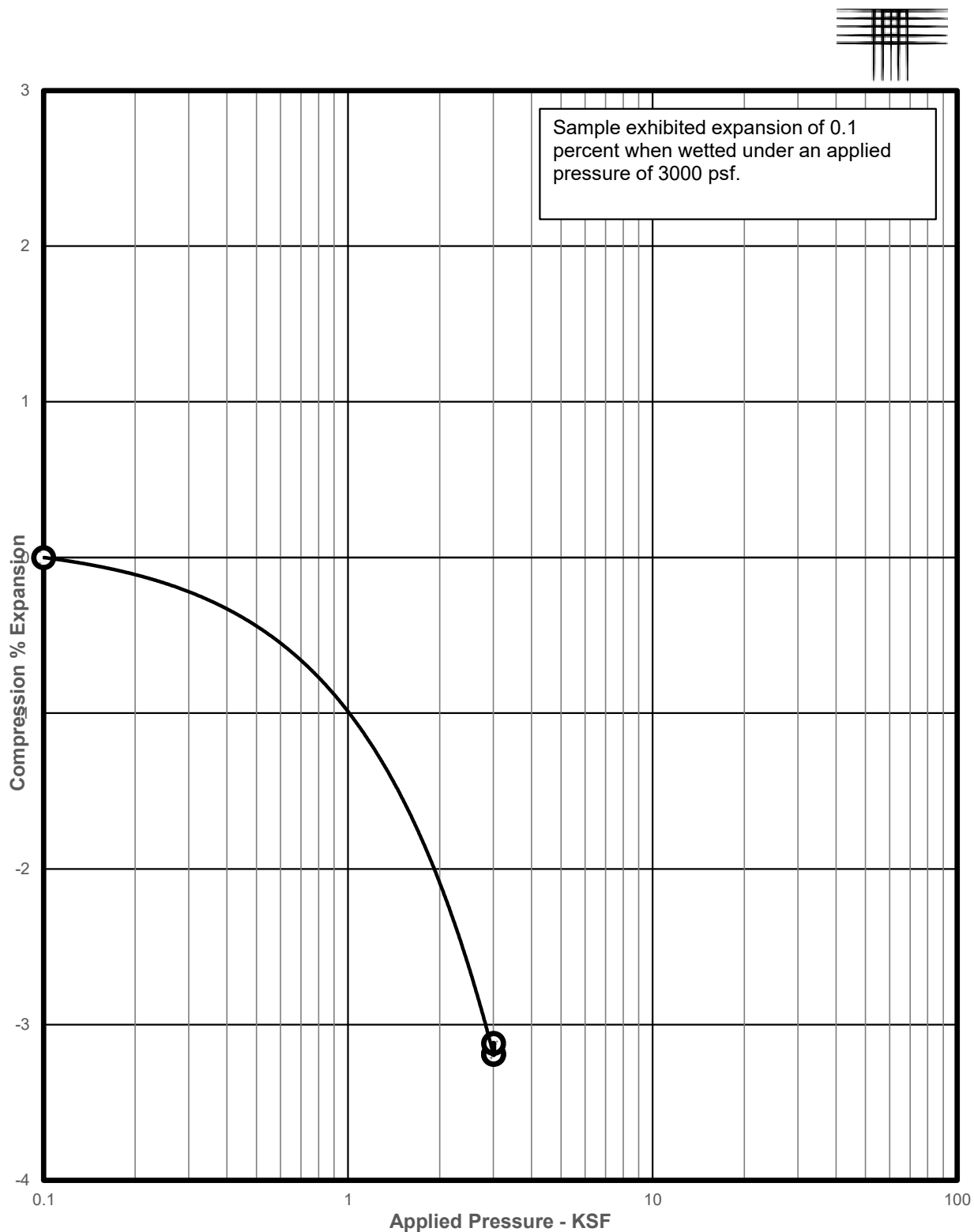


SAMPLE OF: CLAYSTONE
 FROM: TH-65 AT 34 FEET

DRY UNIT WEIGHT: 112 pcf
 MOISTURE CONTENT: 17.9 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 84

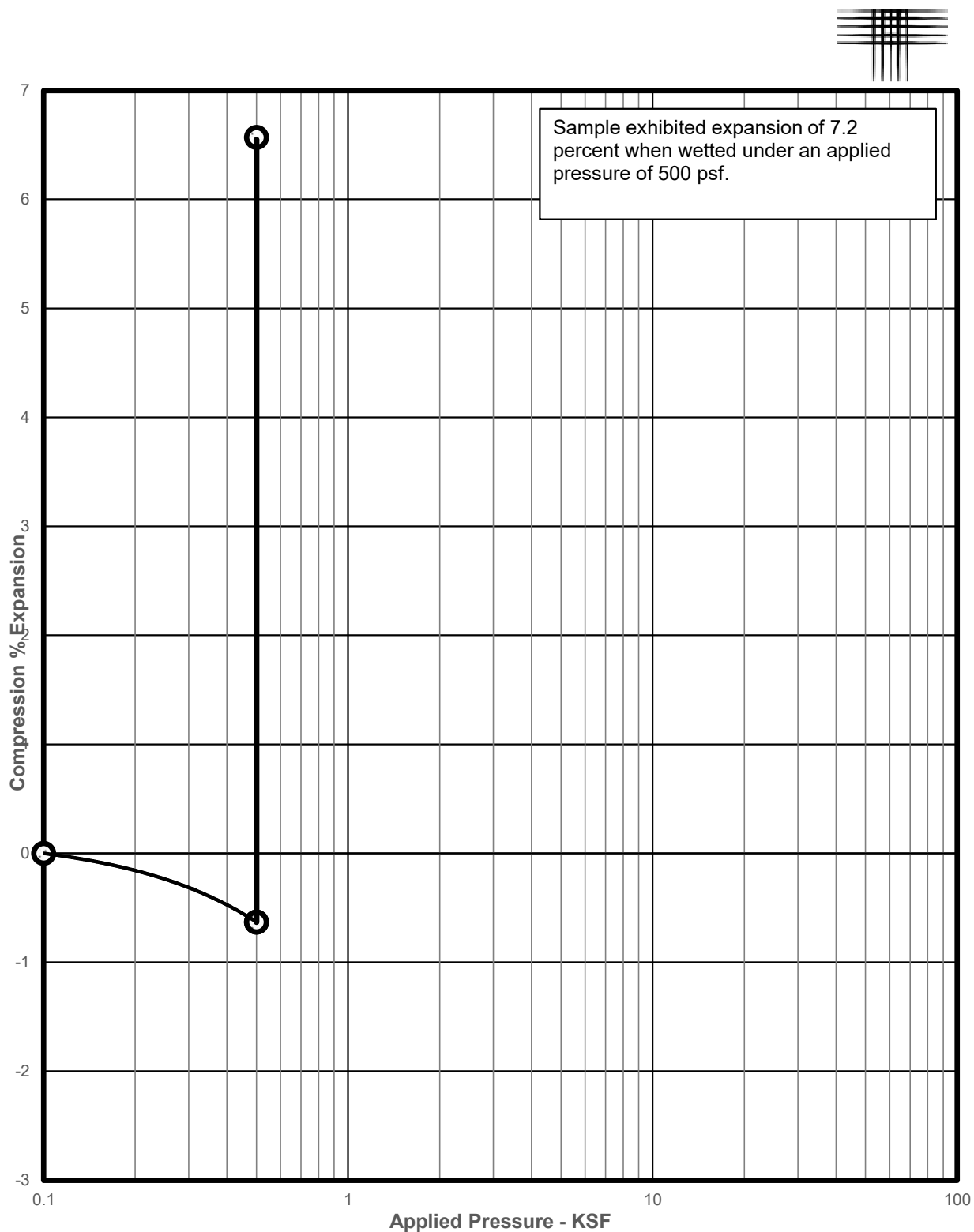


SAMPLE OF: SANDSTONE
 FROM: TH-66 AT 19 FEET

DRY UNIT WEIGHT: 114 pcf
 MOISTURE CONTENT: 12.2 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 85

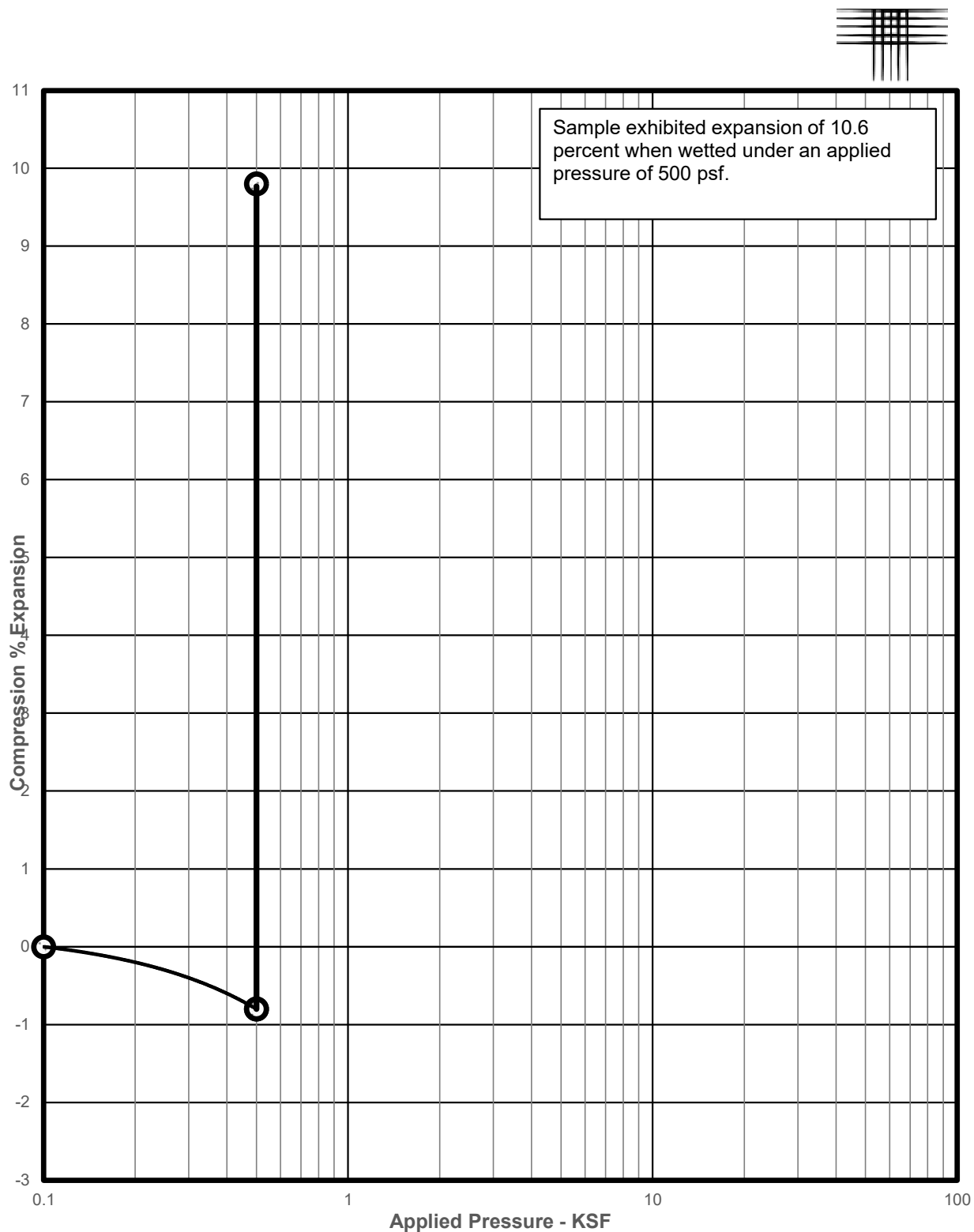


SAMPLE OF: CLAYSTONE
 FROM: TH-68 AT 4 FEET

DRY UNIT WEIGHT: 109 pcf
 MOISTURE CONTENT: 14.4 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 86

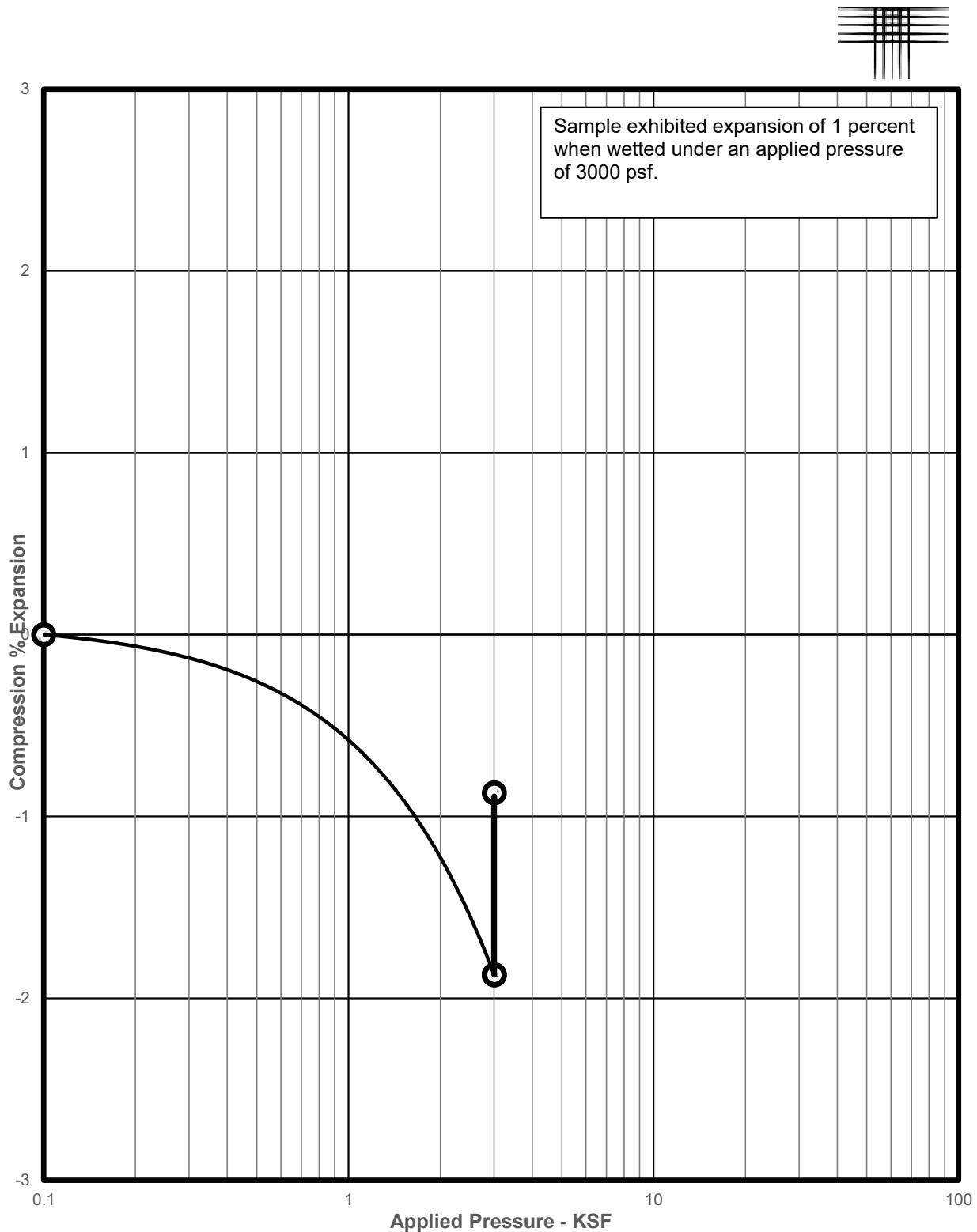


SAMPLE OF: CLAYSTONE
 FROM: TH-69 AT 4 FEET

DRY UNIT WEIGHT: 115 pcf
 MOISTURE CONTENT: 11.8 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 87

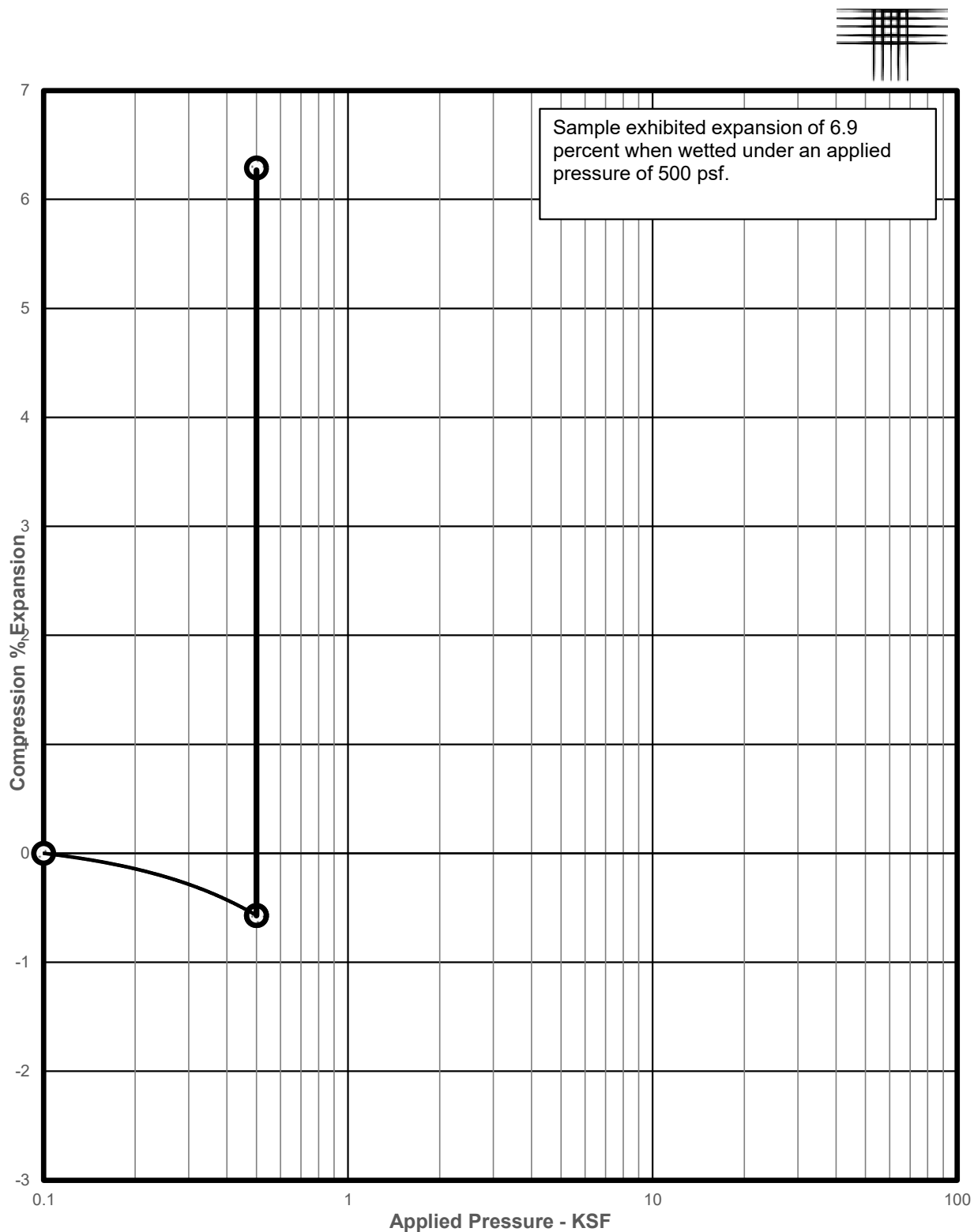


SAMPLE OF: CLAYSTONE
 FROM: TH-69 AT 24 FEET

DRY UNIT WEIGHT: 110 pcf
 MOISTURE CONTENT: 17.4 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 88

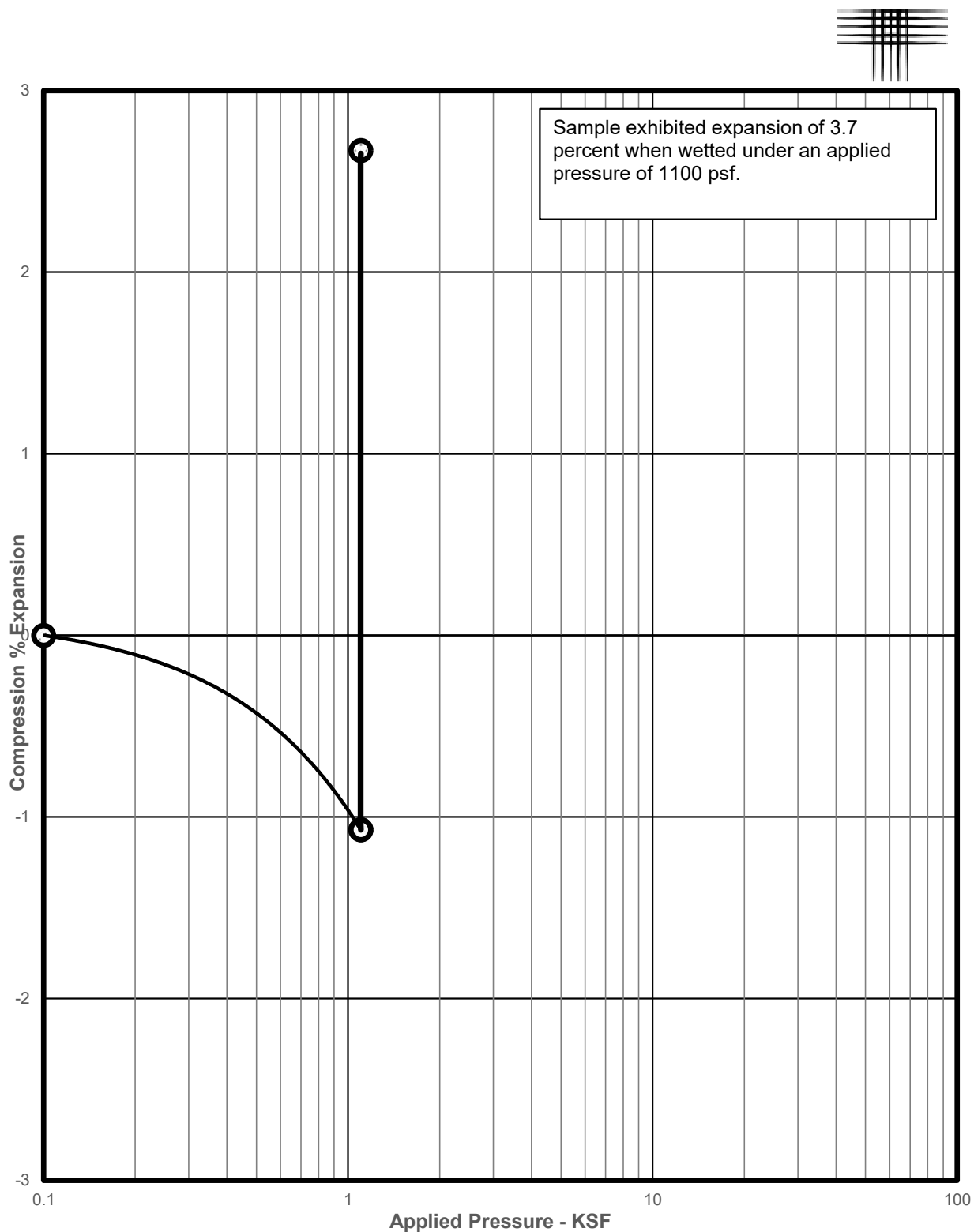


SAMPLE OF: CLAY, SANDY (CL)
 FROM: TH-70 AT 4 FEET

DRY UNIT WEIGHT: 112 pcf
 MOISTURE CONTENT: 11.9 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 89

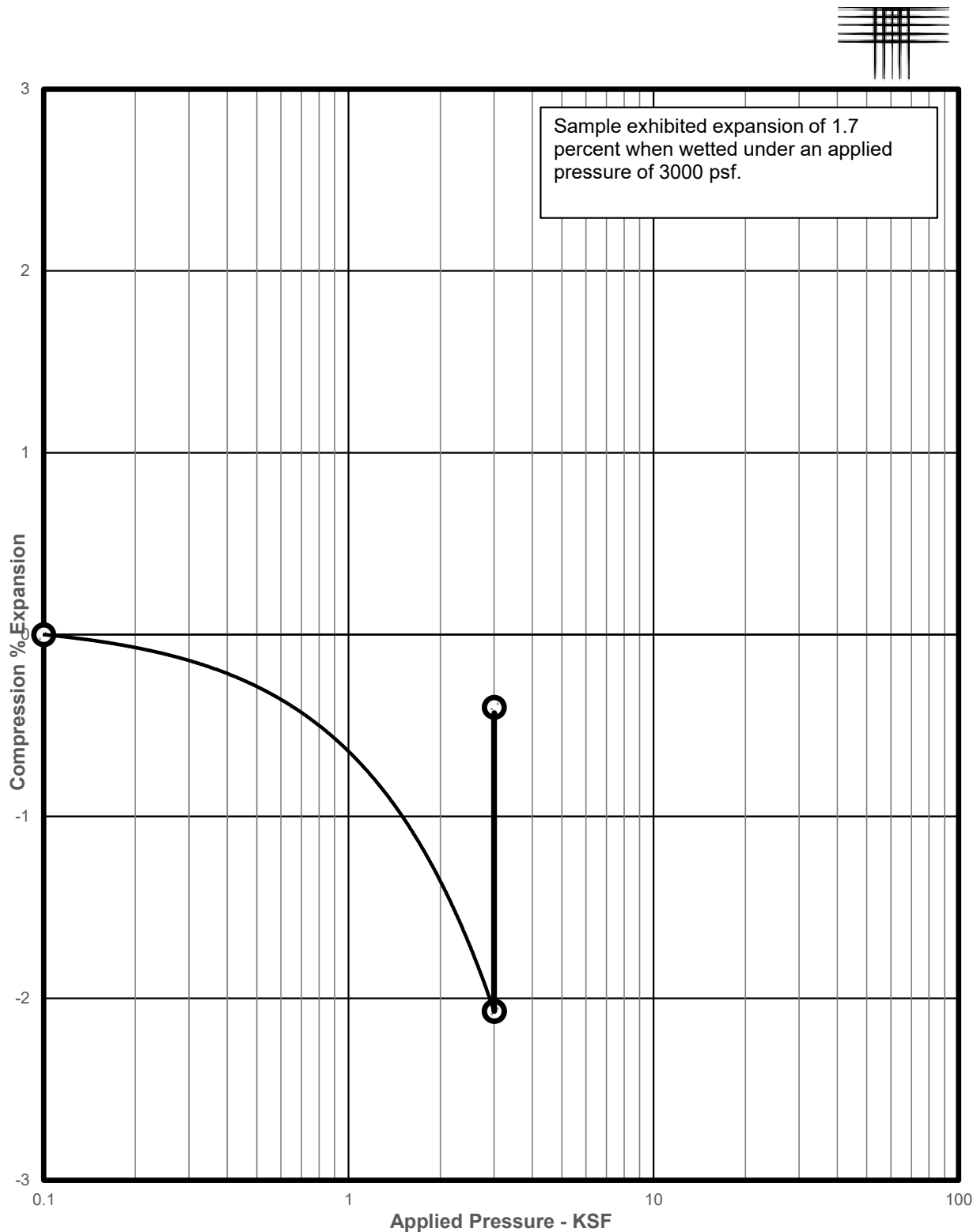


SAMPLE OF: CLAYSTONE
 FROM: TH-70 AT 14 FEET

DRY UNIT WEIGHT: 106 pcf
 MOISTURE CONTENT: 17.1 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 90

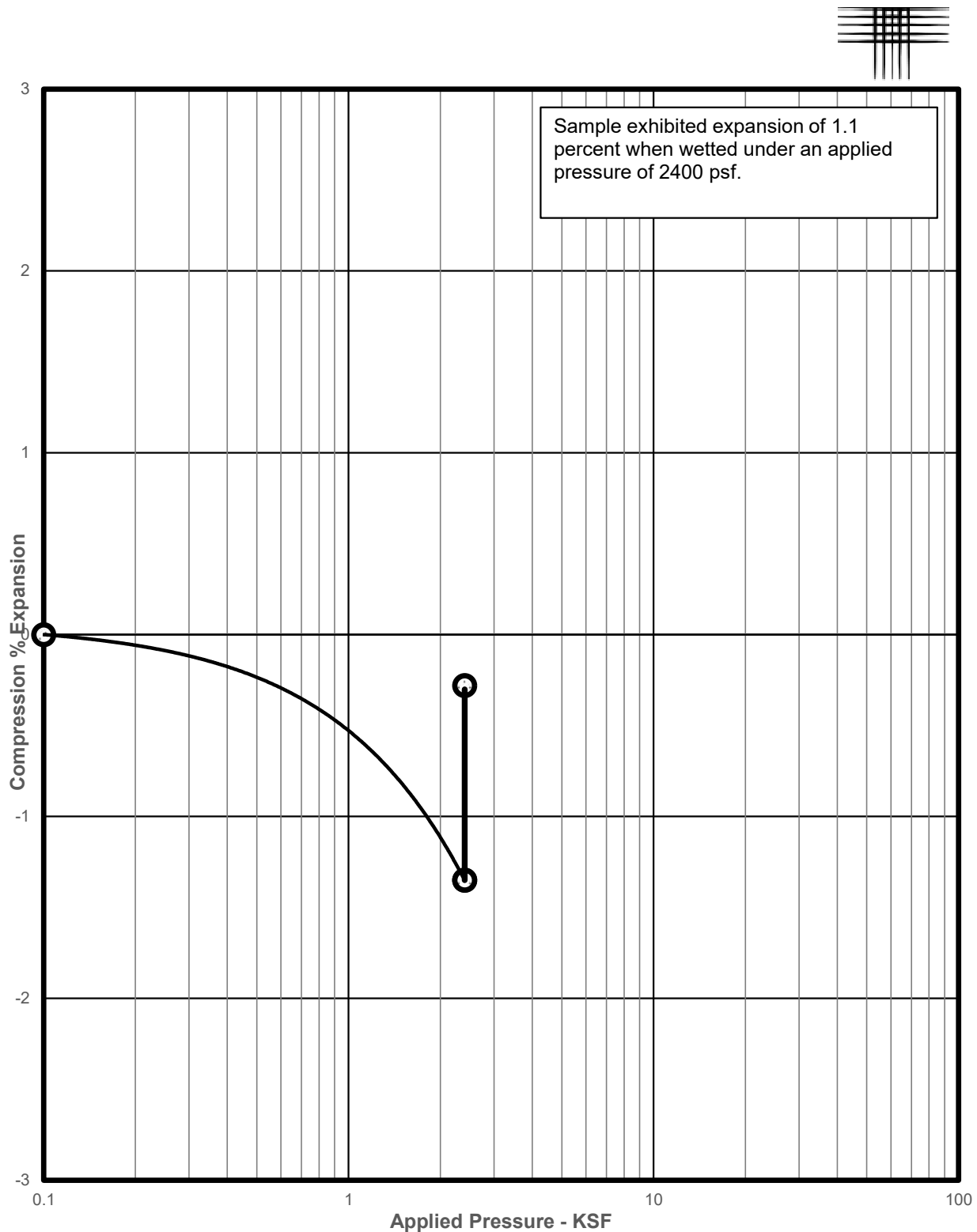


SAMPLE OF: CLAYSTONE
FROM: TH-70 AT 29 FEET

DRY UNIT WEIGHT: 115 pcf
MOISTURE CONTENT: 15.8 %

BMC INVESTMENTS
CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
Test Results** FIG. B- 91

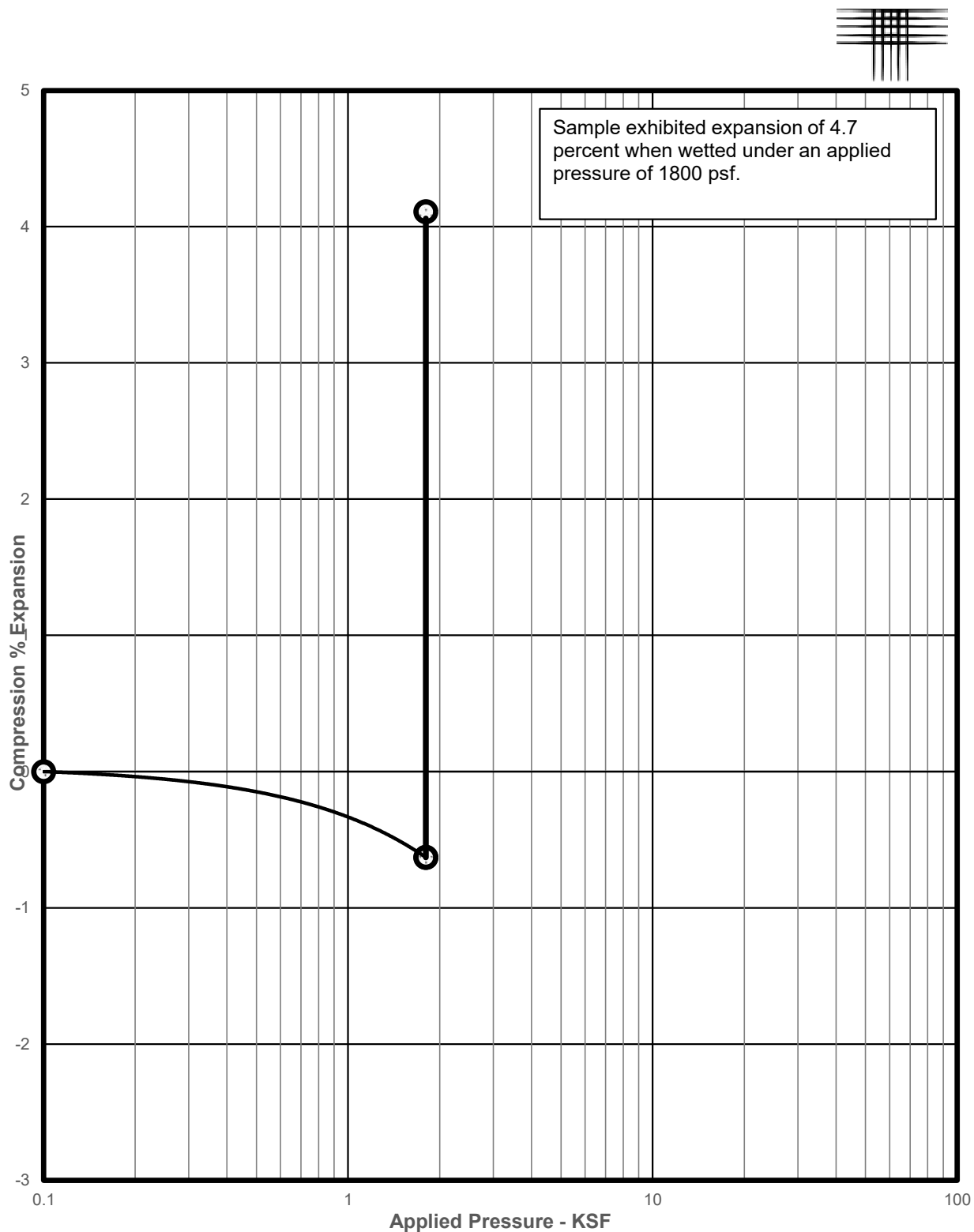


SAMPLE OF: CLAY, VERY SANDY (CL)
FROM: TH-72 AT 14 FEET

DRY UNIT WEIGHT: 114 pcf
MOISTURE CONTENT: 8.9 %

BMC INVESTMENTS
CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
Test Results** FIG. B- 92

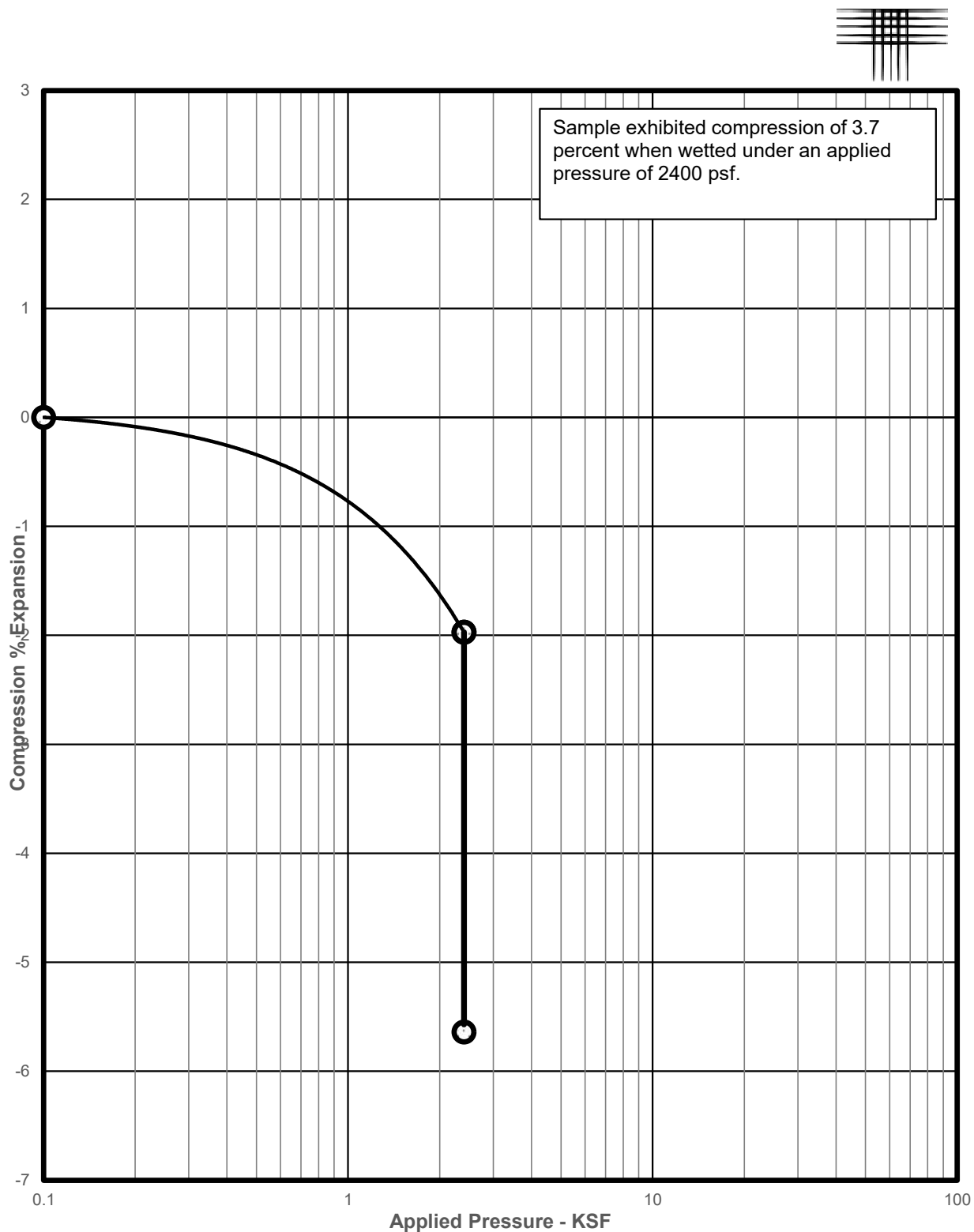


SAMPLE OF: CLAY, SANDY (CL)
 FROM: TH-75 AT 4 FEET

DRY UNIT WEIGHT: 118 pcf
 MOISTURE CONTENT: 11.8 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 93

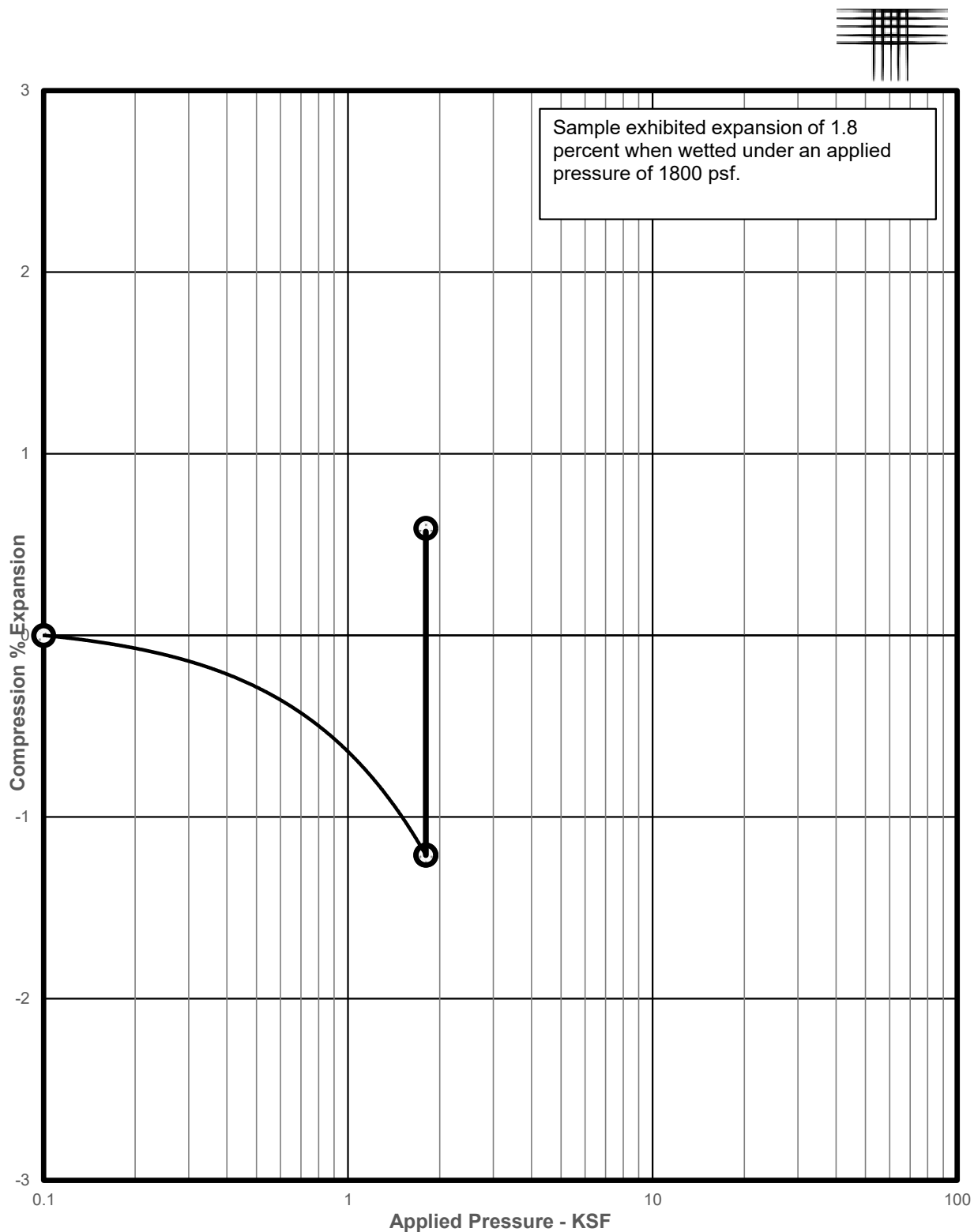


SAMPLE OF: CLAY, SANDY (CL)
 FROM: TH-75 AT 9 FEET

DRY UNIT WEIGHT: 97 pcf
 MOISTURE CONTENT: 11.0 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 94

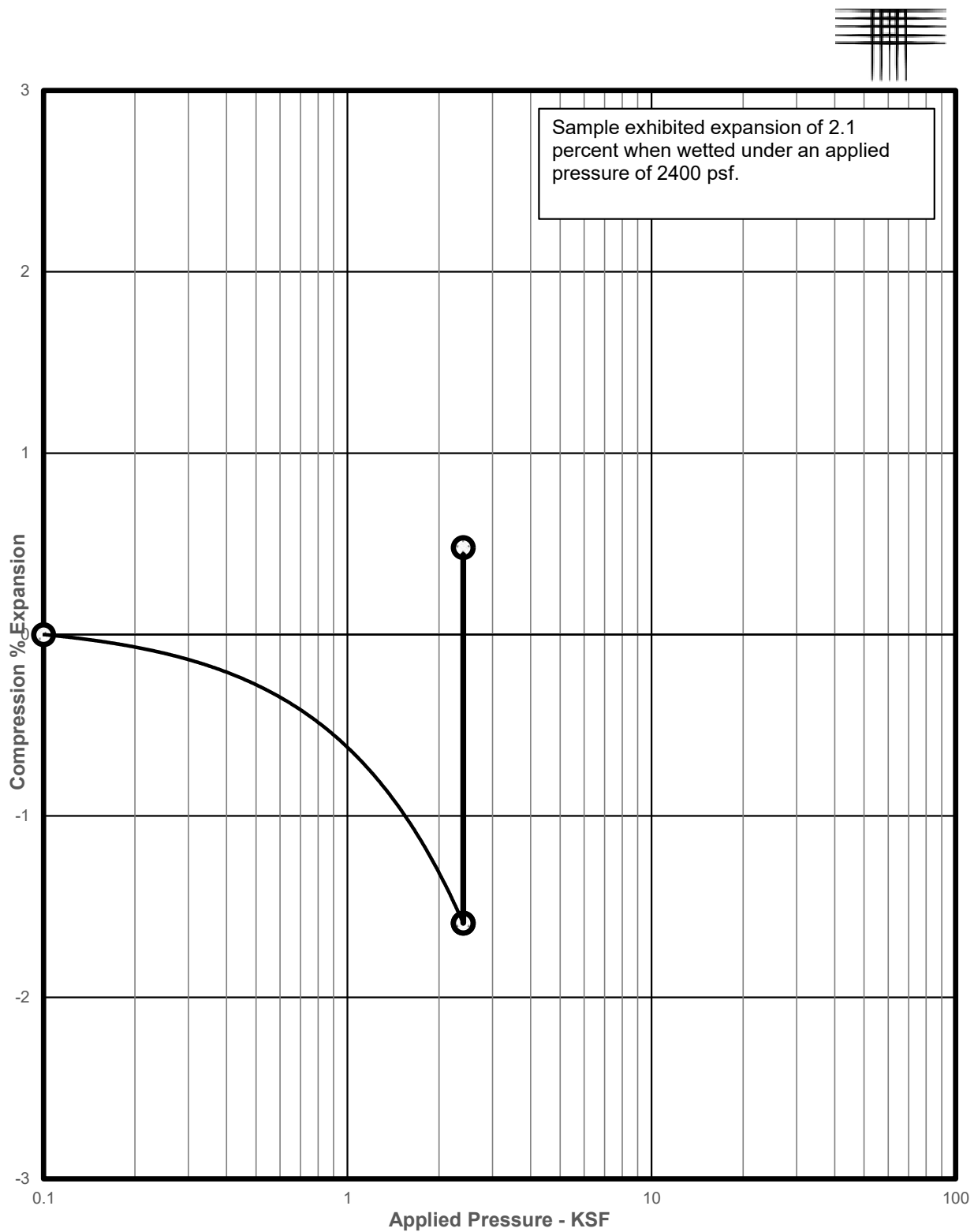


SAMPLE OF: CLAYSTONE
 FROM: TH-76 AT 4 FEET

DRY UNIT WEIGHT: 125 pcf
 MOISTURE CONTENT: 9.4 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 95

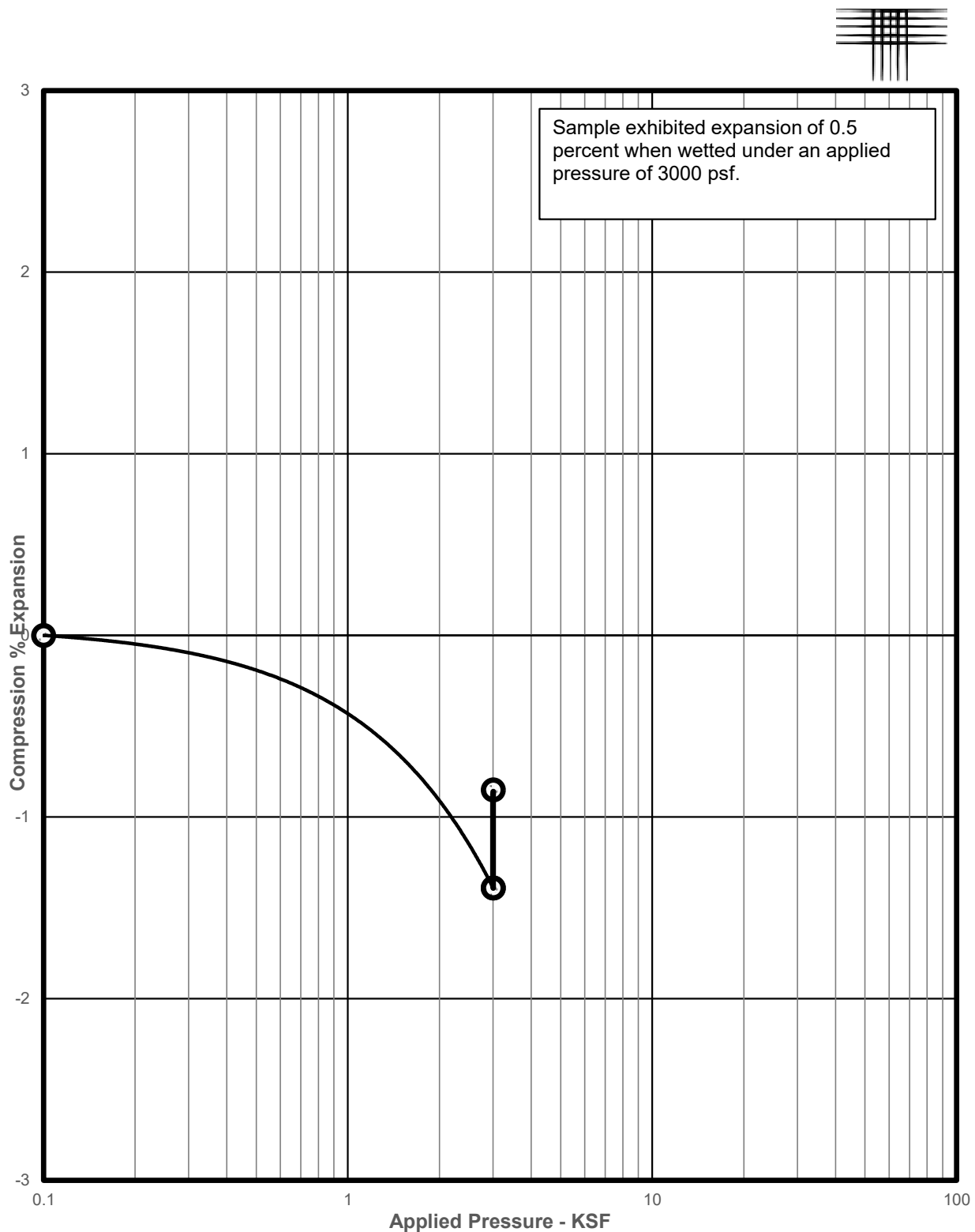


SAMPLE OF: CLAY, SANDY (CL)
FROM: TH-77 AT 4 FEET

DRY UNIT WEIGHT: 110 pcf
MOISTURE CONTENT: 8.3 %

BMC INVESTMENTS
CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
Test Results** FIG. B- 96

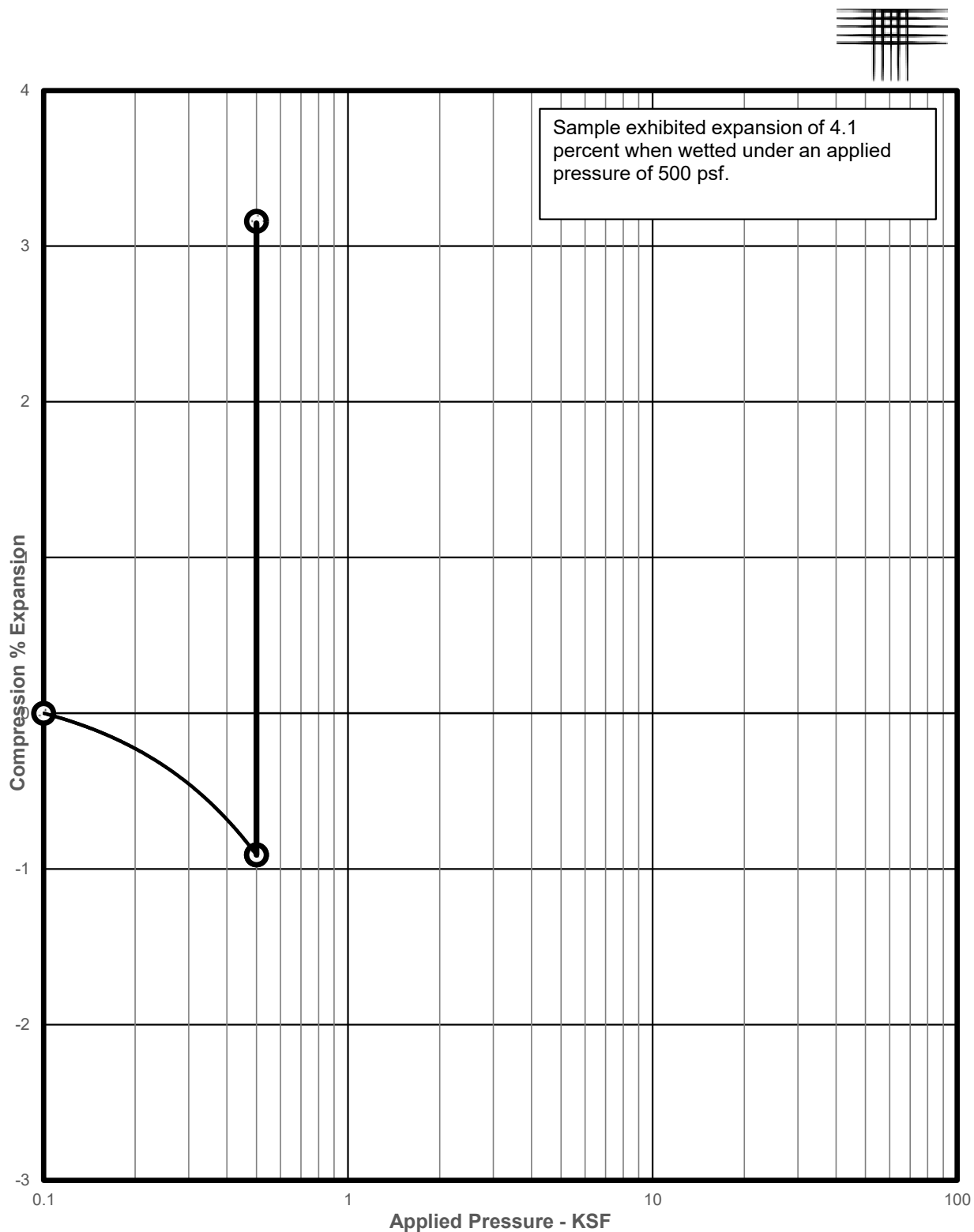


SAMPLE OF: CLAY, VERY SANDY (CL)
 FROM: TH-77 AT 9 FEET

DRY UNIT WEIGHT: 119 pcf
 MOISTURE CONTENT: 8.4 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 97

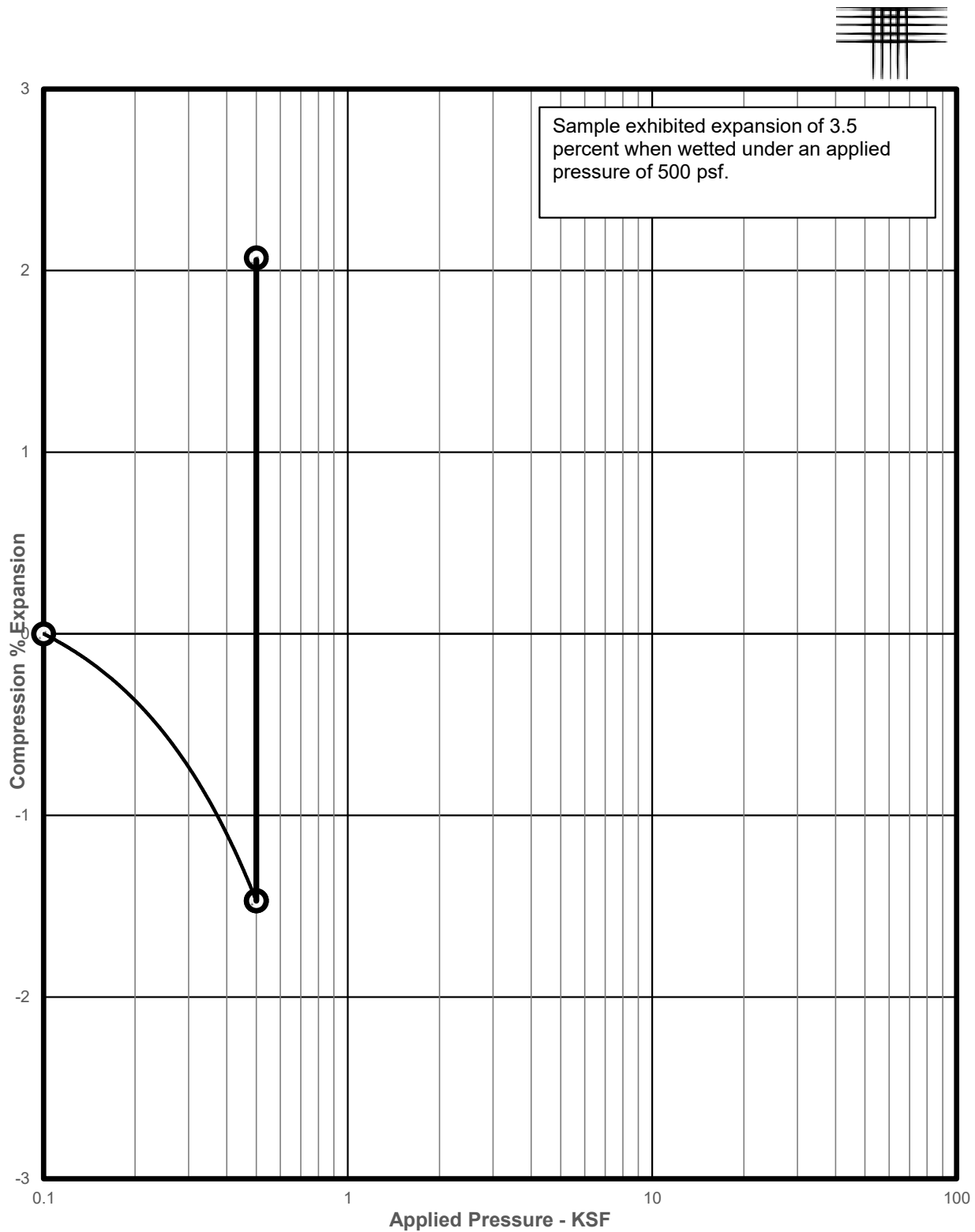


SAMPLE OF: CLAYSTONE
FROM: TH-81 AT 14 FEET

DRY UNIT WEIGHT: 106 pcf
MOISTURE CONTENT: 16.3 %

BMC INVESTMENTS
CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
Test Results** FIG. B- 98

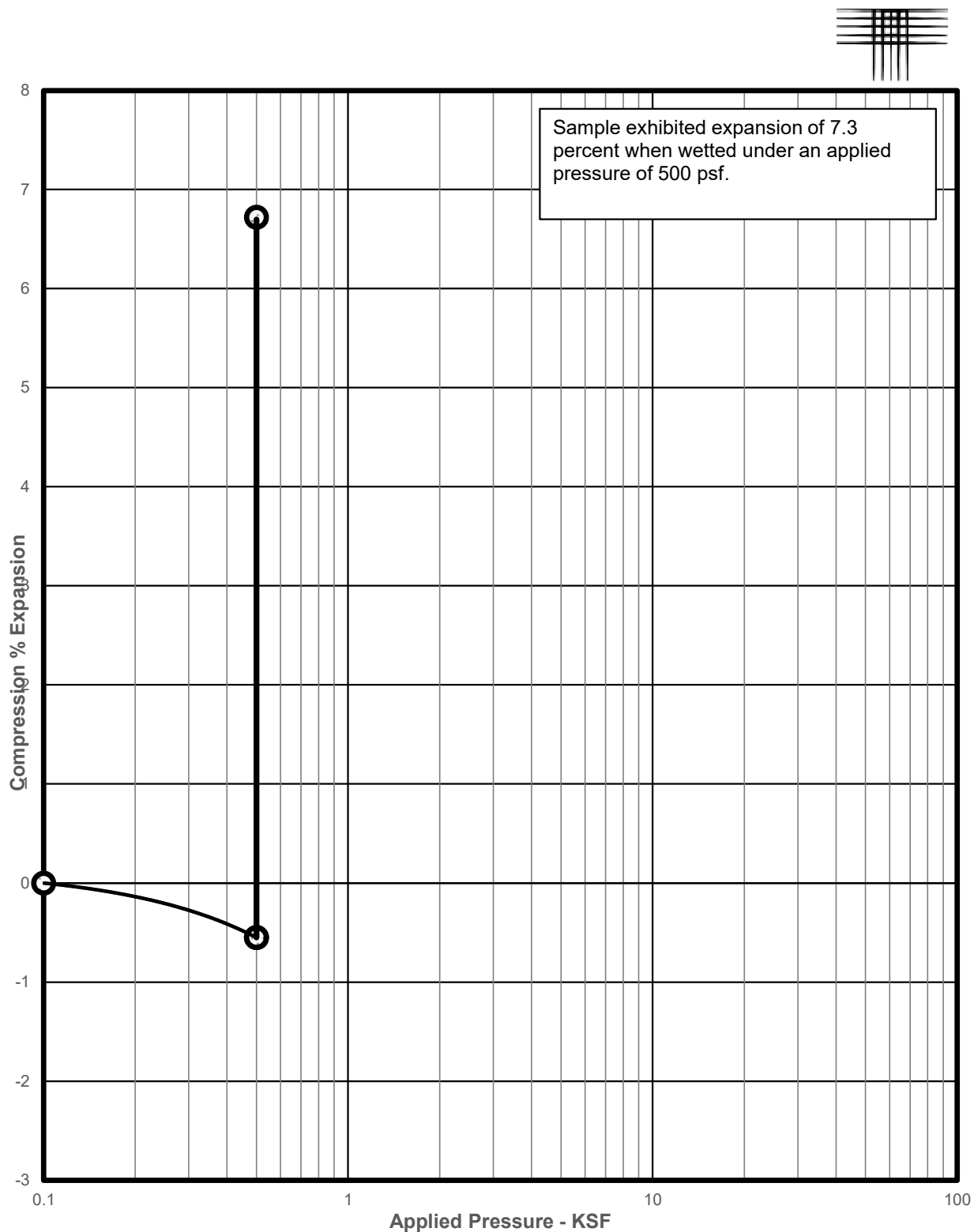


SAMPLE OF: CLAYSTONE
 FROM: TH-81 AT 19 FEET

DRY UNIT WEIGHT: 104 pcf
 MOISTURE CONTENT: 21.5 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 99

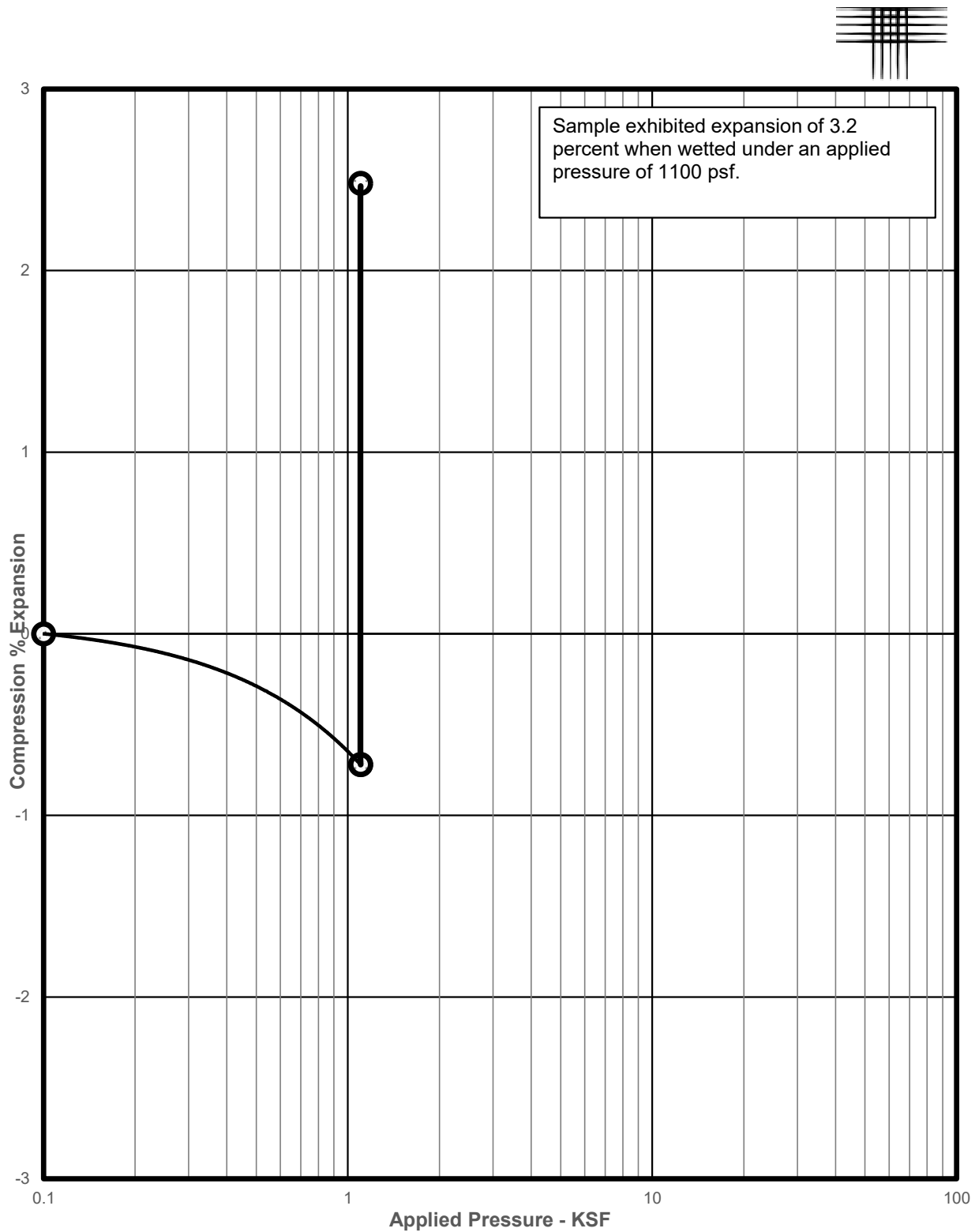


SAMPLE OF: CLAYSTONE
 FROM: S-1 AT 12 FEET

DRY UNIT WEIGHT: 119 pcf
 MOISTURE CONTENT: 12.8 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 100

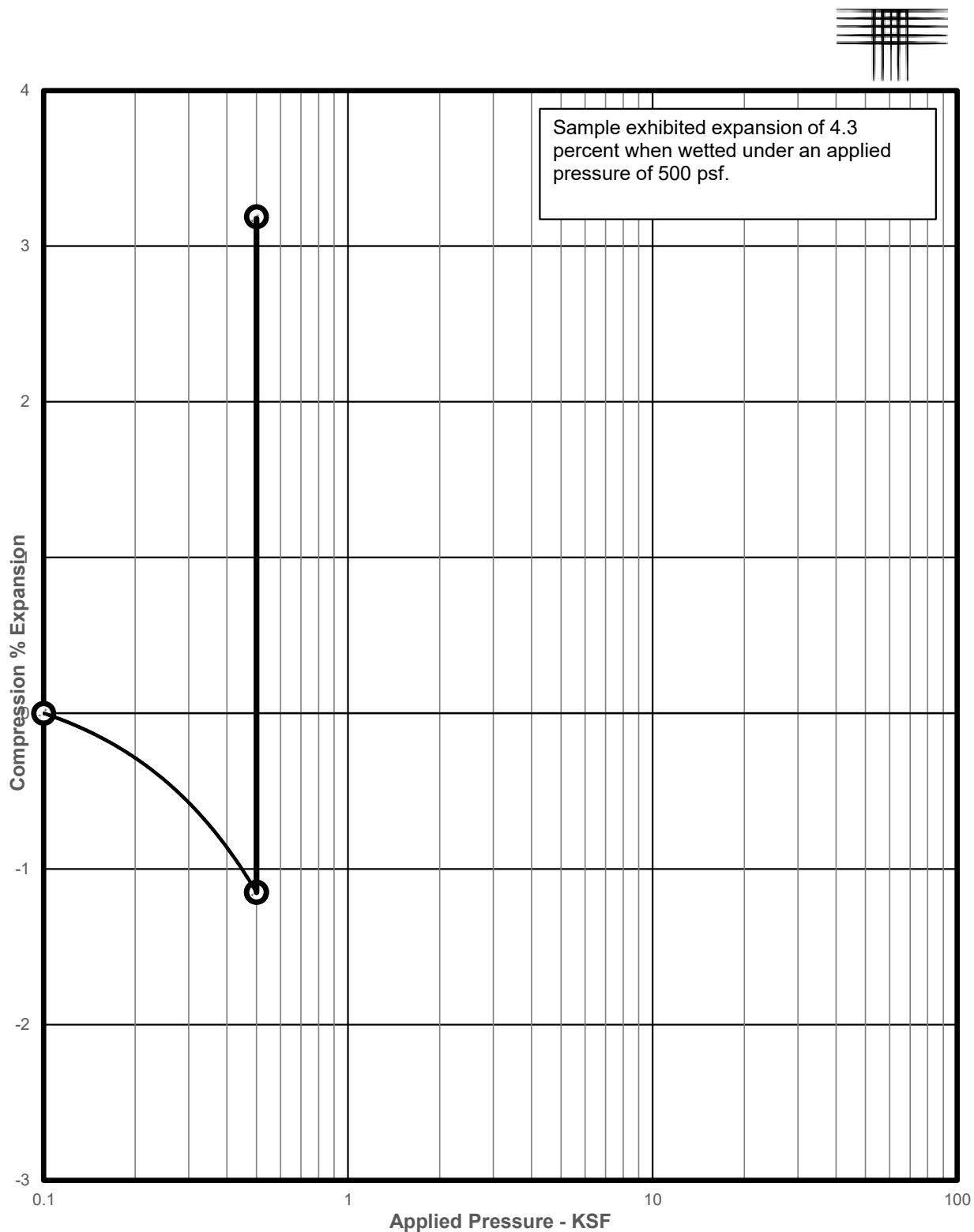


SAMPLE OF: CLAYSTONE
 FROM: S-1 AT 14 FEET

DRY UNIT WEIGHT: 111 pcf
 MOISTURE CONTENT: 15.3 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 101

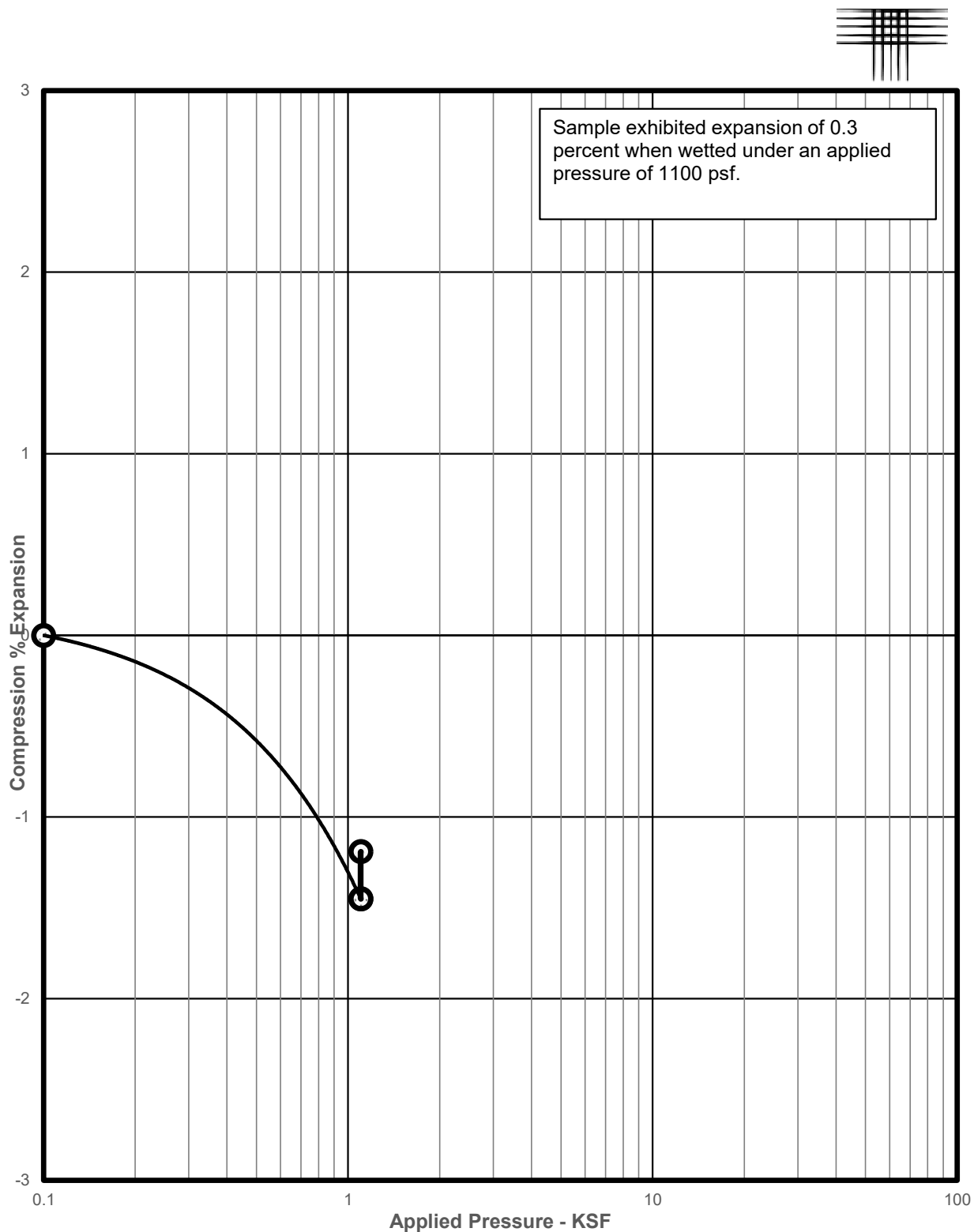


SAMPLE OF: CLAYSTONE
 FROM: S-2 AT 4 FEET

DRY UNIT WEIGHT: 115 pcf
 MOISTURE CONTENT: 11.8 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 102

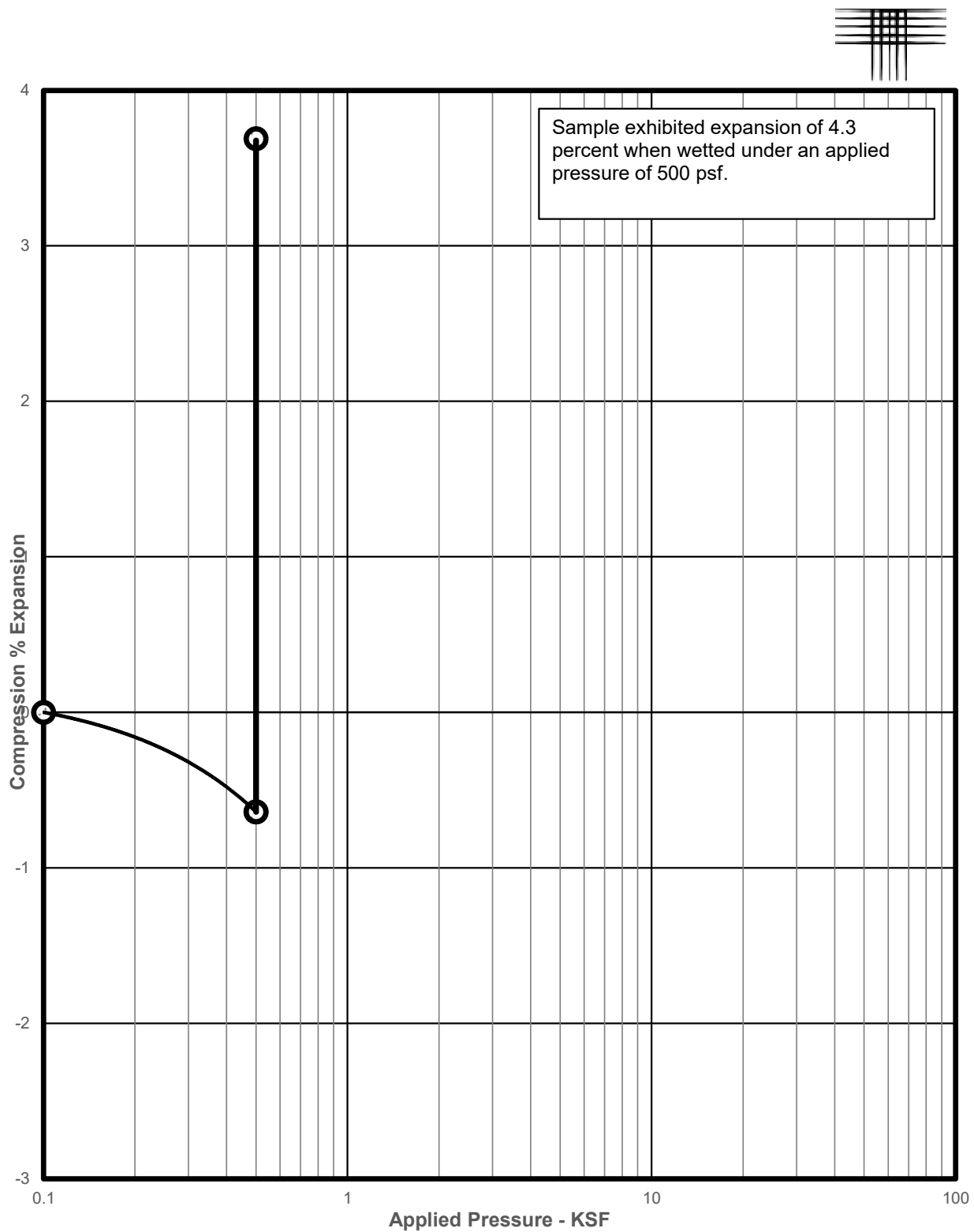


SAMPLE OF: CLAYSTONE
 FROM: S-2 AT 14 FEET

DRY UNIT WEIGHT: 105 pcf
 MOISTURE CONTENT: 15.2 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 103

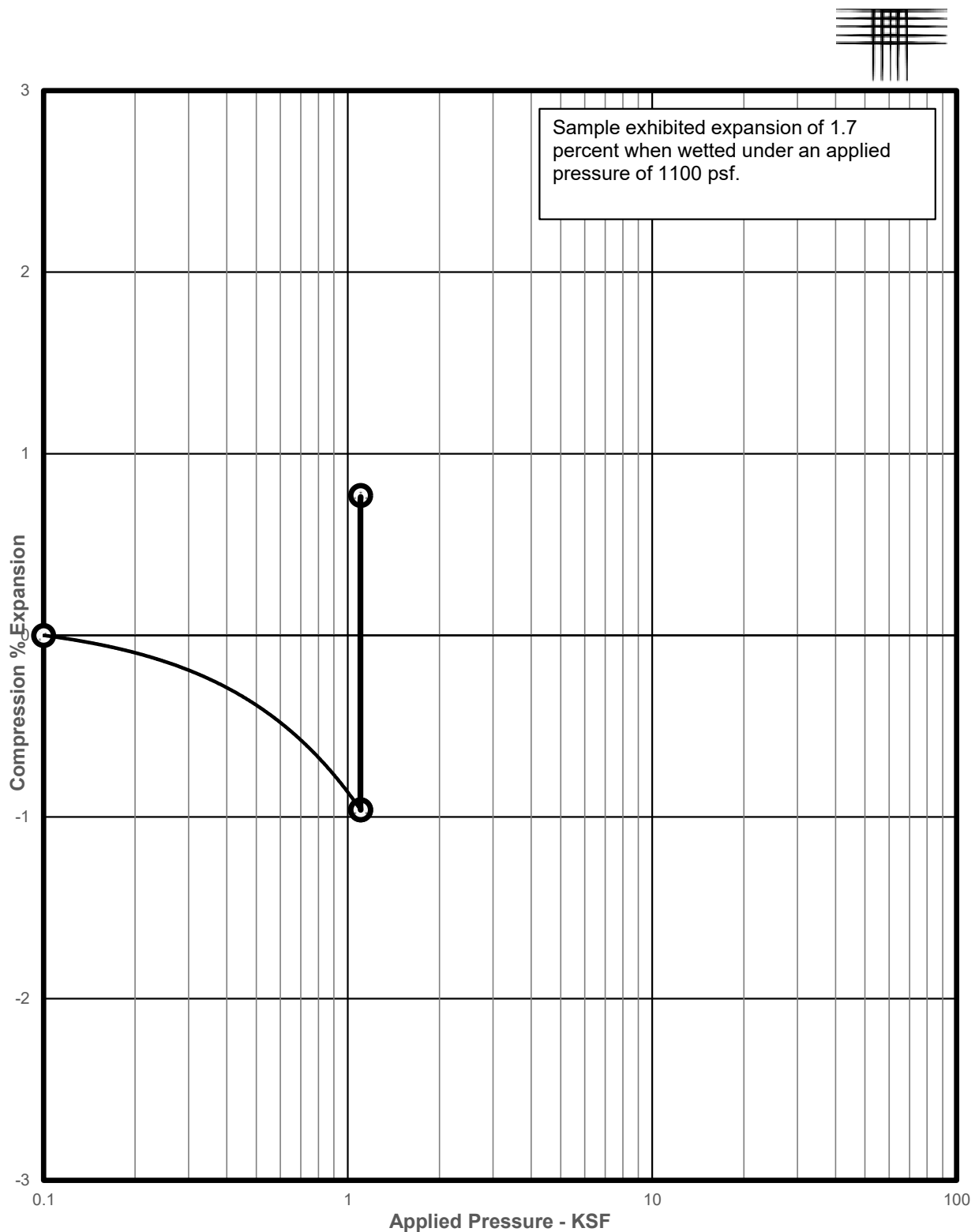


SAMPLE OF: CLAYSTONE
FROM: S-3 AT 19 FEET

DRY UNIT WEIGHT: 106 pcf
MOISTURE CONTENT: 19.6 %

BMC INVESTMENTS
CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
Test Results** FIG. B- 104

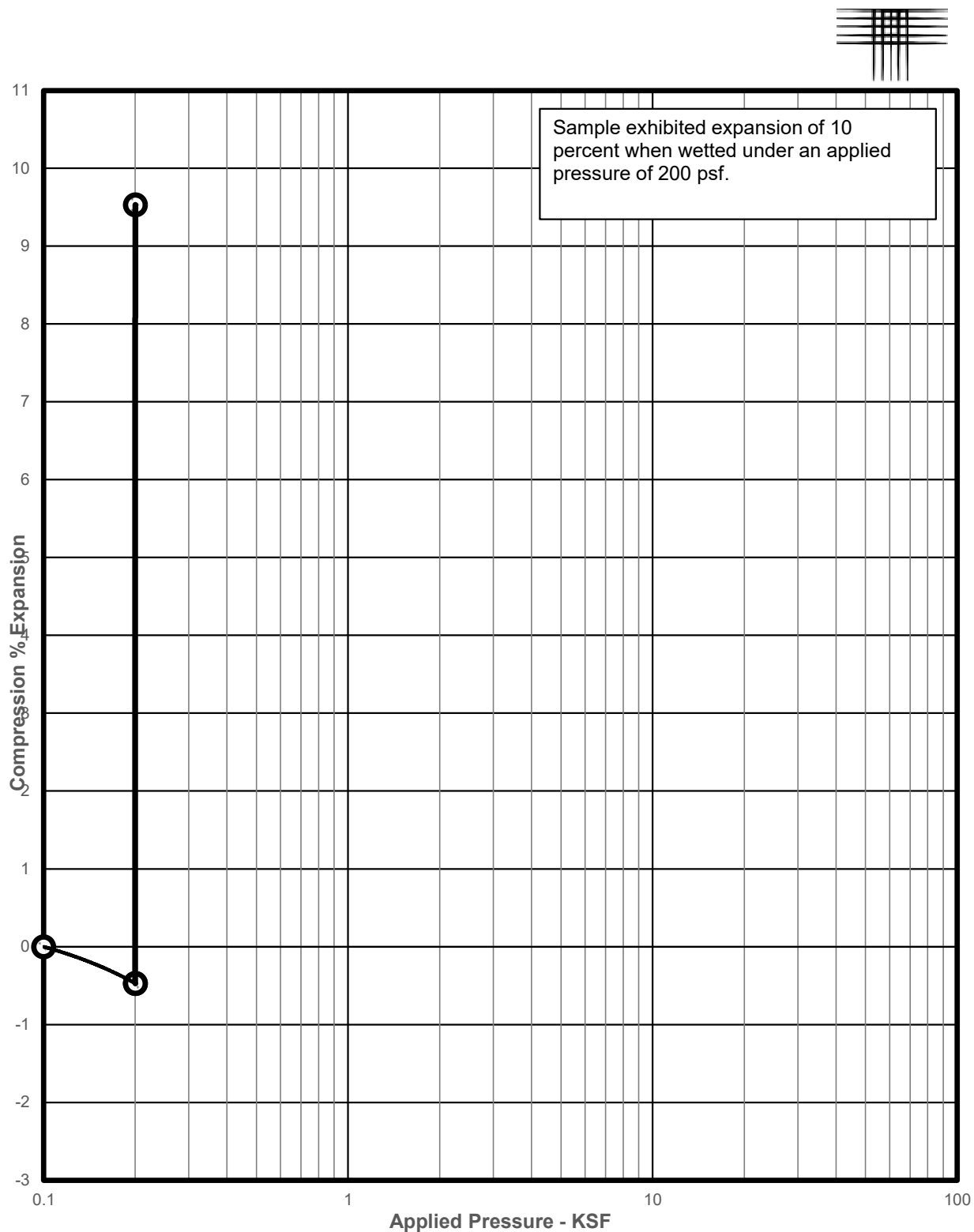


SAMPLE OF: CLAYSTONE
 FROM: S-4 AT 19 FEET

DRY UNIT WEIGHT: 115 pcf
 MOISTURE CONTENT: 14.7 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 105

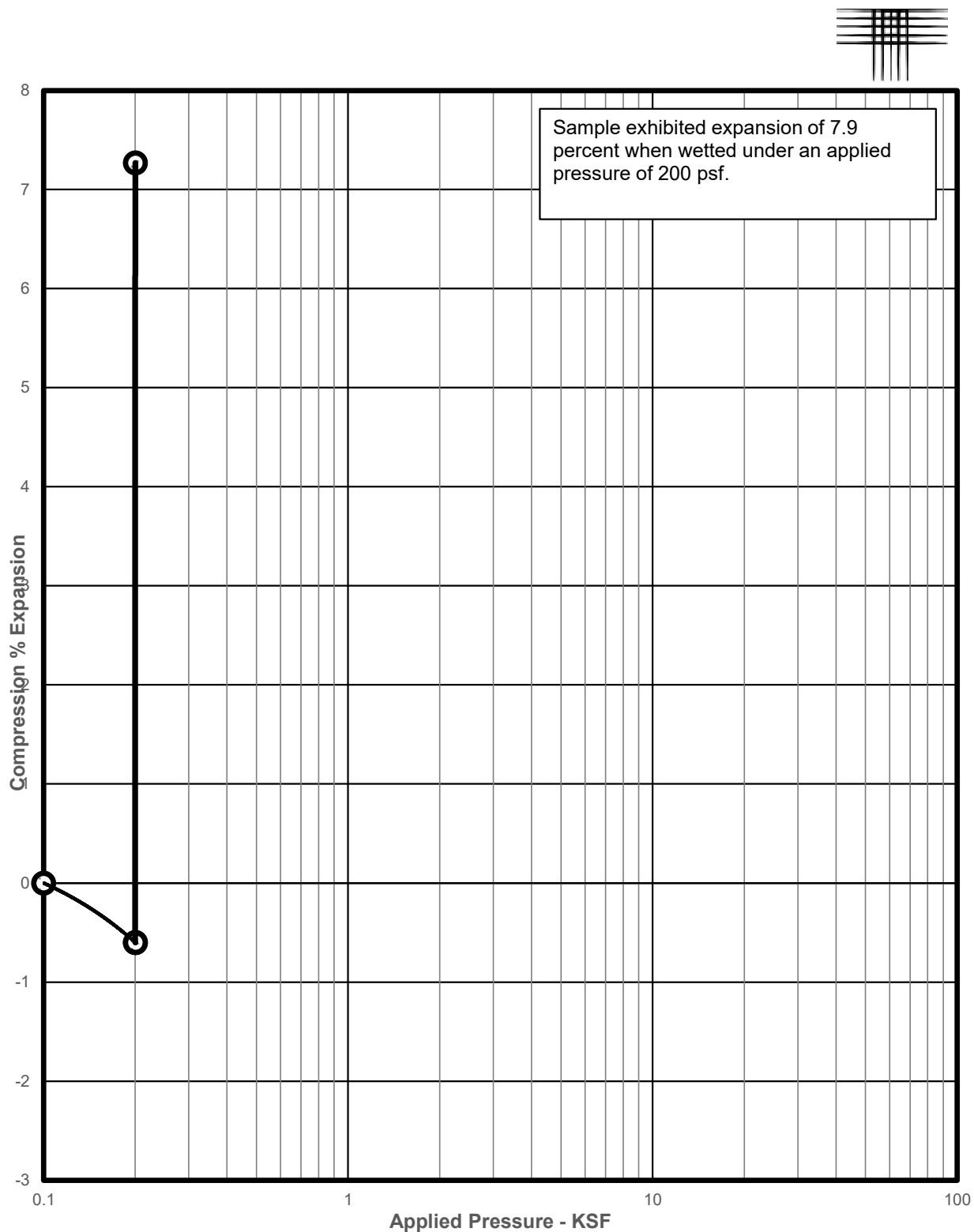


SAMPLE OF: CLAY, SANDY (CL)
 FROM: S-5 AT 4 FEET

DRY UNIT WEIGHT: 108 pcf
 MOISTURE CONTENT: 13.1 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 106

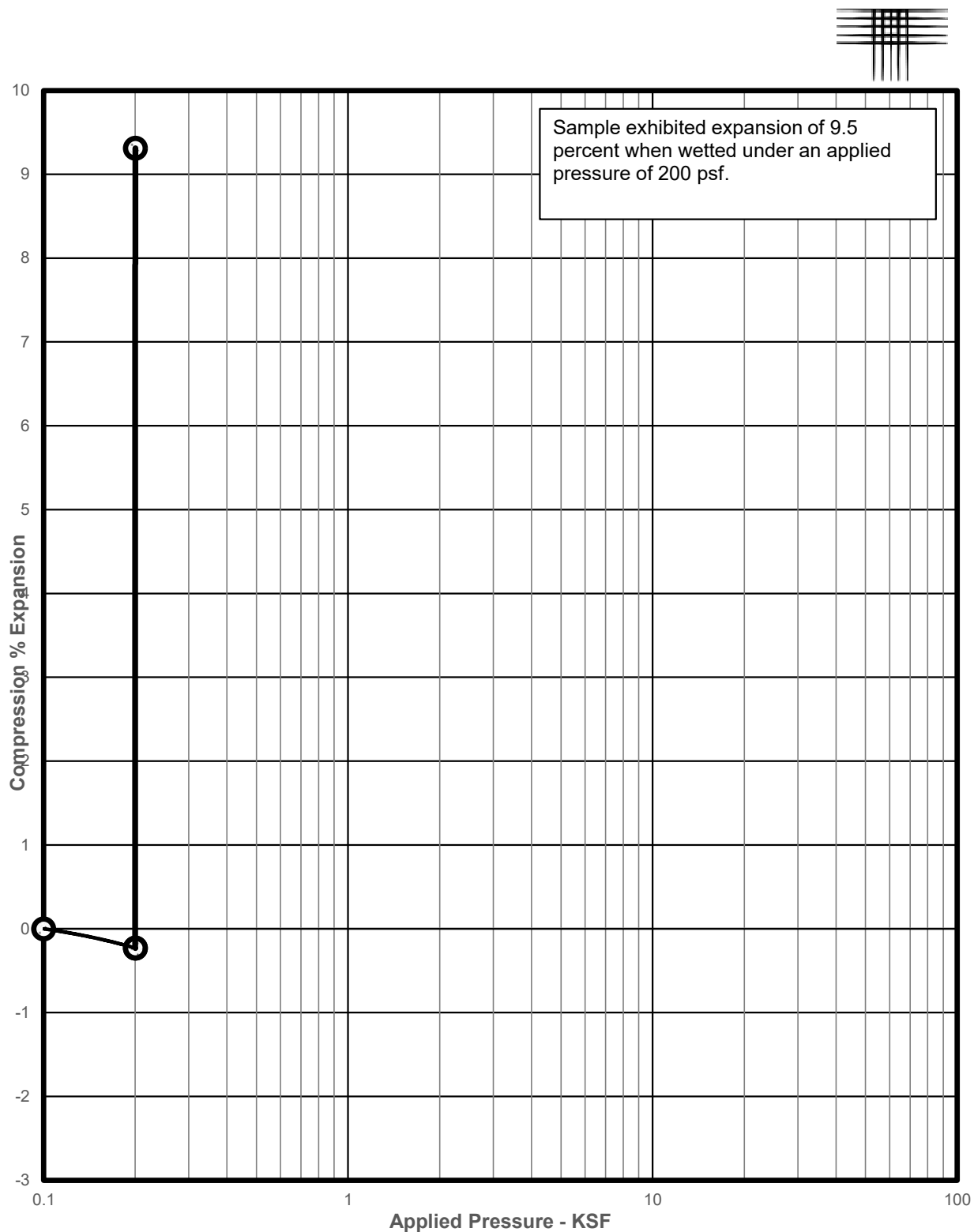


SAMPLE OF: CLAY, VERY SANDY (CL)
 FROM: S-5 AT 7 FEET

DRY UNIT WEIGHT: 93 pcf
 MOISTURE CONTENT: 10.9 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTL/T PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 107

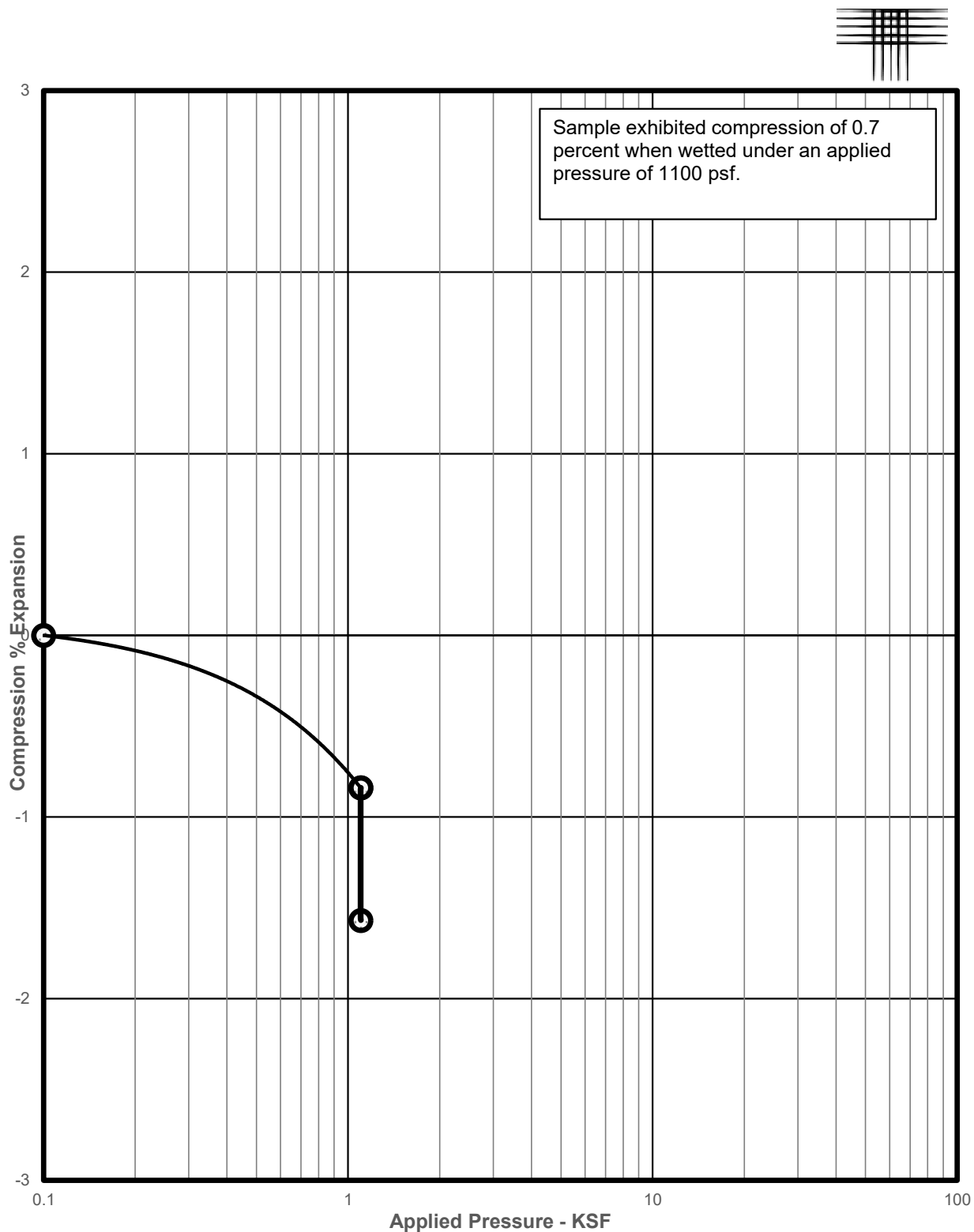


SAMPLE OF: CLAY, SANDY (CL)
 FROM: S-6 AT 4 FEET

DRY UNIT WEIGHT: 113 pcf
 MOISTURE CONTENT: 11.7 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 108

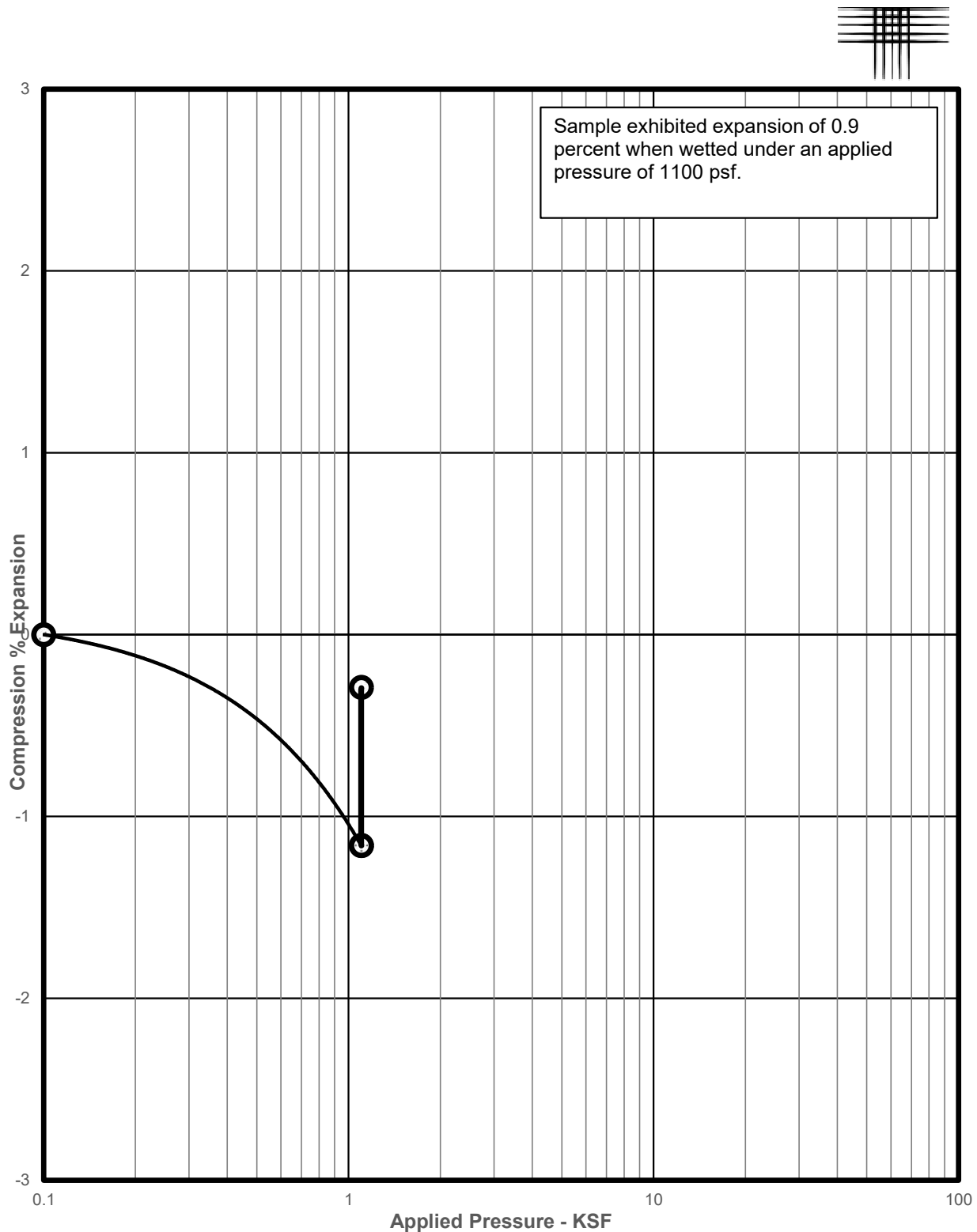


SAMPLE OF: CLAY, SANDY (CL)
 FROM: S-8 AT 2 FEET

DRY UNIT WEIGHT: 98 pcf
 MOISTURE CONTENT: 9.7 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 109

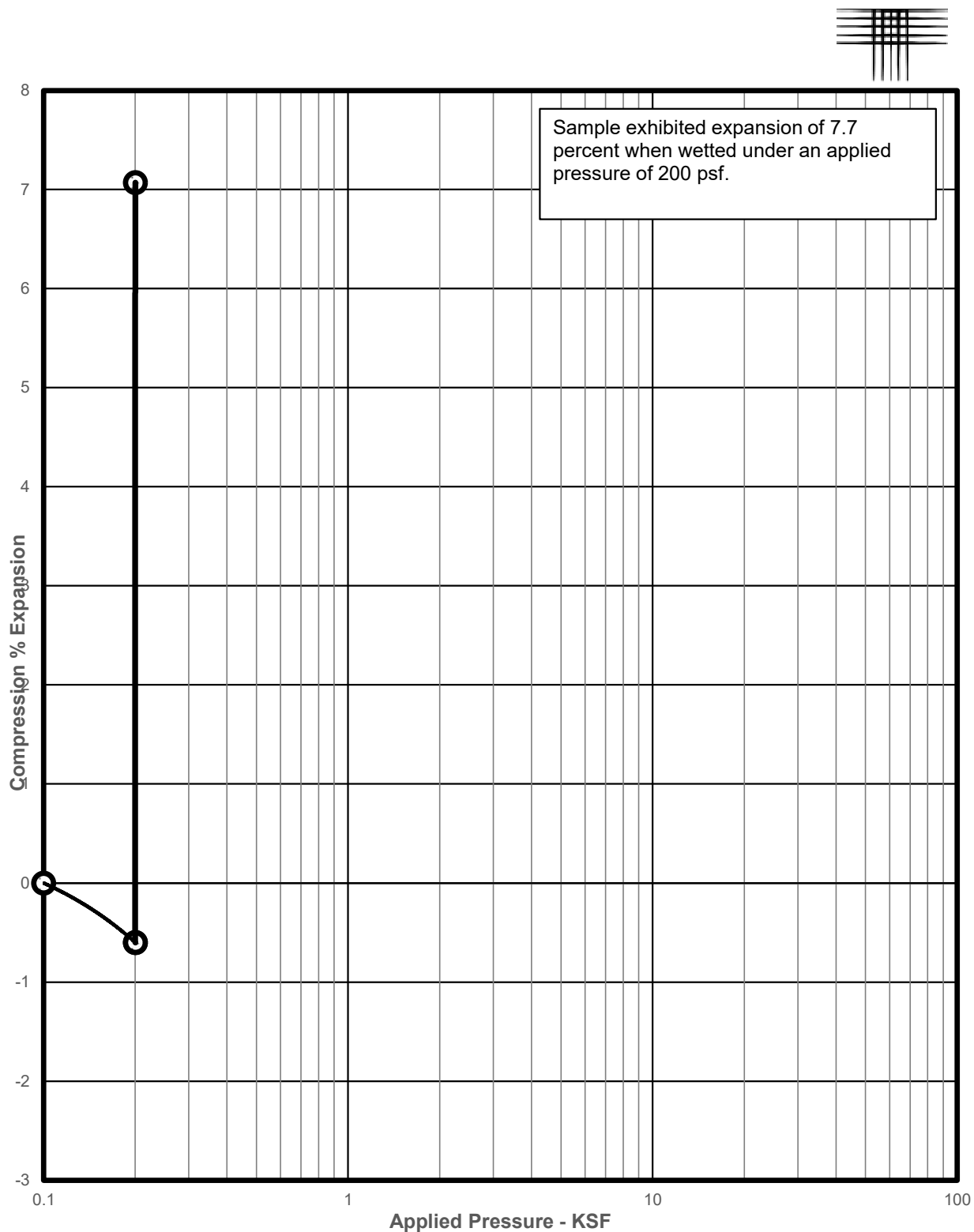


SAMPLE OF: CLAYSTONE
FROM: S-10 AT 9 FEET

DRY UNIT WEIGHT: 112 pcf
MOISTURE CONTENT: 12.5 %

BMC INVESTMENTS
CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
Test Results** FIG. B- 110

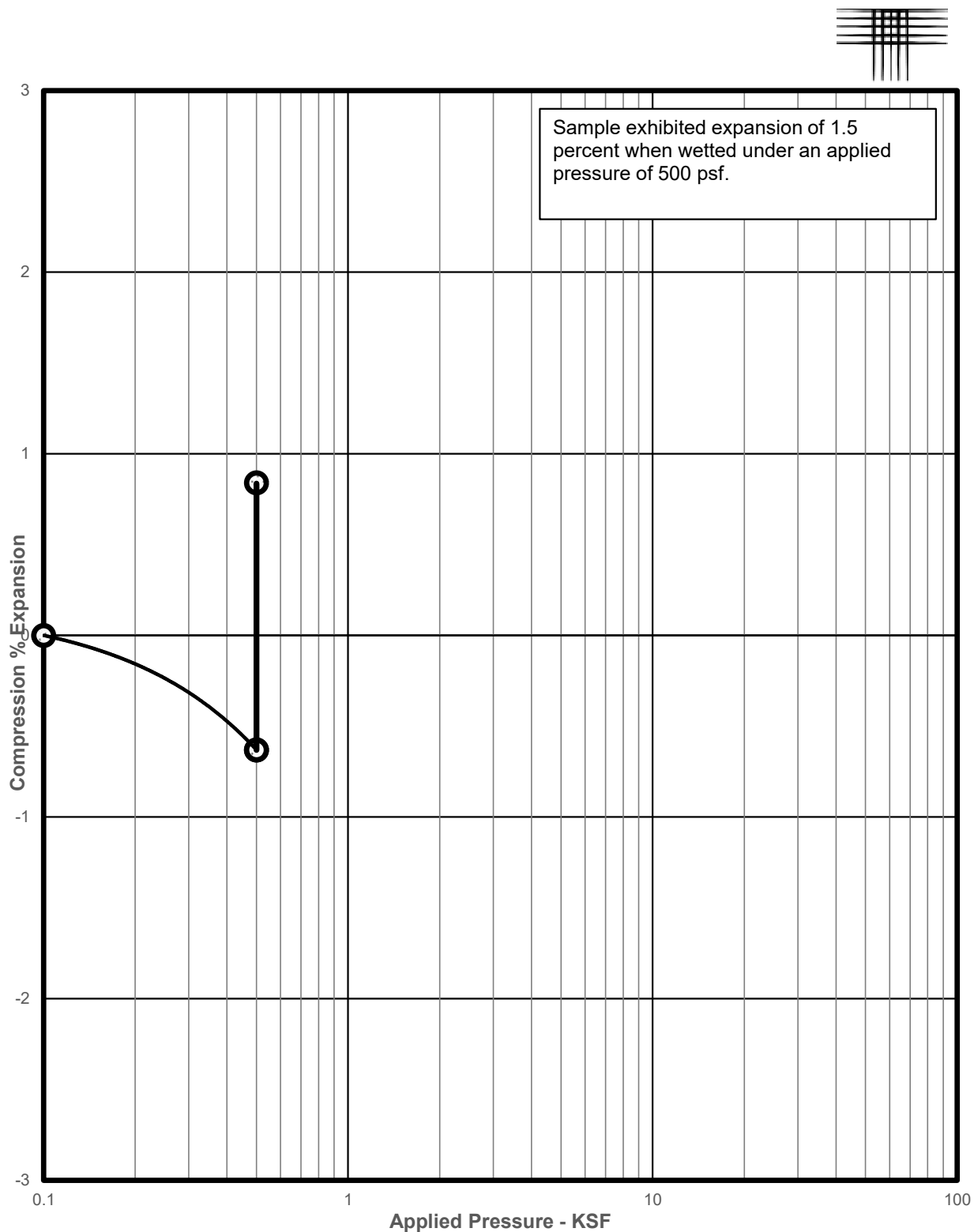


SAMPLE OF: CLAY, SANDY (CL)
 FROM: S-11 AT 2 FEET

DRY UNIT WEIGHT: 101 pcf
 MOISTURE CONTENT: 12.6 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 111



SAMPLE OF: CLAY, VERY SANDY (CL)
 FROM: S-11 AT 4 FEET

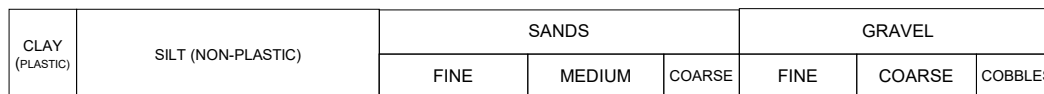
DRY UNIT WEIGHT: 96 pcf
 MOISTURE CONTENT: 8.6 %

BMC INVESTMENTS
 CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
 CTLJT PROJECT NO. DN51,859-125-R1

**Swell Consolidation
 Test Results** FIG. B- 112

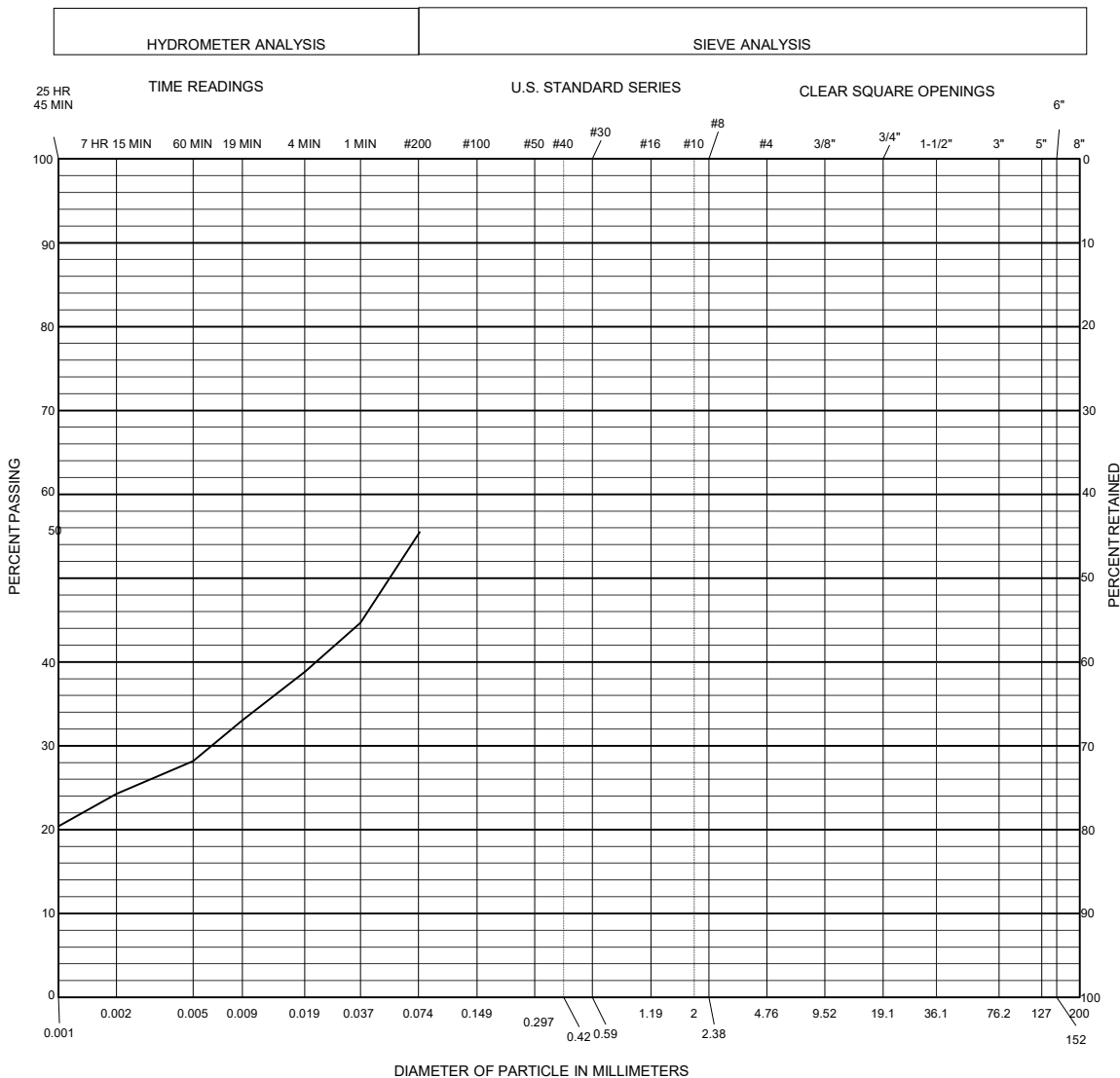
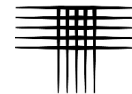


6"



Size (mm)	-	-	-	-	-	-	-	-	-	-	-	0.075	0.037	0.019	0.009	0.005	0.002	0.001
% Passing	-	-	-	-	-	-	-	-	-	-	-	46.9	38	32	29	25	20	15

FIG. B-113



CLAY (PLASTIC)	SILT (NON-PLASTIC)	SANDS			GRAVEL			
		FINE	MEDIUM	COARSE	FINE	COARSE	COBBLES	
Sample of: CLAY, VERY SANDY (CL)		Sand & Gravel: 44.5 %			Silt: 31.2 %			
From: S-11 AT 0 FEET		Clay: 24.3 %			Liquid Limit: 34.0 %			
		Plasticity Index: 17.0 %						

Size (mm)	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.075	0.037	0.019	0.009	0.005	0.002	0.001
% Passing	-	-	-	-	-	-	-	-	-	-	-	-	-	-	55.5	45	39	33	28	24	20

BMC INVESTMENTS
CANYONSIDE - SINGLE AND MULTI-FAMILY DEVELOPMENT
CTLJT PROJECT NO. DN51,859-125-R1

Hydrometer Test Results

FIG. B-114

TABLE B - I
SUMMARY OF LABORATORY TEST RESULTS



BORING	DEPTH (ft)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	SWELL TEST DATA				SOIL SUCTION VALUE (pF)	ATTERBERG LIMITS		SOLUBLE SULFATE CONTENT (%)	PASSING NO. 200 SIEVE (%)	SOIL TYPE
				SWELL (%)	COMPRESSION (%)	APPLIED PRESSURE (psf)	SWELL PRESSURE (psf)		LIQUID LIMIT	PLASTICITY INDEX			
TH-1	9	13.3	105	1.6		500							CLAYSTONE
TH-1	19	10.0	110									15	SANDSTONE
TH-1	29	10.7	126									40	SANDSTONE
TH-2	4	7.3	109									19	WEATHERED SANDSTONE
TH-2	19	9.4	112									44	SANDSTONE
TH-3	9	17.1	106	5.7		500					0.04		CLAYSTONE
TH-3	14	14.0	116	2.2		1,100							CLAYSTONE
TH-3	24	14.0	109									23	SANDSTONE
TH-4	9	7.1	122									35	SANDSTONE
TH-4	24	17.3	111	1.7		2,400							CLAYSTONE
TH-5	9	12.8	94	2.5		500							CLAYSTONE
TH-5	14	12.2	104									48	SANDSTONE
TH-5	19	8.9	119									30	SANDSTONE
TH-6	14	10.6	115	0.2		500							CLAYSTONE
TH-6	19	8.7	110									19	SANDSTONE
TH-7	9	15.5	106	3.5		500							CLAYSTONE
TH-7	24	11.0	105									38	SANDSTONE
TH-8	14	14.0	101	8.9		500							CLAYSTONE
TH-8	19	9.7	104									28	SANDSTONE
TH-9	9	11.3	117	3.9		500							CLAYSTONE
TH-9	14	16.8	104	1.7		500							CLAYSTONE
TH-9	19	16.8	95		0.3	1,100							CLAYSTONE
TH-10	9	15.9	107	1.8		500							CLAYSTONE
TH-10	14	14.9	102		0.3	500							CLAYSTONE
TH-10	29	14.5	111	1.9		1,800							CLAYSTONE
TH-11	14	13.2	105									31	SANDSTONE
TH-11	19	14.0	111	1.8		1,100							CLAYSTONE
TH-12	14	17.8	104	2.7		500							CLAYSTONE
TH-12	24	13.2	111									18	SANDSTONE
TH-12	29	10.8	116									28	SANDSTONE
TH-13	19	11.3	110									21	SANDSTONE
TH-13	24	15.6	118	4.9		1,100							CLAYSTONE
TH-13	34	18.9	106	1.1		2,400							CLAYSTONE
TH-13	39	14.9	112	0.8		3,000							CLAYSTONE
TH-14	14	10.5	118									30	SANDSTONE
TH-14	34	16.4	118	1.6		3,000							CLAYSTONE
TH-15	9	4.2	109		1.3	500		4.73					SAND, CLAYEY, GRAVELLY (SC)
TH-15	14	11.0	116	2.7		500	5,600	3.97					CLAY, SANDY (CL)
TH-15	19	7.8	107		0.6	1,100		3.98					WEATHERED SANDSTONE
TH-15	24	17.3	98		1.8	1,800		4.21					CLAYSTONE
TH-15	29	11.2	115									21	SANDSTONE
TH-15	34	15.1	114	1.3		3,000	8,900	4.44					CLAYSTONE
TH-15	39	14.7	115	1.0		3,600	10,200	4.24					CLAYSTONE
TH-16	14	21.1	101	2.3		1,100			54	31		92	CLAYSTONE
TH-16	19	24.4	100	1.7		1,800							CLAYSTONE
TH-17	9	13.1	116									48	SANDSTONE
TH-17	19	11.8	119									28	SANDSTONE
TH-18	9	4.0	117									15	SANDSTONE
TH-19	4	8.0	126									21	SANDSTONE
TH-19	9	12.2	114	2.3		1,100							CLAYSTONE

TABLE B - I
SUMMARY OF LABORATORY TEST RESULTS



BORING	DEPTH (ft)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	SWELL TEST DATA				SOIL SUCTION VALUE (pF)	ATTERBERG LIMITS		SOLUBLE SULFATE CONTENT (%)	PASSING NO. 200 SIEVE (%)	SOIL TYPE
				SWELL (%)	COMPRESSION (%)	APPLIED PRESSURE (psf)	SWELL PRESSURE (psf)		LIQUID LIMIT	PLASTICITY INDEX			
TH-19	14	10.9	120									37	SANDSTONE
TH-20	4	5.9	117									16	SANDSTONE
TH-20	14	8.7	117									29	SANDSTONE
TH-21	9	7.7	131									40	SANDSTONE
TH-21	19	14.2	108									16	SANDSTONE
TH-21	29	21.4	101		0.1	3,000							CLAYSTONE
TH-22	9	4.8	119									21	SAND, SILTY (SM)
TH-22	14	11.9	115	4.5		1,100							CLAY, SANDY (CL)
TH-22	24	13.0	111									19	SANDSTONE
TH-23	4	6.7	90		2.3	500					<0.01		SAND, CLAYEY (SC)
TH-23	9	6.8	116	0.5		1,100							SAND, CLAYEY (SC)
TH-23	14	14.0	115	3.7		1,800							CLAY, SANDY (CL)
TH-23	19	6.4	117									21	SAND, SILTY (SM)
TH-24	9	2.1	99									12	SAND, SILTY (SM)
TH-24	14	9.8	108		0.3	1,100							SAND, CLAYEY (SC)
TH-24	24	9.7	119									24	SANDSTONE
TH-25	4	5.0	101		1.9	500							SAND, CLAYEY (SC)
TH-25	9	9.2	120	4.4		1,100							CLAY, SANDY (CL)
TH-25	14	7.9	115									24	SANDSTONE
TH-26	4	17.7	105	7.2		500							WEATHERED CLAYSTONE
TH-26	9	12.1	109	1.3		1,100							CLAYSTONE
TH-26	14	11.5	91									29	SANDSTONE
TH-27	14	15.5	115	2.4		500							CLAYSTONE
TH-27	19	21.3	96	0.3		1,100							CLAYSTONE
TH-27	24	12.3	109									27	SANDSTONE
TH-28	14	18.2	109	6.1		500							CLAYSTONE
TH-28	24	13.7	117	1.1		1,100							CLAYSTONE
TH-28	29	11.2	106									24	SANDSTONE
TH-29	9	10.9	118									36	SANDSTONE
TH-29	19	11.1	115									24	SANDSTONE
TH-30	14	14.0	113									20	SANDSTONE
TH-30	19	15.6	109	1.3		1,100							CLAYSTONE
TH-30	29	17.5	105									23	SANDSTONE
TH-31	19	18.6	109	4.7		500							CLAYSTONE
TH-31	24	11.9	113	1.7		500							CLAYSTONE
TH-31	29	9.9	109									29	SANDSTONE
TH-32	24	14.6	111									25	SANDSTONE
TH-32	29	10.4	118	1.5		500							CLAYSTONE
TH-32	34	12.3	113									25	SANDSTONE
TH-33	19	18.3	107	2.4		500							CLAYSTONE
TH-33	24	12.1	118	1.2		500							CLAYSTONE
TH-33	29	12.8	117									36	SANDSTONE
TH-34	19	16.7	104	1.3		500							CLAYSTONE
TH-34	24	12.0	118									33	SANDSTONE
TH-35	19	13.7	114						37	21		46	SANDSTONE
TH-35	24	17.6	109									26	SANDSTONE
TH-35	39	15.5	111									20	SANDSTONE
TH-36	14	17.0	108	2.6		500							CLAYSTONE
TH-36	19	10.5	110									24	SANDSTONE
TH-37	14	12.8	116									19	SANDSTONE

TABLE B - I
SUMMARY OF LABORATORY TEST RESULTS



BORING	DEPTH (ft)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	SWELL TEST DATA				SOIL SUCTION VALUE (pF)	ATTERBERG LIMITS		SOLUBLE SULFATE CONTENT (%)	PASSING NO. 200 SIEVE (%)	SOIL TYPE
				SWELL (%)	COMPRESSION (%)	APPLIED PRESSURE (psf)	SWELL PRESSURE (psf)		LIQUID LIMIT	PLASTICITY INDEX			
TH-37	19	12.9	118						38	21		16	SANDSTONE
TH-37	29	10.1	113									24	SANDSTONE
TH-38	14	10.3	116									25	SANDSTONE
TH-38	24	14.0	118						37	23		50	INTERBEDDED CLAYSTONE/SANDSTONE
TH-39	9	13.7	109	4.7		500							CLAY, SANDY (CL)
TH-39	19	11.5	114	0.4		1,100							SANDSTONE
TH-39	24	13.0	118									19	SANDSTONE
TH-40	14	9.9	114									19	SANDSTONE
TH-40	19	12.2	113									16	SANDSTONE
TH-41	14	8.9	121									22	SANDSTONE
TH-41	24	15.3	105									22	SANDSTONE
TH-41	29	16.2	107	0.2		2,400							CLAYSTONE
TH-42	14	15.0	111	4.1		500							CLAYSTONE
TH-42	29	11.8	107									19	SANDSTONE
TH-43	14	18.2	109	4.5		500							CLAYSTONE
TH-43	19	12.0	117									35	SANDSTONE
TH-43	34	24.9	101	2.6		3,000							CLAYSTONE
TH-44	4	9.3	118									26	SANDSTONE
TH-44	14	11.3	123									47	SANDSTONE
TH-45	4	8.8	110		1.5	2,400							SAND, CLAYEY (SC)
TH-45	9	14.9	98		0.9	3,000			40	19		61	CLAYSTONE
TH-46	4	9.6	120	2.6		1,800							CLAYSTONE
TH-46	9	17.1	104	0.8		2,400							CLAYSTONE
TH-47	4	6.1	113									41	SAND, CLAYEY (SC)
TH-47	14	12.0	118	1.5		1,800							CLAYSTONE
TH-47	19	20.6	107	2.1		2,400							CLAYSTONE
TH-48	4	10.2	80								<0.01	66	CLAY, SANDY (CL)
TH-48	14	21.9	98	2.1		1,800							CLAYSTONE
TH-49	9	9.1	109									17	SANDSTONE
TH-49	24	13.1	116									38	SANDSTONE
TH-50	4	9.3	99						42	27		58	CLAY, VERY SANDY (CL)
TH-50	9	10.3	114	2.7		1,100							CLAYSTONE
TH-50	14	17.0	109	0.8		1,800							CLAYSTONE
TH-50	19	15.3	111		0.2	2,400							CLAYSTONE
TH-51	4	7.8	89		1.3	500							SAND, CLAYEY (SC)
TH-51	9	10.9	119									28	SAND, CLAYEY (SC)
TH-51	24	19.8	104	0.7		3,000							CLAYSTONE
TH-52	9	10.6	116									10	SANDSTONE
TH-52	19	18.4	112	1.6		3,000							CLAYSTONE
TH-53	4	12.0	103	0.7		1,800							CLAY, SANDY (CL)
TH-53	14	8.9	117									15	SANDSTONE
TH-53	19	15.1	109	0.7		3,600							CLAYSTONE
TH-54	4	4.8	113									26	SAND, SILTY (SM)
TH-54	14	9.9	118									33	SAND, CLAYEY (SC)
TH-55	4	9.6	103	5.6		500							CLAY, VERY SANDY (CL)
TH-55	9	7.4	118									26	SAND, CLAYEY (SC)
TH-55	14	13.4	106									10	SANDSTONE
TH-55	24	13.7	110		0.5	3,000							CLAYSTONE
TH-56	4	6.2	108		0.1	1,100							SAND, CLAYEY (SC)
TH-56	9	12.0	110	2.5		1,800							CLAY, VERY SANDY (CL)

TABLE B - I
SUMMARY OF LABORATORY TEST RESULTS



BORING	DEPTH (ft)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	SWELL TEST DATA				SOIL SUCTION VALUE (pF)	ATTERBERG LIMITS		SOLUBLE SULFATE CONTENT (%)	PASSING NO. 200 SIEVE (%)	SOIL TYPE
				SWELL (%)	COMPRESSION (%)	APPLIED PRESSURE (psf)	SWELL PRESSURE (psf)		LIQUID LIMIT	PLASTICITY INDEX			
TH-56	14	11.7	118									19	SANDSTONE
TH-57	9	8.1	124									24	SANDSTONE
TH-57	14	11.5	120									19	SANDSTONE
TH-58	4	8.6	113						32	16		29	FILL, SAND, CLAYEY
TH-59	4	5.4	122									41	SAND, CLAYEY (SC)
TH-59	19	19.3	107									40	SANDSTONE
TH-60	4	8.7	104									49	SAND, CLAYEY (SC)
TH-60	14	27.8	91	0.9		1,800							WEATHERED CLAYSTONE
TH-61	4	9.7	114	0.8		1,100	2,000	4.79					CLAY, VERY SANDY (CL)
TH-61	9	6.3	104		2.3	1,800		4.93					SAND, CLAYEY (SC)
TH-61	14	4.8	111		3.0	1,100		4.10					SAND, CLAYEY (SC)
TH-61	19	12.0	110									19	SANDSTONE
TH-61	24	13.0	118									23	SANDSTONE
TH-62	4	7.3	124									26	SANDSTONE
TH-62	9	7.9	114									17	SANDSTONE
TH-63	9	19.6	105	6.7		500					<0.01		CLAYSTONE
TH-63	14	12.0	111									33	SANDSTONE
TH-63	24	18.6	105	2.3		1,800							CLAYSTONE
TH-64	14	13.6	119	2.5		500							CLAYSTONE
TH-64	24	9.4	128									29	SANDSTONE
TH-65	14	18.6	108									18	SANDSTONE
TH-65	24	13.9	119									22	SANDSTONE
TH-65	34	17.9	112	2.5		3,000					0.06		CLAYSTONE
TH-66	4	9.4	94									68	CLAY, SANDY (CL)
TH-66	19	12.2	114	0.1		3,000							SANDSTONE
TH-67	9	4.4	114									18	SANDSTONE
TH-68	4	14.4	109	7.2		500					0.01		CLAYSTONE
TH-68	14	13.1	119									31	SANDSTONE
TH-68	24	12.1	100									15	SANDSTONE
TH-69	4	11.8	115	10.6		500							CLAYSTONE
TH-69	14	14.2	119						37	22		37	SANDSTONE
TH-69	24	17.4	110	1.0		3,000							CLAYSTONE
TH-70	4	11.9	112	6.9		500							CLAY, SANDY (CL)
TH-70	14	17.1	106	3.7		1,100							CLAYSTONE
TH-70	19	13.7										50	CLAYSTONE
TH-70	29	15.8	115	1.7		3,000							CLAYSTONE
TH-71	4	7.6	99									48	SAND, CLAYEY (SC)
TH-71	9	4.5	128									28	SANDSTONE
TH-72	4	9.2	120								<0.01	52	CLAY, VERY SANDY (CL)
TH-72	14	8.9	114	1.1		2,400							CLAY, VERY SANDY (CL)
TH-73	4	5.3	122									32	SAND, CLAYEY (SC)
TH-74	4	6.9	127									40	SANDSTONE
TH-75	4	11.8	118	4.7		1,800							CLAY, SANDY (CL)
TH-75	9	11.0	97		3.7	2,400							CLAY, SANDY (CL)
TH-76	4	9.4	125	1.8		1,800							CLAYSTONE
TH-76	9	9.2	119									29	SANDSTONE
TH-77	4	8.3	110	2.1		2,400							CLAY, SANDY (CL)
TH-77	9	8.4	119	0.5		3,000							CLAY, VERY SANDY (CL)
TH-78	4	12.5	113						35	12		49	SANDSTONE
TH-79	24	9.2	120									15	SANDSTONE

[illegible]



TABLE B-II

SUMMARY OF ESTIMATED POTENTIAL GROUND SURFACE HEAVE
AT PROPOSED GRADE BASED ON 24-FOOT DEPTH OF WETTING

Boring	Estimated Heave at Proposed Grade* (inches)	Boring	Estimated Heave at Proposed Grade* (inches)	Boring	Estimated Heave at Proposed Grade* (inches)
TH-1	1½	TH-38	1	TH-75	2½
TH-2	1½	TH-39	4½	TH-76	1½
TH-3	4	TH-40	3	TH-77	1½
TH-4	1	TH-41	1	TH-78	1½
TH-5	1½	TH-42	1½	TH-79	½
TH-6	<½	TH-43	2½	TH-80	4½
TH-7	<½	TH-44	1	TH-81	4
TH-8	2½	TH-45	1		
TH-9	2½	TH-46	2½		
TH-10	1½	TH-47	2½		
TH-11	1	TH-48	1		
TH-12	1½	TH-49	<½		
TH-13	1½	TH-50	1½		
TH-14	½	TH-51	1½		
TH-15	2½	TH-52	2		
TH-16	2	TH-53	1		
TH-17	1	TH-54	4		
TH-18	1½	TH-55	5		
TH-19	1	TH-56	1½		
TH-20	½	TH-57	1		
TH-21	1½	TH-58	1		
TH-22	4½	TH-59	3		
TH-23	1½	TH-60	2		
TH-24	½	TH-61	½		
TH-25	3	TH-62	<½		
TH-26	1½	TH-63	1½		
TH-27	1½	TH-64	1½		
TH-28	1½	TH-65	½		
TH-29	<½	TH-66	3		
TH-30	½	TH-67	<½		
TH-31	3	TH-68	3		
TH-32	1	TH-69	6½		
TH-33	1½	TH-70	5½		
TH-34	½	TH-71	2½		
TH-35	½	TH-72	2½		
TH-36	1½	TH-73	2		
TH-37	<½	TH-74	1		

*Future site grading fill was assumed to have an average swell of 0.5%



APPENDIX C
GUIDELINE SITE GRADING SPECIFICATIONS
Canyonside – Single and Multi-Family Development
Castle Pines, Colorado



GUIDELINE SITE GRADING SPECIFICATIONS
Canyonside – Single and Multi-Family Development
Castle Pines, Colorado

1. DESCRIPTION

This item shall consist of the excavation, transportation, placement and compaction of materials from locations indicated on the plans, or staked by the Engineer, as necessary to achieve preliminary street and overlot elevations. These specifications shall also apply to compaction of excess cut materials that may be placed outside of the subdivision and/or filing boundaries.

2. GENERAL

The Soils Representative shall be the Owner's representative. The Soils Representative shall approve fill materials, method of placement, moisture contents and percent compaction, and shall give written approval of the completed fill.

3. CLEARING JOB SITE

The Contractor shall remove all vegetation, trees, brush and rubbish before excavation or fill placement begins. The Contractor shall dispose of the cleared material to provide the Owner with a clean, neat appearing job site. Cleared material shall not be placed in areas to receive fill or where the material will support structures of any kind.

4. SCARIFYING AREA TO BE FILLED

Topsoil and vegetable matter shall be substantially removed from the ground surface upon which fill is to be placed. The surface shall then be plowed or scarified to a depth of 8 inches, moisture treated to above optimum moisture content, and compacted until the surface is free from ruts, hummocks or other uneven features, which would prevent uniform compaction by the equipment to be used.

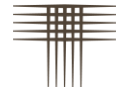
5. DIFFERENTIAL FILL DEPTHS BENEATH PROPOSED FOUNDATIONS

Depth of fill below a building footprint shall not differ more than 5 feet below bottom of foundations. Where walkout basements are planned, the difference should be determined by comparing the bottom of the frost wall footings to the upper-level footings. If fill depths are to vary greater than 5 feet below proposed foundations two methods can be used to lower the risk of differential settlement:

- Bearing the foundations below fill on non-expansive soils and bedrock;
- Sub-excavating the existing ground surface to the lowest existing ground surface beneath the proposed residence footprint.

6. COMPACTING AREA TO BE FILLED

After the foundation for the fill has been cleared and scarified, it shall be disked or bladed until it is free from large clods to a depth of 8 to 12 inches, brought to the proper



moisture content and compacted to not less than 95 percent of maximum density as determined in accordance with ASTM D 698. The foundation materials shall be worked, stabilized, or removed and replaced if necessary in accordance with the soils representative's recommendations in preparation for fill.

7. FILL MATERIALS

Fill soils shall be substantially free from vegetable matter or other deleterious substances, and shall not contain rocks having a diameter greater than six (6) inches and claystone or clay pieces larger than three (3) inches. Fill materials shall be obtained from cut areas shown on the plans or staked in the field by the Engineer.

On-site materials classifying as CL, CH, SC, SM, SW, SP, GP, GC and GM are acceptable. Concrete, asphalt, organic matter and other deleterious materials or debris shall not be used as fill.

8. MOISTURE CONTENT

For fill material classifying as CH, CL or SC, the fill shall be moisture treated to between 1 and 4 percent above optimum moisture content. Soils classifying as SM, SW, SP, GP, GC and GM shall be moisture treated to within 2 percent of optimum moisture content as determined from Proctor compaction tests. Sufficient laboratory compaction tests shall be made to determine the optimum moisture content for the various soils encountered in borrow areas. Fill placed greater than 20 feet below proposed grade should be moisture-conditioned to within 2 percent below and 1 percent above optimum moisture content.

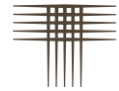
The Contractor may be required to add moisture to the excavation materials in the borrow area if, in the opinion of the Soils Representative, it is not possible to obtain uniform moisture content by adding water on the fill surface. The Contractor may be required to rake or disc the fill soils to provide uniform moisture content through the soils.

The application of water to embankment materials shall be made with any type of watering equipment approved by the Soils Representative, which will give the desired results. Water jets from the spreader shall not be directed at the embankment with such force that fill materials are washed out.

Should too much water be added to any part of the fill, such that the material is too wet to permit the desired compaction from being obtained, rolling and all work on that section of the fill shall be delayed until the material has been allowed to dry to the required moisture content. The Contractor will be permitted to rework wet material in an approved manner to hasten its drying.

9. COMPACTION OF FILL AREAS

Selected fill material shall be placed and mixed in evenly spread layers. After each fill layer has been placed, it shall be uniformly compacted to not less than the specified percentage of maximum density. Fill shall be compacted to at least 95 percent of the maximum density as determined in accordance with ASTM D 698. If fill is placed at depths greater than 20 feet below proposed grade, fill shall be compacted to at least 98 percent of the maximum density as determined in accordance with ASTM D 698. At the option of the Soils Representative, soils classifying as SW, GP, GC, or GM may be compacted to



95 percent of maximum density as determined in accordance with ASTM D 1557 or 70 percent relative density for cohesionless sand soils. Fill materials shall be placed such that the thickness of loose materials does not exceed 8 inches and the compacted lift thickness does not exceed 6 inches.

Compaction as specified above shall be obtained by the use of sheepfoot rollers, multiple-wheel pneumatic-tired rollers, or other equipment approved for soils classifying as CL, CH, or SC. Granular fill shall be compacted using vibratory equipment or other approved equipment. Compaction shall be accomplished while the fill material is at the specified moisture content. Compaction of each layer shall be continuous over the entire area. Compaction equipment shall make sufficient passes to ensure that the required density is obtained.

10. COMPACTION OF SLOPES

Fill slopes shall be compacted by means of sheepfoot rollers or other suitable equipment. Compaction operations shall be continued until slopes are stable, but not too dense for planting, and there is not an appreciable amount of loose soils on the slopes. Compaction of slopes may be done progressively in increments of three to five feet (3' to 5') in height or after the fill is brought to its total height. Permanent fill slopes shall not exceed 3:1 (horizontal to vertical).

11. PLACEMENT OF FILL ON NATURAL SLOPES

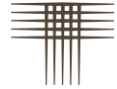
Where natural slopes are steeper than 20 percent in grade and the placement of fill is required, cut benches shall be provided at the rate of one bench for each 5 feet in height (minimum of two benches). Benches shall be at least 10 feet in width. Larger bench widths may be required by the Engineer. Fill shall be placed on completed benches as outlined within this specification.

12. DENSITY TESTS

Field density tests shall be made by the Soils Representative at locations and depths of his choosing. Where sheepfoot rollers are used, the soil may be disturbed to a depth of several inches. Density tests shall be taken in compacted material below the disturbed surface. When density tests indicate that the density or moisture content of any layer of fill or portion thereof is below that required, the particular layer or portion shall be re-worked until the required density or moisture content has been achieved.

13. SEASONAL LIMITS

No fill material shall be placed, spread or rolled while it is frozen, thawing, or during unfavorable weather conditions. When work is interrupted by heavy precipitation, fill operations shall not be resumed until the Soils Representative indicates that the moisture content and density of previously placed materials are as specified.



14. NOTICE REGARDING START OF GRADING

The Contractor shall submit notification to the Soils Representative and Owner advising them of the start of grading operations at least three (3) days in advance of the starting date. Notification shall also be submitted at least 3 days in advance of any resumption dates when grading operations have been stopped for any reason other than adverse weather conditions.

15. REPORTING OF FIELD DENSITY TESTS

Density tests made by the Soils Representative, as specified under "Density Tests" above, shall be submitted progressively to the Owner. Dry density, moisture content, and percentage compaction shall be reported for each test taken.

16. DECLARATION REGARDING COMPLETED FILL

The Soils Engineer shall provide a written declaration stating that the site was filled with acceptable materials, and was placed in general accordance with the specification.



APPENDIX D
GUIDELINE SUB-EXCAVATION SPECIFICATIONS
Canyonside – Single and Multi-Family Development
Castle Pines, Colorado

Note: This guideline is intended only for use with sub-excavation. If sub-excavation is not selected, the guidelines in Appendix C should be followed.



GUIDELINE SUB-EXCAVATION SPECIFICATIONS
Canyonside – Single and Multi-Family Development
Castle Pines, Colorado

1. DESCRIPTION

This item shall consist of the excavation, transportation, placement and compaction of materials from locations indicated on the plans, or staked by the Engineer, as necessary to achieve preliminary street and overlot elevations. These specifications shall also apply to compaction of materials that may be placed outside of the development boundaries.

2. GENERAL

The Soils Engineer shall be the Owner's representative. The Soils Engineer shall observe fill materials, method of placement, moisture content and percent compaction, and shall provide written opinions of the completed fill.

3. CLEARING JOB SITE

The Contractor shall remove all vegetation and debris before excavation or fill placement is begun. The Contractor shall dispose of the cleared material to provide the Owner with a clean, neat appearing job site. Cleared material shall not be placed in areas to receive fill where the material will support structures of any kind.

4. SCARIFYING AREA TO BE FILLED

All topsoil and vegetable matter shall be removed from the ground surface where fill is to be placed. The surface shall then be plowed or scarified to a depth of 8 inches, moisture treated to above optimum moisture content, and compacted until the surface is free from ruts, hummocks or other uneven features, which would prevent uniform compaction by the equipment to be used.

5. COMPACTING AREA TO BE FILLED

After the foundation for the fill has been cleared and scarified, it shall be disked or bladed until it is free from large clods, brought to the proper moisture content, (1 percent to 4 percent above optimum moisture content) and compacted to not less than 95 percent of maximum density as determined in accordance with ASTM D 698.

6. FILL MATERIALS

Fill soils shall be free from vegetable matter or other deleterious substances, and shall not contain rocks having a diameter greater than six (6) inches or clay and claystone having a diameter greater than three (3) inches. Fill materials shall be obtained from cut areas shown on the plans or staked in the field by the Engineer.

On-site materials classifying as CL, CH, SC, SM, SP, GP, GC and GM are acceptable. Concrete, asphalt, and other deleterious materials or debris shall not be used as fill.



7. MOISTURE CONTENT

Fill materials shall be moisture-conditioned to within limits of optimum moisture content specified in "Moisture Content and Density Criteria". Sufficient laboratory compaction tests shall be made to determine the optimum moisture content for the various soils encountered in borrow areas or imported to the site.

The Contractor may be required to add moisture to the excavation materials in the borrow area if, in the opinion of the Soils Engineer, it is not possible to obtain uniform moisture content by adding water on the fill surface. The Contractor will be required to rake or disc the fill to provide uniform moisture content throughout the fill.

The application of water to embankment materials shall be made with any type of watering equipment that will give the desired results. Water jets from the spreader shall not be directed at the embankment with such force that fill materials are washed out.

Should too much water be added to any part of the fill, such that the material is too wet to permit the desired compaction from being obtained, rolling and all work on that section of the fill shall be delayed until the material has been allowed to dry to the required moisture content. The Contractor will be permitted to rework wet material in an approved manner to hasten its drying.

8. COMPACTION OF FILL MATERIALS

Selected fill material shall be placed and mixed in evenly spread layers. After each fill layer has been placed, it shall be uniformly compacted to not less than the specified percentage of maximum density given in "Moisture Content and Density Criteria". Fill materials shall be placed such that the thickness of loose material does not exceed 8 inches and the compacted lift thickness does not exceed 6 inches.

Compaction, as specified above, shall be obtained by the use of suitable equipment. Compaction shall be accomplished while the fill material is at the specified moisture content. Compaction of each layer shall be continuous over the entire area. Compaction equipment shall make sufficient trips to ensure that the required density is obtained.

9. MOISTURE CONTENT AND DENSITY CRITERIA

Fill material shall be substantially compacted to at least 95 percent of standard Proctor maximum dry density (ASTM D 698, AASHTO T 99) dry density at 1 percent to 4 percent above optimum moisture content. Additional criteria for acceptance are presented in DENSITY TESTS.

10. DENSITY TESTS

Field density tests shall be made by the Soils Engineer at locations and depths of his choosing. Where sheepfoot rollers are used, the soil may be disturbed to a depth of several inches. Density tests shall be taken in compacted material below the disturbed surface. When density tests indicate the density or moisture content of any layer of fill or portion thereof not within specifications, the particular layer or portion shall be reworked until the required density or moisture content has been achieved.



Allowable ranges of moisture content and density given in MOISTURE CONTENT AND DENSITY CRITERIA are based on design considerations. The moisture shall be controlled by the Contractor so that moisture content of the compacted earth fill, as determined by tests performed by the Soils Engineer, shall be within the limits given. The Soils Engineer will inform the Contractor when the placement moisture is less than or exceeds the limits specified and the Contractor shall immediately make adjustments in procedures as necessary to maintain placement moisture content within the specified limits, to satisfy the following requirements.

A. Moisture

1. The average moisture content of material tested each day shall not be less than 1.5 percent over optimum moisture content.
2. Material represented by samples tested having moisture lower than 1 percent over optimum will be rejected. Such rejected materials shall be reworked until moisture equal to or greater than 1 percent above optimum is achieved.

B. Density

1. The average dry density of material tested each day shall not be less than 95 percent of standard Proctor maximum dry density (ASTM D 698).
2. No more than 10 percent of the material represented by the samples tested shall be at dry densities less than 95 percent of standard Proctor maximum dry density (ASTM D 698).
3. Material represented by samples tested having dry density less than 93 percent of standard Proctor maximum dry density (ASTM D 698) will be rejected. Such rejected materials shall be reworked until a dry density equal to or greater than 95 percent of standard Proctor maximum dry density (ASTM D 698) is obtained.

11. OBSERVATION AND TESTING OF FILL

Observation by the Soils Engineer shall be sufficient during the placement of fill and compaction operations so that they can declare the fill was placed in general conformance with specifications. All observations necessary to test the placement of fill and observe compaction operations will be at the expense of the Owner.

12. SEASONAL LIMITS

No fill material shall be placed, spread or rolled while it is frozen, thawing, or during unfavorable weather conditions. When work is interrupted by heavy precipitation, fill operations shall not be resumed until the Soils Engineer indicates the moisture content and density of previously placed materials are as specified.



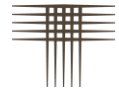
13. REPORTING OF FIELD DENSITY TESTS

Density tests made by the Soils Engineer, as specified under "Density Tests" above, shall be submitted progressively to the Owner. Dry density, moisture content and percentage compaction shall be reported for each test taken.



APPENDIX E

FLEXIBLE AND RIGID PAVEMENT MATERIALS,
CONSTRUCTION AND MAINTENANCE GUIDELINES



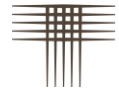
MATERIAL GUIDELINES FOR FLEXIBLE AND RIGID PAVEMENTS

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 9 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).
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)

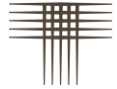
1. HMA should be composed of a mixture of aggregate, filler, hydrated lime and asphalt cement. Some mixes may require polymer modified asphalt cement or make use of up to 20 percent reclaimed asphalt pavement (RAP). A project mix design is recommended and periodic checks on the project site should be 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 meet the requirements of the Superpave Performance Graded (PG) Binders. The minimum performing asphalt cement should be PG 64-22 for use along the Front Range. The use of PG 58-28 or PG 58-22 asphalt cement has been known to cause tenderness in pavements in the Front Range area and should be avoided.
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 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 3.0 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. Proof-rolling should be performed with the heaviest machine available at the time. The proof roller should be selected from machines providing both mass and high contact pressure.

Portland Cement Concrete (PCC)

1. Portland cement concrete should meet CDOT Class P concrete and have a minimum compressive strength of 4,200 psi at 28 days and a minimum modulus of rupture (flexural strength) of 650 psi. A job mix design is recommended and periodic checks on the job site should be made to verify compliance with specifications.
2. Portland cement should conform to ASTM C 150.
3. Portland cement concrete should not be placed when the subgrade or air temperature is below 40°F.
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 debris, vegetation/organics and other deleterious materials, scarified, moisture conditioned and compacted. Subgrade soils should be moisture conditioned to within 2 percent of optimum moisture content for sand/gravel or between 1 and 4 percent above optimum for clay, and compacted to at least 95 percent of maximum standard Proctor dry density (ASTM D 698, AASHTO T 99).
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, 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, AASHTO T 180).
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 3.0 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 City of Castle Pines should apply.

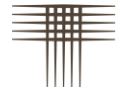


RIGID PAVEMENT CONSTRUCTION GUIDELINES

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 debris, vegetation/organics and other deleterious materials, scarified, moisture conditioned to within 2 percent of optimum moisture content for sand/gravel or between 1 and 4 percent above optimum for clay, and compacted to at least 95 percent of standard Proctor maximum dry density (ASTM D 698, AASHTO T 99).
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 City of Castle Pines 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.

Annual Preventive Maintenance

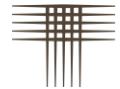
- Visual pavement evaluations should be performed each year.
- Reports documenting the progress of distress should be kept current to provide information on effective times to apply preventive maintenance treatments.
- Crack sealing should be performed annually as new cracks appear.

3 to 5-Year Preventive Maintenance

- 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.

5 to 10-Year Corrective Maintenance

- 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.