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GEOTECHNICAL ENGINEERING STUDY
PAVEMENT THICKNESS DESIGN
PROPOSED THORNTON FIRE STATION NO. 7
NORTHWEST OF YORK STREET AND EAST 156TH AVENUE
THORNTON, COLORADO

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Project No. 21-3-157

August 12, 2021
Revised: October 6, 2021

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SUMMARY

1. The subsurface conditions encountered at the site were evaluated by drilling six (6) exploratory borings to depths ranging from about 5 to 35 feet below existing ground surface. The four (4) structure borings generally encountered about 8 inches of topsoil overlying 11.5 to 15.5 feet of naturally deposited lean clay soil. The clayey soils were underlain by a 5- to 13-foot-thick lens of granular soils, which was in turn underlain by sandstone and claystone bedrock. The bedrock continued to the explored depths of about 35 feet. The two (2) pavement borings also encountered about 8 inches of topsoil overlying naturally deposited clayey soils. The clayey soils continued to the explored depths ranging from about 5 to 10 feet in these borings.

   Bedrock was encountered in four (4) of the exploratory borings at depths ranging from about 21 feet to about 26 feet below ground surface. The bedrock encountered in the borings generally consisted of sandstone and claystone bedrock. Borings 1 and 2 encountered 11- and 10-foot-thick layers of sandstone bedrock, respectively, between the natural granular soils and the underlying claystone bedrock.

2. Groundwater was encountered in four (4) of the borings drilled within the fire station footprint during drilling at depths ranging from about 16 to 17 feet below the ground surface. The borings were left open in order to measure stabilized groundwater levels, where present. Follow-up groundwater level measurements made 7 days after drilling encountered groundwater at depths ranging from about 15 to 16 feet.

3. We recommend that straight-shaft drilled piers be used to support the structure(s) and should be designed using an allowable end-bearing value of 25,000 psf and an allowable skin friction value of 2,500 psf within the bedrock.

4. Floor slabs should be supported structurally by the building foundation system and elevated above the underlying expansive soils.

5. We recommend that all pavement sections be underlain by at least 2 feet of properly compacted fill material. Two subgrade material types are presented within the body of this report, which provide two alternatives for pavement thickness designs.

PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical engineering study for the proposed Thornton Fire Station No. 7 project to be constructed on the west side of York Street approximately 700 feet north of East 156th Avenue in Thornton, Colorado. The project site is generally shown on Fig. 1. The study was conducted in accordance with the scope of work in our Proposal No. P3-21-116 with a date of February 2, 2021.

A field exploration program consisting of exploratory borings was conducted to obtain information on subsurface conditions. Samples of soils obtained during the field exploration were tested in the laboratory to determine their strength, compressibility or swell characteristics, and classification. Results of the field exploration and laboratory testing were analyzed to develop recommendations for the building foundations and floor slabs, exterior flatwork areas, and pavements. The results of the field exploration and laboratory testing are presented herein.

This report has been prepared to summarize the data obtained during this study and to present our conclusions and recommendations based on the proposed construction and the subsurface conditions encountered. Design parameters and a discussion of geotechnical engineering considerations related to construction of the proposed facility are included in the report.

PROPOSED CONSTRUCTION

Proposed site development drawings provided to us indicate that the fire station site will be developed with an apparatus bay, bunk rooms and other traditional fire station appurtenances. The current concept shows a total structure footprint on the order of about 6,400 ft^2. Paved access drive lanes and parking areas will also be constructed to the west and south of the building and allowing access to the apparatus bay. Site access will be provided to and from York Street on the south side of the building.

If the proposed construction varies significantly from that generally described above or depicted in this report, we should be notified to reevaluate the conclusions and recommendations provided herein.

SITE CONDITIONS

The project site at the time of drilling was being actively farmed to grow corn. The site was nearly flat within the project limits and appeared to have recently been planted prior to our field investigation. A farmhouse was located north of the site, York Street was located east of the site, and more agricultural land was located south and west of the site.

SUBSURFACE CONDITIONS

The subsurface conditions encountered at the site were evaluated by drilling a total of six (6) exploratory borings to depths ranging from about 5 to 35 feet below existing ground surface. Four (4) of the exploratory borings were drilled within the proposed fire station footprint to depths of about 35 feet. Two (2) of the exploratory borings were drilled with the proposed driveways and parking areas to depths ranging from about 5 to 10 feet. The approximate locations of the borings are shown on Fig. 1. The logs of the exploratory borings are presented on Fig. 2, and a legend and associated explanatory notes are also presented on Fig. 3.

Subsurface Soil and Bedrock Conditions: The four (4) structure borings generally encountered about 8 inches of topsoil overlying 11.5 to 15.5 feet of naturally deposited lean clay soil. The clayey soils were underlain by a 5- to 13-foot-thick lens of granular soils, which was in turn underlain by sandstone and claystone bedrock. The bedrock continued to the explored depths of about 35 feet. The two (2) pavement borings also encountered about 8 inches of topsoil overlying naturally deposited clayey soils. The clayey soils continued to the explored depths ranging from about 5 to 10 feet.

The natural clayey soils encountered in the borings varied from lean clay to sandy lean clay and were fine to coarse grained, slightly moist to moist and brown with occasional red deposits. The natural granular soils varied between poorly graded sand with silt and clayey sand and were fine to coarse grained, moist to wet, and light brown. Based on sampler penetration resistance blow counts, the natural clayey overburden soils were generally stiff to very stiff and the granular soils were medium dense to dense.

Bedrock was encountered in four (4) of the exploratory borings at depths ranging from about 21 feet to about 26 feet below ground surface. The bedrock encountered in the borings generally consisted of sandstone and claystone bedrock. Borings 1 and 2 encountered 11- and 10-foot-thick layers of sandstone bedrock, respectively, between the natural granular soils and the

underlying claystone bedrock. The sandstone bedrock was fine to medium grained, moist and tan to orange with occasional iron staining. The claystone bedrock was fine grained, moist and olive gray. Based on sampler penetration resistance blow counts, the bedrock was hard to very hard.

Groundwater Conditions: Groundwater was encountered in four (4) of the borings drilled within the fire station footprint during drilling at depths ranging from about 16 to 17 feet below the ground surface. The borings were left open in order to measure stabilized groundwater levels, where present. Follow-up groundwater level measurements made 7 days after drilling encountered groundwater at depths ranging from about 15 to 16 feet.

The borings were backfilled after stabilized groundwater levels were measured. The borings were backfilled with cuttings from the borings. Metal and/or wood rods were used to compact the cuttings in the borings.

LABORATORY TESTING

Laboratory testing was performed on selected samples obtained from the borings to determine in-situ soil moisture content and dry density, Atterberg limits, swell-consolidation characteristics, percent passing the No. 200 sieve, and concentration of water-soluble sulfates. The results of the laboratory tests are shown to the right of the logs on Fig. 2 and are summarized in Table 1. The results of specific tests are graphically plotted on Figs. 4 through 6. The testing was conducted in general accordance with recognized test procedures, primarily those of the ASTM International and the Colorado Department of Transportation (CDOT).

Swell-Consolidation: Swell-consolidation tests were conducted on samples of the natural lean clay. The swell-consolidation tests were performed in order to determine the compressibility and swell characteristics of the samples under loading and when submerged in water. Each sample was prepared and placed in a confining ring between porous discs, subjected to a surcharge pressure of 200 or 1,000 psf, and allowed to consolidate before being submerged. The sample height was monitored until deformation practically ceased under each load increment.

Results of the swell-consolidation tests are plotted as a curve of the final strain at each increment of pressure against the log of the pressure, and are presented on Figs. 4 through 6. Based on the results of swell-consolidation tests, the natural clayey overburden soil samples exhibited low swell

potential (0.4% to 1.0%) upon wetting at a surcharge pressure of 200 psf. The overburden clayey soils exhibited nil to low consolidation potential (0.0 to 0.7%) upon wetting at a surcharge pressure of 1,000 psf. The claystone bedrock exhibited low swell potential (0.4 to 1.8%) upon wetting at a surcharge pressure of 1,000 psf.

Index Properties: Samples were classified into categories of similar engineering properties in general accordance with the Unified Soil Classification System. This system is based on index properties, including liquid limit and plasticity index and grain size distribution. Values for moisture content, dry density, liquid limit and plasticity index, and the percent of soil passing the U.S. No. 4 and 200 sieves are presented in Table I and adjacent to the corresponding sample on the boring logs.

Corrosion Protection: Testing for corrosion potential of the on-site soils was not performed as part of this study. The on-site overburden materials are predominantly clayey with fines contents on the order of about 64 to 86%, with the near surface soils being largely near the upper end of that range. Our experience is that clayey soils such as those at this site have a low electrical resistivity, which in turn, indicates a high potential for corrosive attack on buried metal piping due to stray electrical currents. As such, we recommend that buried metal utilities be protected from the potential corrosive environment. Common protection products include epoxy coatings or wrapping the piping and connections with a polyethylene wrap. Any common method of buried metal pipe corrosion protection is acceptable at this site.

GEOTECHNICAL CONSIDERATIONS

We have not been provided with existing or proposed site grades. We should be contracted to provide additional and/or revised recommendations if the proposed building finished floor elevation will be located below the existing ground surface elevation. We strongly encourage the elevation of the finished floor slab of the proposed building be located at least 2 feet above the existing ground surface to help promoted positive surface drainage away from the proposed building foundations and floor slabs.

Numerous foundation and ground improvement alternatives exist for construction on the site that will mitigate post-construction settling movements. Two such alternatives appear to be the most likely and economical. Specifically, 1) subexcavation and replacement of the near surface soils with properly moisture conditioned and compacted structural fill, which would allow use of shallow

foundation systems; and 2) installation of deep foundations (driven piles, drilled shafts extending into the bedrock, or screw piles). These options were discussed in detail with the design team and based on the risk level that the City of Thornton is willing to accept, deep foundations were selected as the foundation system the best met the needs and desires of the City.

Deep Foundation Option: The most positive method of mitigating foundation movement would be to support the structure(s) on drilled shafts, driven piles or screw piles bearing in the underlying bedrock materials. This alternative would mitigate the risks of potential movements associated with placing shallow foundations on compacted fill materials; however, potential soil supported floor slab movements may occur as discussed below in the “Floor Slab Considerations” section.

A down-side to drilled shafts is the necessity to case the borings during construction of the shafts as well as the need to dispose of cuttings brought to the ground surface as a result of shaft excavation. Driven piles and/or screw piles do not bring spoils to the ground surface; however, lateral support is generally limited requiring installation of battered piles to help resist lateral loads.

Floor Slab Considerations: Utilization of the deep foundation option described above does not mitigate potential heaving movements of the on-site soils. In order to mitigate post-construction floor slab movements using a deep foundation alternative, we recommend that the fire station be constructed using floor slabs supported structurally on the deep foundation elements.

SITE GRADING

Cut and Fill Slopes: Major stability problems are not anticipated if site grading is carefully planned and cut and fill slopes do not exceed approximately 10 feet in height.

Permanent unretained cuts in the overburden soils less than 10 feet in height should be sloped at 3 horizontal to 1 vertical, or flatter. The risk of slope instability will be significantly increased if seepage is encountered in cuts. For shallow cuts in the existing overburden soils, we do not anticipate seepage will be encountered. Where groundwater seepage is encountered during construction, a stability analysis should be conducted to determine if the seepage will adversely affect the cut.

Permanent fills up to 20 feet in height can be used if the fill slopes do not exceed 3 horizontal to 1 vertical and the fills are properly compacted and drained. The ground surface underlying all fills should be carefully prepared by removing all organic matter, scarification to a depth of 12 inches and compacting to 95% of the standard Proctor (ASTM D698) maximum dry density at moisture contents as described in under the “Placement and Compaction Conditions” subheading of the Site Grading section of this report. Fills should be continuously benched into existing slopes that exceeding 4 horizontal to 1 vertical.

Good surface drainage should be provided around all permanent cuts and fills to direct surface runoff away from the slope faces. Fill slopes, cut slopes and other stripped areas should be protected against erosion by vegetation or other methods.

No formal stability analyses were performed to evaluate the slopes recommended above. Published literature and our experience with similar cuts and fills indicate the recommended slopes should have adequate factors of safety. If a detailed stability analysis is required, we should be notified.

Temporary Excavations: We assume that the site excavations will be constructed by generally over-excavating the side slopes to a stable configuration where enough space is available. All excavations greater than 4 feet and less than 20 feet in depth should be constructed in accordance with OSHA requirements, as well as state, local and other applicable requirements. OSHA requires excavations or trenching over 20 feet deep be designed by a registered professional engineer.

The natural lean clay soils generally classify as OSHA Type B. The granular soils will classify as OSHA Type C. The bedrock underlying the site is anticipated to classify as OSHA Type A soil, although fractured bedrock and non- to weakly-cemented sandstone bedrock would classify as OSHA Type B soils and may classify as Type C soils depending on the degree of fracturing and/or cementation. If unstable soil conditions or groundwater are encountered, the geotechnical engineer should be notified so that additional recommendations can be provided, if necessary.

Excavated slopes may soften or loosen due to construction traffic and erode from surface runoff. Measures to keep surface runoff from excavation slopes, including diversion berms, should be considered.

Existing Fills within Building Areas: Existing fills, if encountered, should not be considered suitable for support of any structure. Flatwork may be supported on a thickness of properly moisture conditioned and compacted fill as discussed in the Floor Slabs and/or Pavement Design section(s) of this report.

Material Specifications: The following material specifications are presented for fills on the project site. A geotechnical engineer should evaluate the suitability of all proposed import fill material, if required, for the project prior to placement.

1. *Structural Fill Beneath Buildings*: Fill placed beneath building structure(s) should consist of on-site overburden soil or imported materials free of deleterious matter and claystone fragments. Consideration should be given to using existing fill materials from the parking area to fill the building footprint and then replacing the removed materials from the pavement areas with imported granular soils.

2. *Pavement Subgrade*: Materials within 2 feet of the pavement subgrade elevation should consist of the on-site soils or imported materials (properly moisture conditioned and compacted) exclusive of claystone. Properly moisture conditioned fill materials are those materials whose moisture content has been adjusted to within the range(s) indicated herein for each soil type. Similarly, properly compacted fill materials are those materials whose dry density, which placed, has been adjusted to within the range(s) indicated herein for each soil type.

3. *Pipe Bedding Material*: Pipe bedding material should be a free draining, coarse grained sand and/or fine gravel. The on-site soils are generally non to very cohesive, fine-grained soils and are susceptible to erosion and scour.

   Consideration should be given to installation of a cutoff wall around utilities that are placed on and surrounded by granular bedding material. If installed, a cutoff wall should be constructed at the highest point of elevation where the utility(ies) enter or depart the site to mitigate water from flowing through the bedding onto the site. Cutoff walls should be constructed with materials having low permeability such as clayey soils with fines contents in excess of 70%. Use of controlled low strength material (CLSM) is also an acceptable cutoff wall material.

A cutoff wall should be constructed for the entire width and depth of a utility trench and have a distance parallel to the utility of at least 3 feet. The intent of the cutoff wall is to mitigate groundwater from being able to freely (relatively unimpeded) flow within the utility trench bedding material and trench backfill from off-site sources.

4. *Aggregate Base Course*: Material should be crushed stone, crushed slag, recycled concrete, crushed gravel or natural gravel which conforms to CDOT Specifications for Class 6 criteria for aggregate base course.

5. *Utility Trench Backfill*: Material excavated from the utility trenches may be used for backfill provided it does not contain unsuitable material such as organic matter, deleterious substances or particles larger than 4 inches.

6. *Material Suitability*: Material placed within the building envelope plus 10 feet outside of the building envelope should consist of on-site overburden or imported materials.

   Fill materials used within the building footprint to achieve the final subgrade elevations should consist of the on-site soils or an imported fill material having less than 70% passing the No. 200 sieve, a maximum liquid limit of 35 and a maximum plasticity index of 12.

   Imported granular fill used below pavement sections, if this option is selected, should contain less than 50% passing the No. 200 sieve, have a maximum liquid limit of 30 and a maximum plasticity index of 10. The granular material should have a minimum R-value of 40. Also, the swell potential when remolded to 95% of the ASTM D 698 standard Proctor maximum dry density at optimum moisture content should be less than ½% under a 200 psf surcharge pressure.

   All fill material should be free of vegetation, brush, sod and other deleterious substances and should not contain rocks, debris or lumps having a diameter of more than 4 inches. Rocks, debris or lumps should be dispersed throughout the fill and "nesting" of these materials should be avoided. The geotechnical engineer should evaluate the suitability of proposed import fill materials prior to placement. To avoid nesting, we recommend that particles between 3 to 4 inches in size be placed such that no more than about 3 to 4 particles occur within any given 1-cubic foot of material placed. The intent of this criteria is to help ensure that the larger particles “float” within the soil matrix and reduces the potential that voids could be created during placement.

Placement and Compaction Specifications: We recommend the following compaction criteria be used on the project:

1. *Moisture Content*: Fill materials should be compacted as outlined below with moisture contents between the optimum moisture content and 3 percentage points above optimum moisture for predominantly fine-grained material and within -2 to +2 percentage points of optimum for predominantly granular soils. Predominantly fine-grained soils are considered materials that have 50% or more materials passing the No. 200 sieve. Predominantly granular soils are considered to be materials that have less than 50% passing the No. 200 sieve.

   The contractor should be aware that the on-site and/or imported fine-grained soils may become somewhat unstable and deform under wheel loads if placed near the upper end of the moisture range(s). Some fill instability is not a concern in deeper fills provided the required density is achieved; instability is a concern primarily in the upper 2 to 3 feet of subgrade elevations.

2. *Placement and Degree of Compaction*: Structural fill beneath foundations and slab-on-grade floors, fill adjacent to shallow spread footing foundations, and wall backfill should be placed in 6- to 12-inch lifts as necessary, provided proper compaction can be achieved.

   The following compaction criteria should be followed during construction:

Fill Location	Percentage of Maximum Standard Proctor Density (ASTM D-698)
Wall Backfill	
Backfill Less than 8 Feet below finished grade	95%
Settlement Sensitive Areas	98%[1]
Exterior Backfill More than 8 Feet below finished grade	98%[1]
Beneath Pavements and Settlement-Sensitive Hardscape Areas	
Fill Less Than 8 Feet below finished grade	95%
Fill More Than 8 Feet below finished grade	98%[2]
Utility Trenches	
Interior	95%
Landscape and Other Areas	95%

1. Some difficulty could be encountered achieving adequate compaction with small equipment to avoid exerting excessive compaction stresses on walls.
2. It should be noted that some of the existing fills on the site deeper than 8 feet below the proposed subgrade elevation were likely placed to levels less than the recommended values. Discussion of the risks associated with the lower compaction level are discussed above in Geotechnical Considerations

3. *Subgrade Preparation*: Areas receiving new fill should be prepared as recommended in specific sections of this report to provide a uniform base for fill placement. All other areas to receive new fill not specifically addressed herein should be scarified to a depth of at least 8 inches and recompacted to at least 95% of the standard Proctor (ASTM D 698) maximum dry density at moisture contents recommended above.

Construction Monitoring: A representative of the geotechnical engineer should observe and test fill placement. Structural fills beneath buildings and foundations should be observed and tested on a full-time basis. Full time observation and testing is a critical component to reducing the risk of post-construction settlement of the fills.

FOUNDATION RECOMMENDATIONS

Given the discussion of the proposed foundation options provided above and the perceived sensitivity to post-construction movements, we recommend that drilled shaft foundations be utilized to support the building and floor slabs. We should be contacted to revise the below recommendations if the risks associated with floor slabs on grade and shallow foundations is deemed acceptable.

The design and construction criteria presented below should be observed for a straight-shaft pier foundation system. The construction details should be considered when preparing project documents.

1. Piers should be designed for an allowable end bearing pressure of 25,000 psf and a skin friction of 2,500 psf for the portion of the pier in bedrock. Uplift due to structural loadings on the piers can be resisted by using 75% of the allowable skin friction value plus an allowance for pier weight.

2. Piers should also be designed for a minimum dead load pressure of 15,000 psf calculated as the unfactored dead load applied to the pier cross sectional area. Our experience indicates application of dead load pressure is the most effective way to resist foundation movement due to swelling soils. However, if the minimum dead load requirement cannot be achieved and the piers are loaded as heavily as practicable, the pier length should be extended beyond the minimum bedrock penetration and minimum length to mitigate the dead load deficit. This can be accomplished by assuming one-half of the skin friction given above acts in the direction to resist uplift caused by swelling soil around the upper portion of the pier. The owner should be aware of an increased potential for foundation movement if the recommended minimum dead load pressure is not met.

3. A minimum penetration of 8 feet into the bedrock and a minimum pier length of 20 feet are recommended. Both requirements for minimum pier length and minimum bedrock penetration should be met.

4. Piers should be designed to resist lateral loads using a modulus of horizontal subgrade reaction in the properly compacted fill and natural clay soils of 50 tcf and a modulus of horizontal subgrade reaction of 250 tcf in the bedrock. The modulus values given are linear modulus values intended for use in simplified hand calculations and are for a long one-foot wide pier and must be corrected for pier size. If more rigorous analysis is desired, a computer application such as LPILE should be used.

5. The lateral capacity of the piers may be analyzed using the LPile computer program and the parameters provided in the following table. The strength criteria provided in the table are for use with that software application only and may not be appropriate for other usages.

Material	c (psf)	ø	γ_T	k_s	k_c	ε_{50}	Soil Model Type
Overburden soils/ Properly compacted fill	750	0	120	500	200	0.007	1
Bedrock	8,000	0	125	2,000	800	0.004	1

c Cohesion intercept (pounds per square foot)
Φ Angle of internal friction (degrees)
γ_T Total unit weight (pounds per cubic foot)
k_s Initial static modulus of horizontal subgrade reaction (pounds per cubic inch)
k_c Initial cyclic modulus of horizontal subgrade reaction (pounds per cubic inch)
ε_{50} Strain at 50 percent of peak shear strength

Soil Types:
1. Stiff clay without free water (Reese)

6. Closely-spaced piers and pier groups will require appropriate reductions of the axial, uplift and lateral capacities based on the effective envelope of the pier group. These reductions can be avoided by spacing the piers at a distance of at least 3 pier diameters center-to-center for axial loading, 6 pier diameters center-to-center in the direction parallel to lateral loading, and 5 pier diameters center-to-center in the direction perpendicular to lateral loading. More closely spaced piles should be studied on an individual basis to determine the appropriate reduction in axial and lateral load design parameters.

7. If the minimum pier spacings recommended above for lateral loading cannot be achieved, we recommend that the lateral load-displacement curve (p-y curve) for an isolated pier be modified for closely-spaced piers using p-multipliers to reduce all the p-values on the curve. With this approach, the computed load carrying capacity of the pier in a group is reduced relative to the isolated pile capacity. The modified p-y curve should then be reentered into the L-Pile software to calculate the pile deflection. The reduction in capacity for the leading pier, the pier leading the direction of movement of the group, is less than that for the trailing piers.

   For center-to-center spacing of piers in the group in the direction of loading expressed in multiples of the pier diameter, we recommend p-multipliers of 0.8 and 1.0 for pier spacings of 3 and 5 diameters, respectively, for the leading row of piers, 0.4 and 0.85 for pier spacings of 3 and 5 diameters, respectively, for the second row of piers, and 0.3 and 0.7 for pier spacings of 3 and 5 diameters, respectively, for rows 3 and higher. For loading in a direction perpendicular to the row of piers, the p-multipliers are 1.0 for a pier spacing of 5 diameters, 0.8 for a pier spacing of 3 diameters, and 0.5 for a pier spacing of 1 diameter. P-multiplier values for other pier spacing values should be determined by interpolation. These values are generally consistent with Table 10.7.2.4-1 of the 2017 AASHTO LRFD Bridge Design Specifications (8th Edition). It will be necessary to determine the load distribution between the piers that attain deflection compatibility because the leading pier carries a higher proportion of the group load and the pier cap prevents differential movement between the piers.

8. Piers should be reinforced their full length to resist an unfactored net tensile force from swelling soil pressure of at least 40 kips. The recommended tensile force is for a 1-foot diameter pier and should be increased in proportion to the pier diameter for larger piers.

If the design dead load greater than or less than the recommended dead load, the requirement for tension reinforcement should be decreased or increased accordingly to account for the difference.

9. A 4-inch void should be provided beneath the grade beams to concentrate pier loadings and to separate the expansive soil from the grade beams. Absence of a void space will result in a reduction in dead load pressure on the piers which could result in upward movement of the foundation system. A void should also be provided beneath necessary pier caps.

10. The pier length-to-diameter ratio should not exceed 30 to facilitate proper cleaning and observation of the pier hole.

11. Concrete used in the piers should be a fluid mix with sufficient slump so it will fill the void between reinforcing steel and the pier hole. We recommend a concrete slump in the range of 5 to 8 inches be used.

12. Based on the results of our field exploration, laboratory testing, and our experience with similar, properly constructed drilled pier foundations, we estimate pier settlement will be low. Generally, we estimate the settlement of a pier 1 to 3 feet in diameter will be less than 1-inch of total settlement with approximately ¾-inch of differential settlement across the building foot print when designed according to the criteria presented herein. The settlement of closely spaced piers will be larger and should be studied on an individual basis.

13. Pier holes should be properly cleaned prior to the placement of concrete. Pier holes should be considered properly cleaned when, upon visual inspection, there are no particles larger than about 1-inch at the bottom of the hole. Additionally, the bottom of the hole should expose undisturbed claystone bedrock.

14. The presence of water in the exploratory borings indicates the use of temporary casing or dewatering equipment in the pier holes may be required to reduce water infiltration. The requirements for casing and dewatering equipment can sometimes be reduced by placing concrete immediately upon cleaning and observing the pier hole. In no case should concrete be placed in more than 3 inches of water unless placed with an approved tremie method.

15. When water and/or drilling slurry is present outside the casing, care should be taken that concrete of sufficiently high slump is placed to a sufficiently high elevation inside the casing to prevent intrusion of the water and/or slurry into the concrete when the casing is withdrawn.

16. The drilled shaft contractor should mobilize equipment of sufficient size and operating condition to achieve the required bedrock penetration.

17. Care should be taken that the pier shafts are not oversized at the top. Mushroomed pier tops can reduce the effective dead load pressure on the piers.

18. Concrete should be placed in piers the same day they are drilled. The presence of water or caving soils may require that concrete be placed immediately after the pier hole is completed. Failure to place concrete the day of drilling will normally result in a requirement for additional bedrock penetration.

19. Difficulty may be encountered in establishing a casing seat in the bedrock to achieve a positive cutoff of groundwater seepage into the hole. Additional bedrock penetration may be required to compensate for the skin friction lost due to disturbance caused by installation of the casing. Skin friction should be neglected in the cased portion of the hole. The amount of additional penetration should be determined in the field at the time of construction. The contract documents should advise potential drilled shaft contractors of these subsurface conditions. In addition, careful consideration should be given to preparing bid items to avoid high costs for potential overruns.

20. A representative of the geotechnical engineer should observe pier drilling operations on a full-time basis to assist in identification of adequate bedrock strata and monitor pier construction procedures.

FLOOR SLABS

Floor slabs present a very difficult problem where expansive materials are present near floor slab elevation because sufficient dead load cannot be imposed on them to resist the uplift pressure generated when the materials are wetted and expand. Based on the moisture-volume change

characteristics of the materials encountered and the sensitivity to post-construction movements, we recommend use of a structural floor above a well-ventilated void space. The floor should be supported on grade beams and piers the same as the main structure.

Design of an underfloor void should consider drainage and moisture control. We recommend a minimum 6-inch void beneath floors. Utility lines should not be supported on the subgrade, unless adequate measures are taken to account for differential movement between grade supported utilities and slabs. If utilities are connected to the floor or floor openings, void spaces should also be provided below the utility lines. The utility lines should be supported by suitable means such as hangers as necessary. We recommend that void spaces be designed with positive surface drainage and a collection point or outlet so that free-water introduced into these spaces can be removed. High humidity can develop in void spaces due to the transmission of water vapor through moist soils. Void space humidity should be controlled through ventilation and/or the use of a vapor barrier on the underside of the structure floor (between the void form and the floor slab).

It is extremely important that exterior slabs-on-grade and pavements be isolated from the building foundations. Many expansive soil related problems are related to ineffective isolation between pavements/floor slabs and foundation-supported components of structures. Careful design detailing is necessary at locations such as exterior stairway landings and entry points. Consideration should be considered to incorporating doorway stoops into the structurally supported portions of the structure.

The exterior slab-on-grade flat work adjacent to building should be placed on a minimum of 2 feet of on-site soils, or imported granular pavement subgrade fill as described in the Site Grading section.

RETAINING STRUCTURES

Retaining structures constructed on the site which are laterally supported and can be expected to undergo only a moderate amount of deflection should be designed for an at-rest lateral earth pressure computed on the basis of an equivalent fluid unit weight of 75 pcf for backfill consisting of the on-site fine grained soils and 60 pcf for backfill consisting of imported granular materials conforming to CDOT Class 1 Structure Backfill requirements.

Cantilevered retaining structures less than 15 feet in height which can be expected to deflect sufficiently to mobilize the full active earth pressure condition should be designed for a lateral earth pressure computed on the basis of an equivalent fluid unit weight of 55 pcf for backfill consisting of the on-site soils and 40 pcf for backfill consisting of imported granular materials conforming to CDOT Class 1 Structure Backfill.

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

SEISMIC DESIGN CRITERIA

The soil profile generally will consist of about varying depths of overburden soils underlain by hard to very hard bedrock. The bedrock is considered to extend to a depth of at least 100 feet below ground surface. The existing and anticipated overburden soils will classify as International Building Code (IBC) Site Class D. The underlying bedrock generally classifies as IBC Site Class C. Based on the proposed depth of overburden and the proposed site grading, we recommend a design soil profile of IBC Site Class D. Based on the subsurface profile, site seismicity, and the anticipated depth of ground water, liquefaction is not a design consideration.

WATER-SOLUBLE SULFATES

Concentrations of water-soluble sulfates measured in a sample of on-site soils was 0.02%. This concentration represents a Class 0 severity exposure to sulfate attack on concrete exposed to these materials. The degree of attack is based on a range of Class 0, Class 1, Class 2, and Class 3 severity exposure as presented in ACI 201 and in Section 601 of the 2011 Colorado Department of Transportation (CDOT) Standards and Specifications.

Based on the laboratory data and our experience, we believe special sulfate resistant cement will generally not be required for concrete exposed to the natural overburden soils.

UNDERDRAIN SYSTEM

We recommend that an underdrain system be considered. The underdrain system should consist of a perimeter drain extending along the perimeter of the building. The alignment of the drain

system should preferably be just outside of the building perimeter. The drains should consist of 4-inch diameter, rigid, perforated PVC pipe placed in the bottom of a trench and surrounded above the invert level with free-draining granular material. The drain pipe should have perforations smaller than ½-inch. The free draining granular material should have a maximum particle size of 1½ inches with no particles smaller than ½-inch. The stone should be wrapped with a non-woven geotextile product to prevent migration of the surrounding fine materials into the drainage aggregate. A conceptual drain detail is provided in Appendix A.

The free-draining granular material should extend up to within 1.5 feet of the finished ground surface. An alternative to using free-draining gravel behind the wall would be to incorporate a geocomposite drainage board in lieu of the aggregate only along the foundation wall. The drainage pipe should still be surrounded by drainage aggregate with a geotextile warp surrounding the aggregate. The drain lines should be placed no higher than the bottom of the footing subgrade elevation and graded to a sump or sumps at a minimum slope of ½%. The granular underdrain system should be sloped to a sump or multiple sumps where water can be removed by pumping or gravity drainage.

An alternative to using minus 1½-inch stone would be to provide a filter sock around drain pipe. If a filter sock is installed around the drain pipe, then an aggregate product that meets CDOT Class C Filter criteria may be used. The non-woven geotextile may be omitted if this alternative is selected.

We recommend that the underdrain pumps be designed to handle a minimum flow of 20 gpm on a continuous basis, and that the sumps be sized to accept pumps capable of pumping up to 50 gpm on a continuous basis in case larger pumps are required. Standby pump capacity should be provided in the event of pump failure. We also believe an overdesigned pump capacity is desirable in the event ground water conditions change.

SURFACE DRAINAGE

Proper surface drainage is very important for acceptable performance of site structures during construction and after the construction has been completed. Drainage recommendations provided by local, state and national entities should be followed based on the intended use of each structure. The following recommendations should be used as guidelines and changes should be made only after consultation with the geotechnical engineer.

1. Excessive wetting or drying of the foundation and slab subgrade(s) should be avoided during construction.

2. Exterior backfill meet the material and placement requirements outlined in the “Site Grading” section of this report.

3. Care should be taken when compacting around the foundation walls and underground structures to avoid damage to the structures. Hand compaction procedures, if necessary, should be used to prevent lateral pressures from exceeding the design values.

4. The ground surface surrounding the exterior of site structures should be sloped to drain away from the foundations in all directions. We recommend a minimum slope of 12 inches in the first 10 feet in unpaved areas. Site drainage beyond the 10-foot zone should be designed to promote runoff and reduce infiltration. A minimum slope of 3 inches in the first 10 feet is recommended in the paved areas. These slopes may be changed as required for handicap access points in accordance with the Americans with Disabilities Act.

5. The upper 2 feet of the backfill should be relatively impervious material compacted as recommended above to limit infiltration of surface runoff.

6. Ponding of water should not be allowed in backfill material or in a zone within 10 feet of the foundations, whichever is greater.

7. Roof downspouts and drains should discharge well beyond the limits of all backfill.

8. Landscaping which requires relatively heavy irrigation and lawn sprinkler heads should be located at least 10 feet from foundations. Irrigation schemes are available which allow placement of lightly irrigated landscape near foundation walls in moisture sensitive soil areas. Drip irrigation heads with main lines located at least 10 feet from the foundation walls are acceptable provided irrigation quantities are limited.

9. Plastic membranes should not be used to cover the ground surface adjacent to foundation walls.

PAVEMENT DESIGN

A pavement section is a layered system designed to distribute concentrated traffic loads to the subgrade. Performance of the pavement structure is directly related to the physical properties of

the subgrade soils and traffic loadings. Soils are represented for pavement design purposes by means of a soil support value for flexible pavements and a modulus of subgrade reaction for rigid pavements. Both values are empirically related to strength.

Subgrade Materials: Based on the results of the field and laboratory studies, the majority of the subgrade materials at the site classify as A-6 with group indices between 9 and 18 in accordance with the American Association of State Highway and Transportation Officials (AASHTO) classification. Soils classifying as A-6 would generally be considered to provide poor subgrade support. The probable subgrade soils are expected to consist primarily of compacted fill composed of material generally classifying as A-6 soils. For design purposes, a resilient modulus value of 3,025 psi was selected for flexible pavements and a corrected modulus of subgrade reaction of 40 pci was selected for rigid pavements.

Design Traffic: Since anticipated traffic loading information was not available at the time of this report preparation, an 18-kip equivalent single axle loading (ESAL) value of 73,000 was assumed for the paved parking surfaces and an ESAL of 219,000 was assumed for truck routes. The values are selected based on our past experience for facilities of this nature. We believe that the ESAL values of 73,000 and 219,000 should be considered to classify as “Light Duty” and “Heavy Duty” pavement sections, respectively. The “Heavy Duty” pavement section should be constructed in locations of heavy vehicular traffic movements such as fire apparatus routes.

If estimated daily traffic volumes for the development are known to be different from those assumed, we should be provided with this information in order to reevaluate the pavement sections provided below.

Pavement Design: The following table presents the minimum pavement thickness recommendations for this project site using fill materials similar to the on-site soils:

Pavement Thickness Requirements Using Materials Similar to On-Site Soils

Paved Area	Full Depth Asphalt (inches)	Composite Section Asphalt/ABC (inches)	PCCP (inches)
Light Duty	7	4.5 / 8.0	6.0
Heavy Duty	8	5.5 / 9.0	7.0

ABC – Aggregate Base Course
PCCP – Portland Cement Concrete Pavement

The following table presents the minimum pavement thickness recommendations for this project site using at least 2 feet of granular fill materials as described above and having and R-Value of at least 40 (R-40):

Pavement Thickness Requirements Using Minimum R-40 Material

Paved Area	Full Depth Asphalt (inches)	Composite Section Asphalt/ABC (inches)	PCCP (inches)
Light Duty	4.5	3.5 / 6.0	5.0
Heavy Duty	5.5	4.0 / 6.0	6.5

ABC – Aggregate Base Course
PCCP – Portland Cement Concrete Pavement

For design purposes using the imported R-40 material, a resilient modulus value of 9,497 psi was selected for flexible pavements and a modulus of subgrade reaction of 60 pci was selected for rigid pavements.

Pavement Materials: The following are recommended material and placement requirements for pavement construction for this project site. We recommend that properties and mix designs for all materials proposed to be used for pavements be submitted for review to the geotechnical engineer prior to placement.

1. *Aggregate Base Course*: Aggregate base course (ABC) used beneath HMA pavements should meet the material specifications for Class 6 ABC stated in the current CDOT *"Standard Specifications for Road and Bridge Construction"*. The ABC should be placed and compacted as outlined in the "SITE GRADING" section of this report.

2. *Hot Mix Asphalt:* Hot mix asphalt (HMA) materials and mix designs should meet the applicable requirements indicated in the current CDOT *"Standard Specifications for Road and Bridge Construction"*. We recommend that the HMA used for this project is designed in accordance with the SuperPave gyratory mix design method. The mix should meet Grading S specifications with a SuperPave gyratory design revolution (N_{DESIGN}) of 75.

   A mix meeting Grading SX specification can be used for the top lift wearing course, however, this is optional. The mix design(s) for the HMA should use a performance grade (PG) asphalt binder of PG 58-28 or PG 64-22. However, we recommend the PG 58-28 binder which tends to perform better under relatively low traffic volumes. Placement and compaction of HMA should follow current CDOT standards and specifications.

3. *Portland Cement Concrete*: Portland Cement Concrete (PCC) pavement should meet Class P or D specifications and requirements in the current CDOT *"Standard Specifications for Road and Bridge Construction"*. Rigid PCC pavements are more sensitive to distress due to movement resulting from settlement or heave of the underlying base layer and/or subgrade than flexible asphalt pavements. The PCC pavement should contain sawed or formed joints to ¼ of the depth of the slab at a maximum distance of 12 to 14 feet on center.

   The above PCC pavement thicknesses are presented as un-reinforced slabs. Based on projects with similar vehicular loading in certain areas, we recommend that dowels be provided at transverse and longitudinal joints within the slabs located in the travel lanes of heavily loaded vehicles, loading docks and areas where truck turning movements are likely to be concentrated. Additionally, curbs and/or pans should be tied to the slabs. The dowels and tie bars will help minimize the risk for differential movements between slabs to assist in more uniformly transferring axle loads to the subgrade. The current CDOT *"Standard Specifications for Road and Bridge Construction"* provides some guidance on dowel and tie bar placement, as well as in the Standard Plans: M&S Standards. The proper sealing and maintenance of joints to minimize the infiltration of surface water is critical to the performance of PCC pavement, especially if dowels and tie bars are not installed.

<u>Subgrade Preparation:</u> The pavement subgrade within 2 feet of subgrade elevation should be properly moisture conditioned and compacted as outlined in the "Site Grading" section of this report. Prior to placing new fill for the pavement section, the entire subgrade area should be thoroughly scarified and well-mixed to a minimum depth of 12 inches, adjusted in moisture content and compacted as indicated in the "Site Grading" section of this report. Fill placed beneath the pavement should meet the material and compaction requirements for structural fill presented in the "Site Grading" section of this report. It should be noted that potential movements of the pavement surface(s) associated with moisture changes in the underlying soil will not be eliminated by providing the 2 feet of subexcavation and replacement. The most positive method to mitigate post-construction pavement movements is to remove all of the existing fill materials below the pavement subgrade and replacing the removed soils with imported granular fill meeting CDOT Class 6 aggregate base course criteria. Even replacing the removed materials with Class 6 base course will not eliminate the possibility of movement; however, the total movements will be significantly reduced. Based upon our experience, we recommended 2 feet of subgrade

improvement below pavements to provide a pavement structure that would perform reasonably well if good surface drainage is provided and routine maintenance such as crack sealing is performed. Generally, the deeper the subexcavation and replacement that is provided below the pavement section, the lower the risk of potential movements of the pavement surface. Reuse of the on-site fill materials below the pavement section will also result in an increased risk of movement given the measured swell potential and material properties determined in our laboratory.

Pavement design procedures assume a stable subgrade and the pavement subgrade should be proof-rolled, preferably within 48 hours prior to paving. The proof-roll should be performed using a heavily loaded pneumatic-tired vehicle such as a loaded water truck or large front-end loader. Areas that deform under wheel loads that are not stable should be removed and replaced to achieve a stable subgrade prior to paving. The contractor should be aware that the clay soils may become somewhat unstable and deform under wheel loads if placed near the upper end of the moisture content range.

Drainage: The collection and diversion of surface drainage away from paved areas is extremely important to the satisfactory performance of pavement. Drainage design should provide for the removal of water from paved areas and prevent the wetting of the subgrade soils.

DESIGN AND CONSTRUCTION SUPPORT SERVICES

Kumar & Associates, Inc. should be retained to review the project plans and specifications for conformance with the recommendations provided in our report. We are also available to assist the design team in preparing specifications for geotechnical aspects of the project, and performing additional studies, if necessary, to accommodate possible changes in the proposed construction.

We recommend that Kumar & Associates, Inc. be retained to provide construction observation and testing services to document that the intent of this report and the requirements of the plans and specifications are being followed during construction. This will allow us to identify possible variations in subsurface conditions from those encountered during this study and to allow us to re-evaluate our recommendations, if needed. We will not be responsible for implementation of the recommendations presented in this report by others, if we are not retained to provide construction observation and testing services.

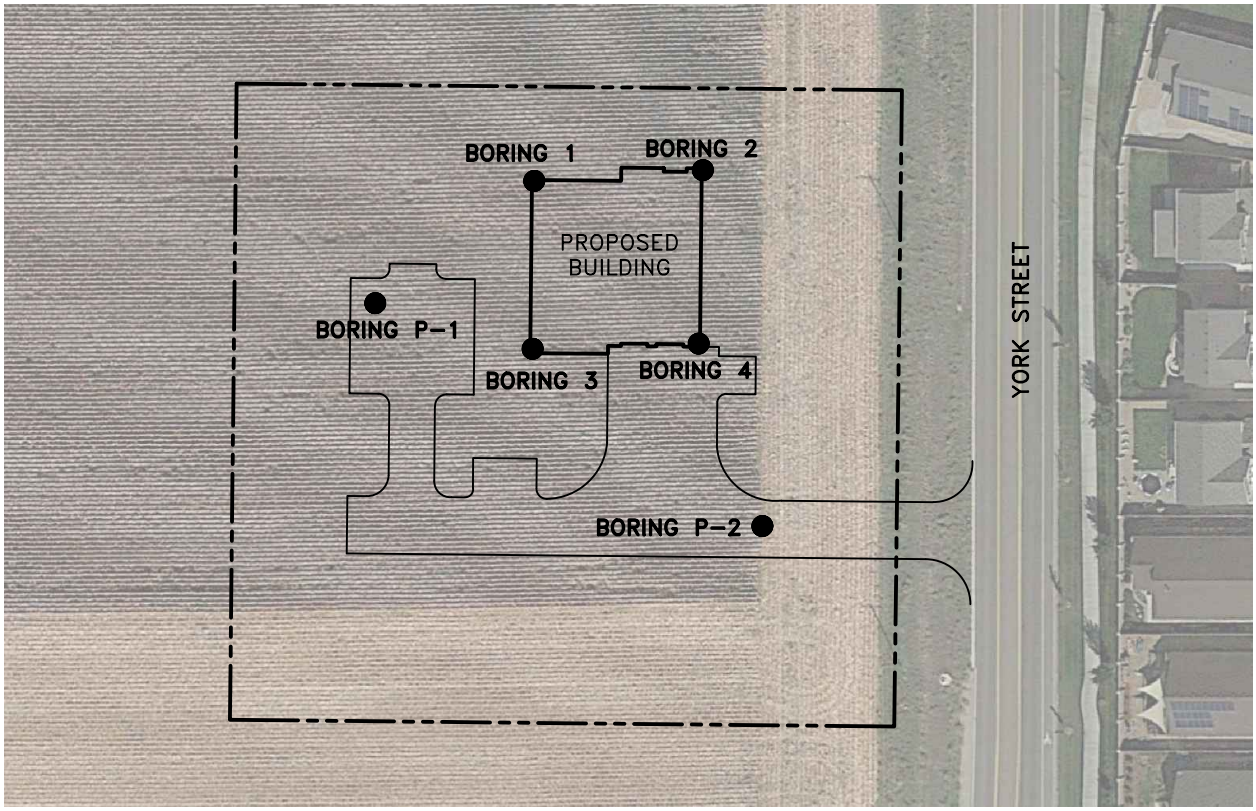
LIMITATIONS

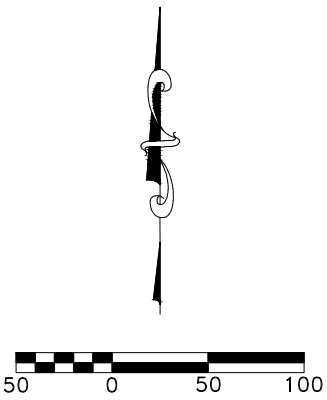
This study has been conducted in accordance with generally accepted geotechnical engineering practices in this area for exclusive use by the client for design purposes. The conclusions and recommendations submitted in this report are based upon the data obtained from the exploratory borings at the locations indicated on Fig. 1, and the proposed type of construction. This report may not reflect subsurface variations that occur between the exploratory borings, and the nature and extent of variations across the site may not become evident until site grading and excavations are performed. If during construction, fill, soil, rock or water conditions appear to be different from those described herein, Kumar & Associates, Inc. should be advised at once so that a re-evaluation of the recommendations presented in this report can be made. Kumar & Associates, Inc. is not responsible for liability associated with interpretation of subsurface data by others.

Swelling soils occur on this site. Such soils are stable at their natural moisture content but will undergo high volume changes with changes in moisture content. The extent and amount of perched water beneath the building site as a result of area irrigation and inadequate surface drainage is difficult, if not impossible, to foresee.

The recommendations presented in this report are based on current theories and experience of our engineers on the behavior of swelling soil in this area. The owner should be aware that there is a risk in constructing a building in an expansive soil area. Following the recommendations given by a geotechnical engineer, careful construction practice and prudent maintenance by the owner can, however, decrease the risk of foundation movement due to expansive soils.

JLB/ma
Rev. by: JH
cc: book, file





VICINITY MAP
NOT TO SCALE

21-3-157	Kumar & Associates	LOCATION OF EXPLORATORY BORINGS	Fig. 1

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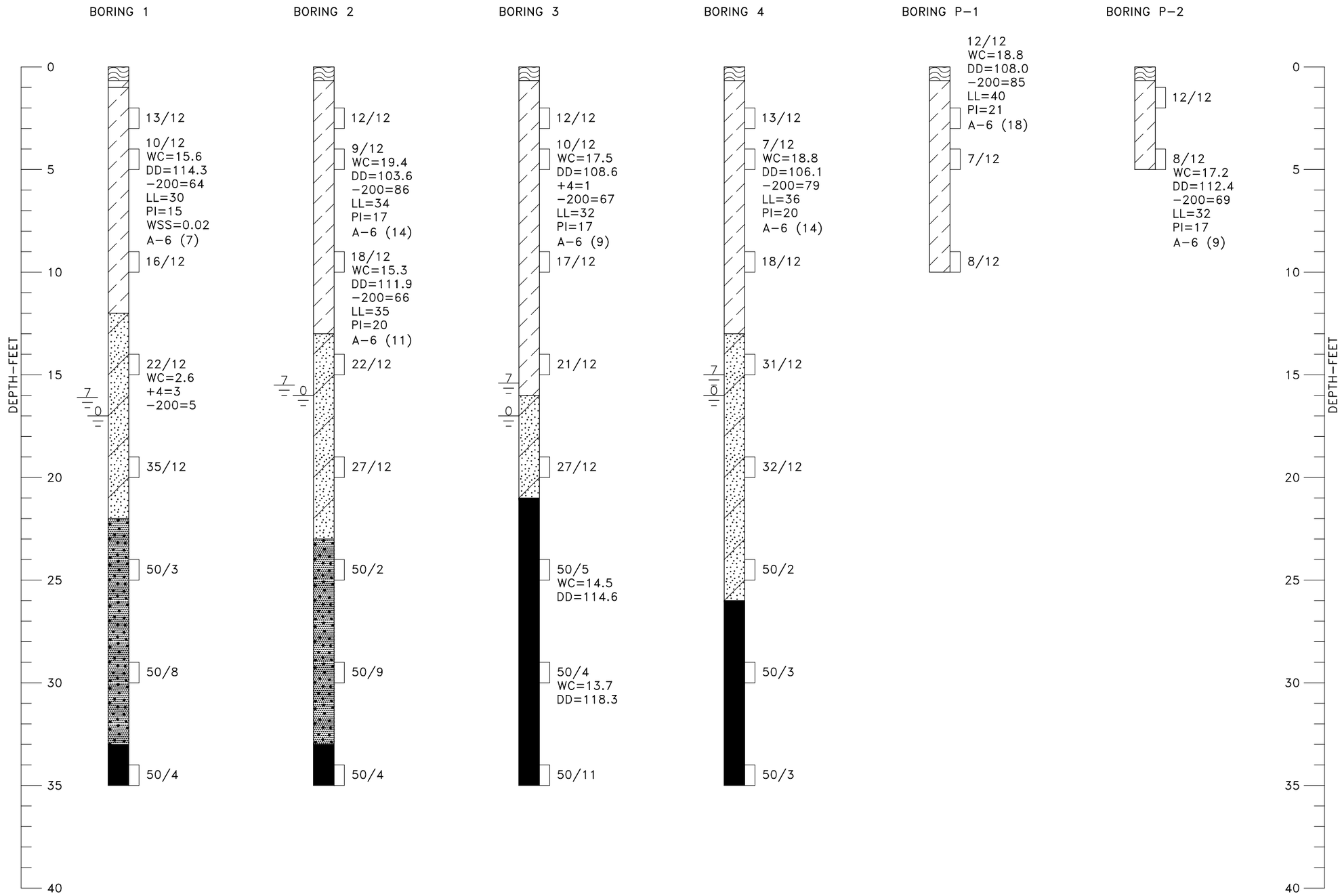


21−3−157	Kumar & Associates	THORNTON FIRE STATION No. 7	LOGS OF EXPLORATORY BORINGS	Fig. 2

LEGEND

TOPSOIL;

LEAN CLAY TO SANDY LEAN CLAY (CL), FINE TO COARSE GRAINED SAND FRACTION, STIFF TO VERY STIFF, SLIGHTLY MOIST TO BROWN, BROWN WITH OCCASIONAL RED DEPOSITS.

POORLY GRADED SAND WITH SILT (SP-SM) TO CLAY SAND (SC), FINE TO COARSE GRAINED SAND FRACTION, MEDIUM DENSE TO DENSE, MOIST TO WET, LIGHT BROWN. MATERIAL APPEARS TO TRANSITION BETWEEN SP-SM AND SC THROUGHOUT THE THICKNESS.

SANDSTONE BEDROCK, FINE TO MEDIUM GRAINED SAND FRACTION, OCCASIONAL IRON STAINING. HARD TO VERY HARD, TAN TO ORANGE.

CLAYSTONE BEDROCK, FINE GRAINED SAND FRACTION, VERY HARD, MOIST, OLIVE GRAY.

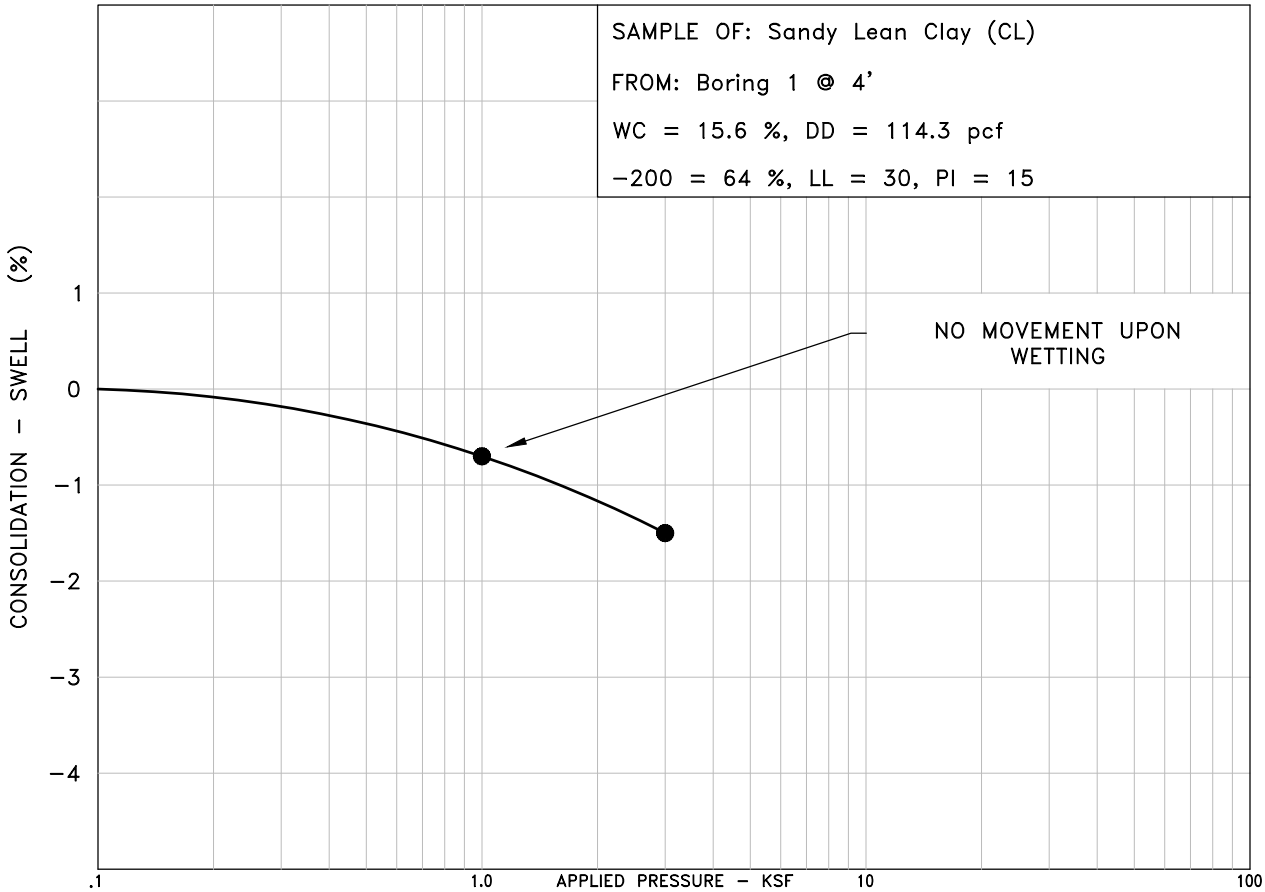
DRIVE SAMPLE, 2-INCH I.D. CALIFORNIA LINER SAMPLE.

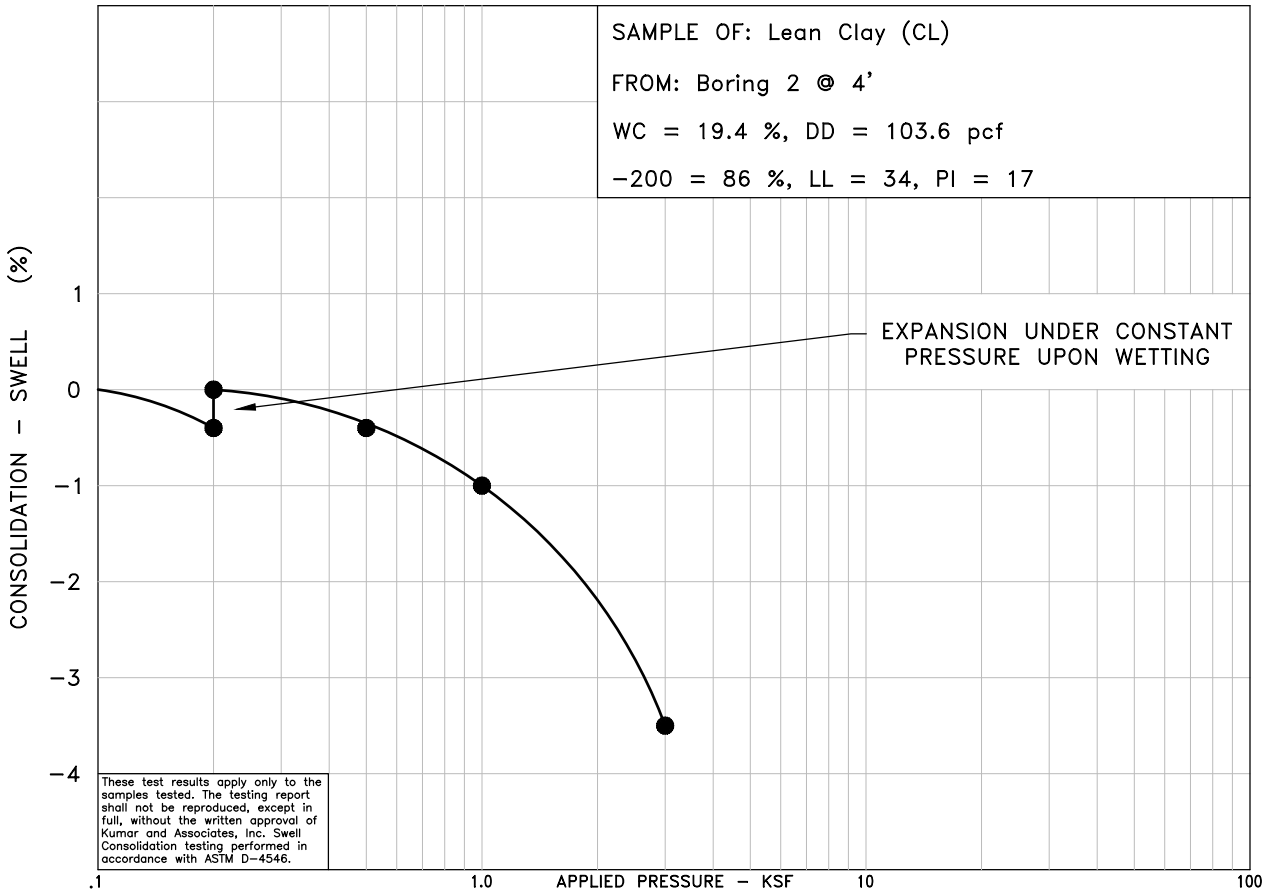
13/12 DRIVE SAMPLE BLOW COUNT. INDICATES THAT 13 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE THE SAMPLER 12 INCHES.

0 DEPTH TO WATER LEVEL ENCOUNTERED AT THE TIME OF DRILLING.

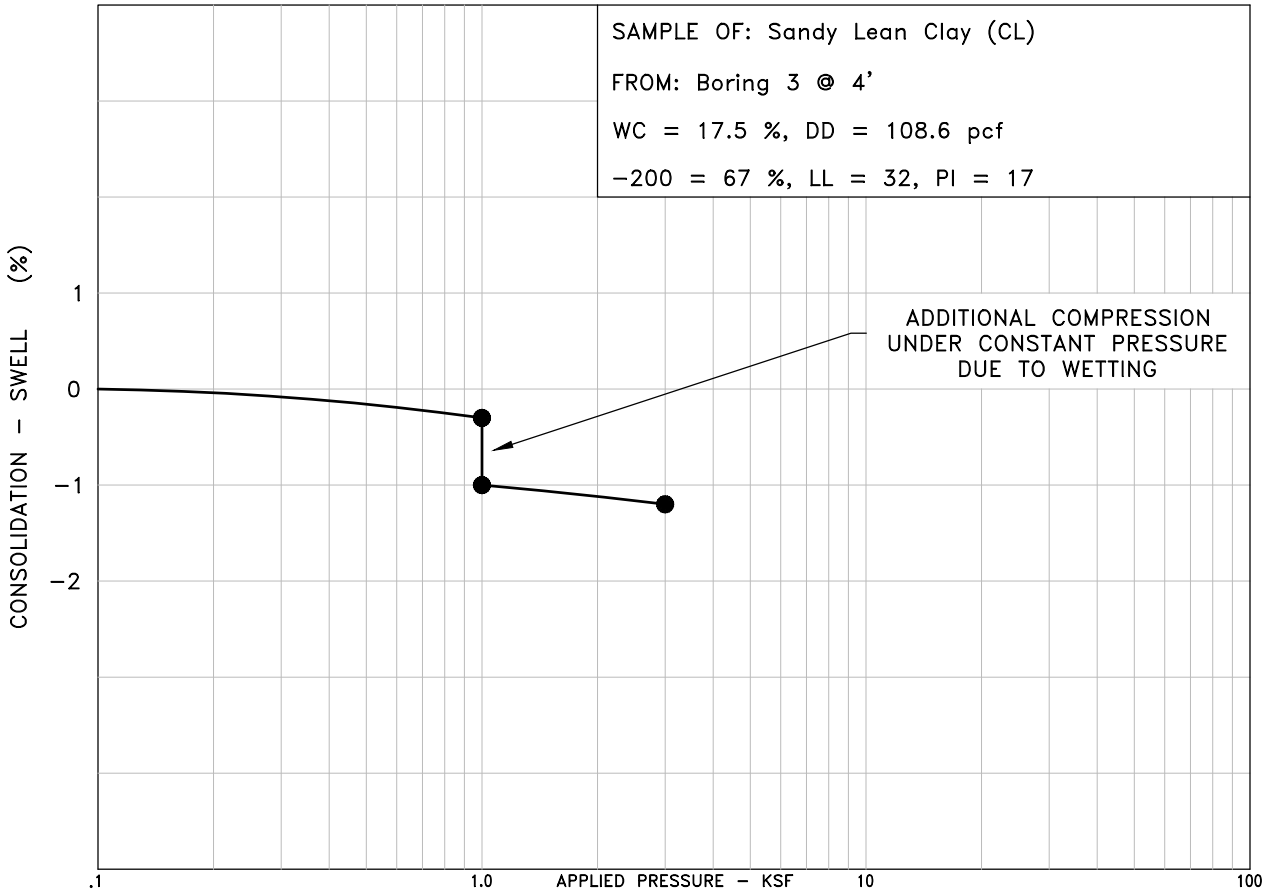
NOTES

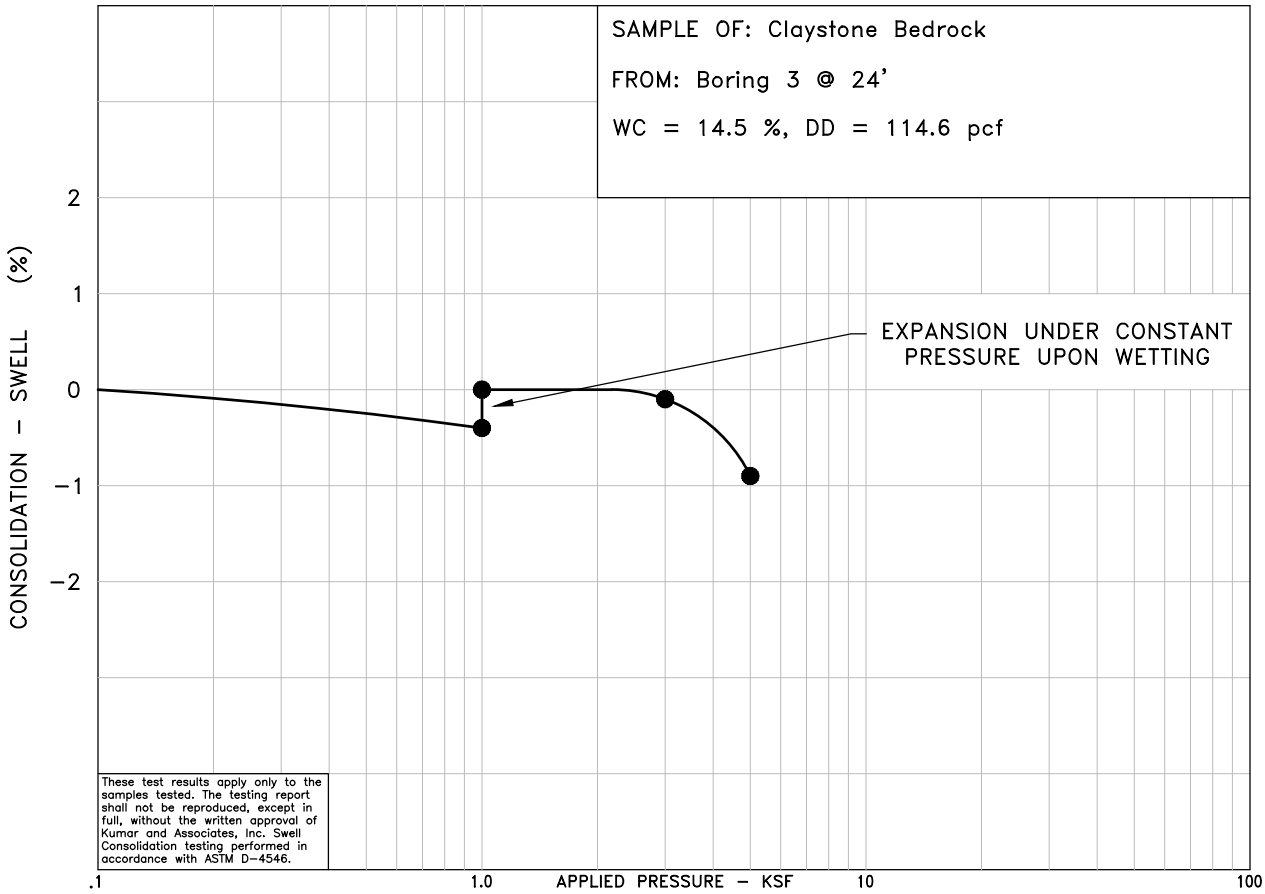
1. THE EXPLORATORY BORINGS WERE DRILLED ON JUNE 17 AND 18 WITH A 4-INCH-DIAMETER CONTINUOUS-FLIGHT POWER AUGER.
2. THE LOCATIONS OF THE EXPLORATORY BORINGS WERE MEASURED APPROXIMATELY BY PACING FROM FEATURES SHOWN ON THE SITE PLAN PROVIDED.
3. THE ELEVATIONS OF THE EXPLORATORY BORINGS WERE NOT MEASURED AND THE LOGS OF THE EXPLORATORY BORINGS ARE PLOTTED TO DEPTH.
4. THE EXPLORATORY BORING LOCATIONS SHOULD BE CONSIDERED ACCURATE ONLY TO THE DEGREE IMPLIED BY THE METHOD USED.
5. THE LINES BETWEEN MATERIALS SHOWN ON THE EXPLORATORY BORING LOGS REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN MATERIAL TYPES AND THE TRANSITIONS MAY BE GRADUAL.
6. GROUNDWATER LEVELS SHOWN ON THE LOGS WERE MEASURED AT THE TIME AND UNDER CONDITIONS INDICATED. FLUCTUATIONS IN THE WATER LEVEL MAY OCCUR WITH TIME.
7. LABORATORY TEST RESULTS:
   WC = WATER CONTENT (%) (ASTM D2216);
   DD = DRY DENSITY (pcf) (ASTM D2216);
   +4 = PERCENTAGE RETAINED ON NO. 4 SIEVE (ASTM D6913);
   -200= PERCENTAGE PASSING NO. 200 SIEVE (ASTM D1140);
   LL = LIQUID LIMIT (ASTM D4318);
   PI = PLASTICITY INDEX (ASTM D4318);
   WSS = WATER SOLUBLE SULFATES (%) (CP-L 2103);
   A-6 (7) = AASHTO CLASSIFICATION (GROUP INDEX) (AASHTO M145).



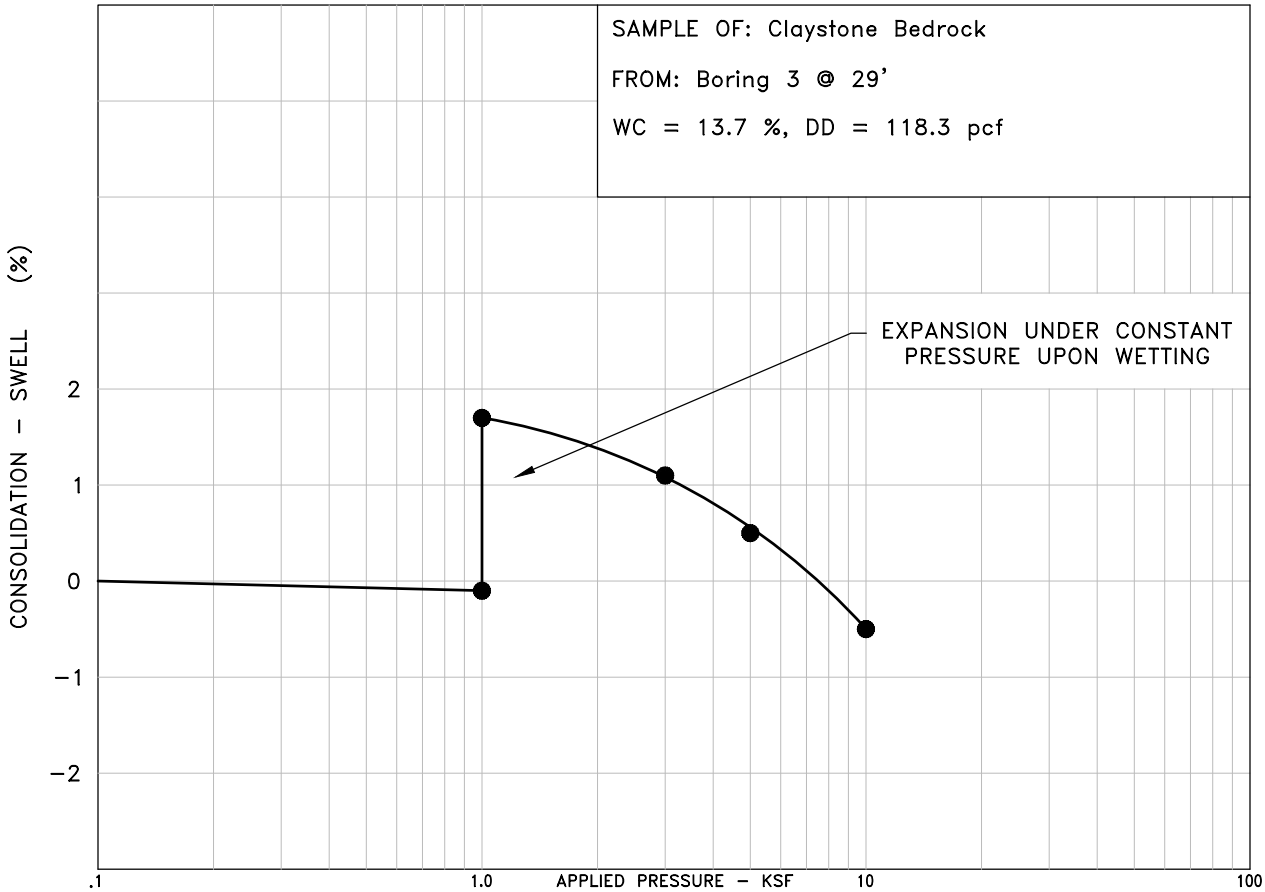


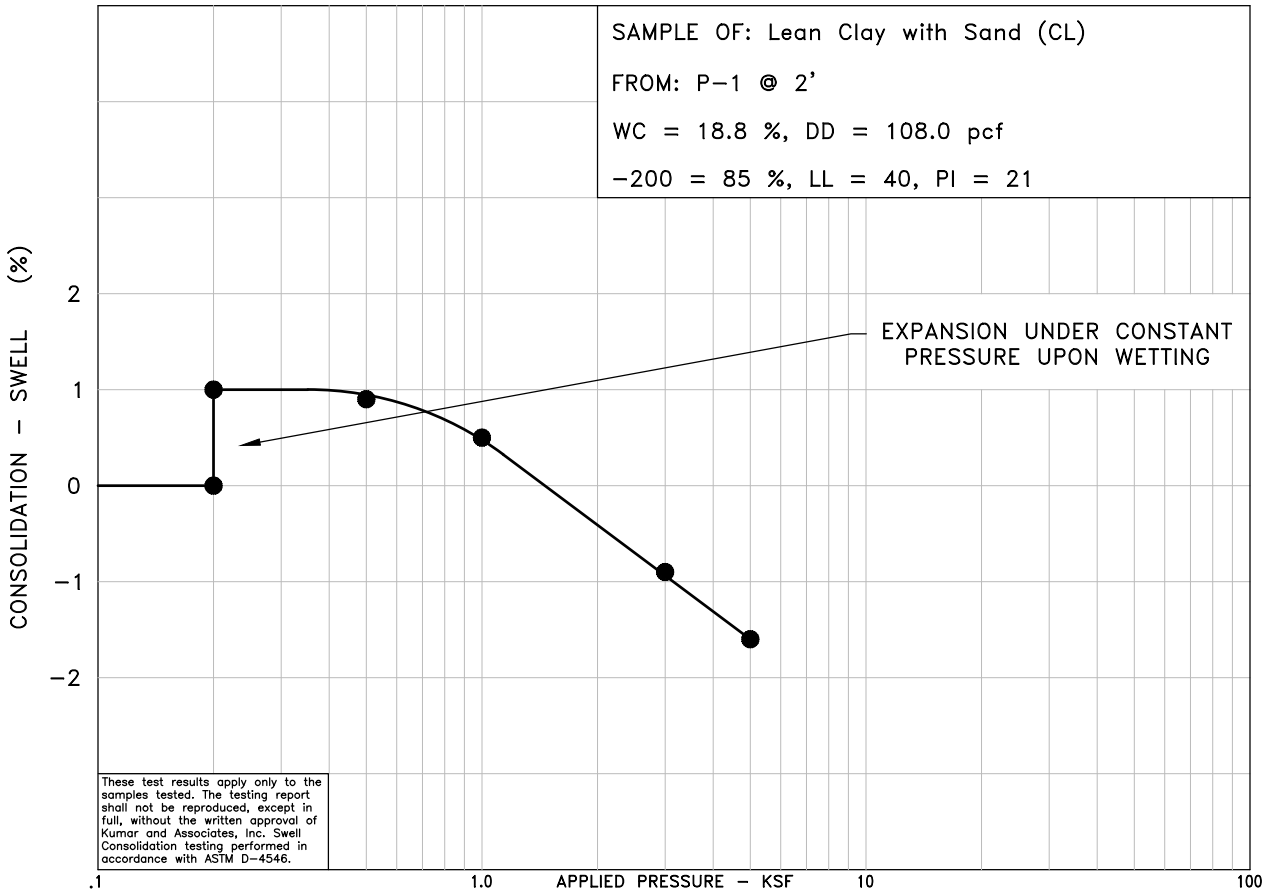
21−3−157	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULTS	Fig. 4



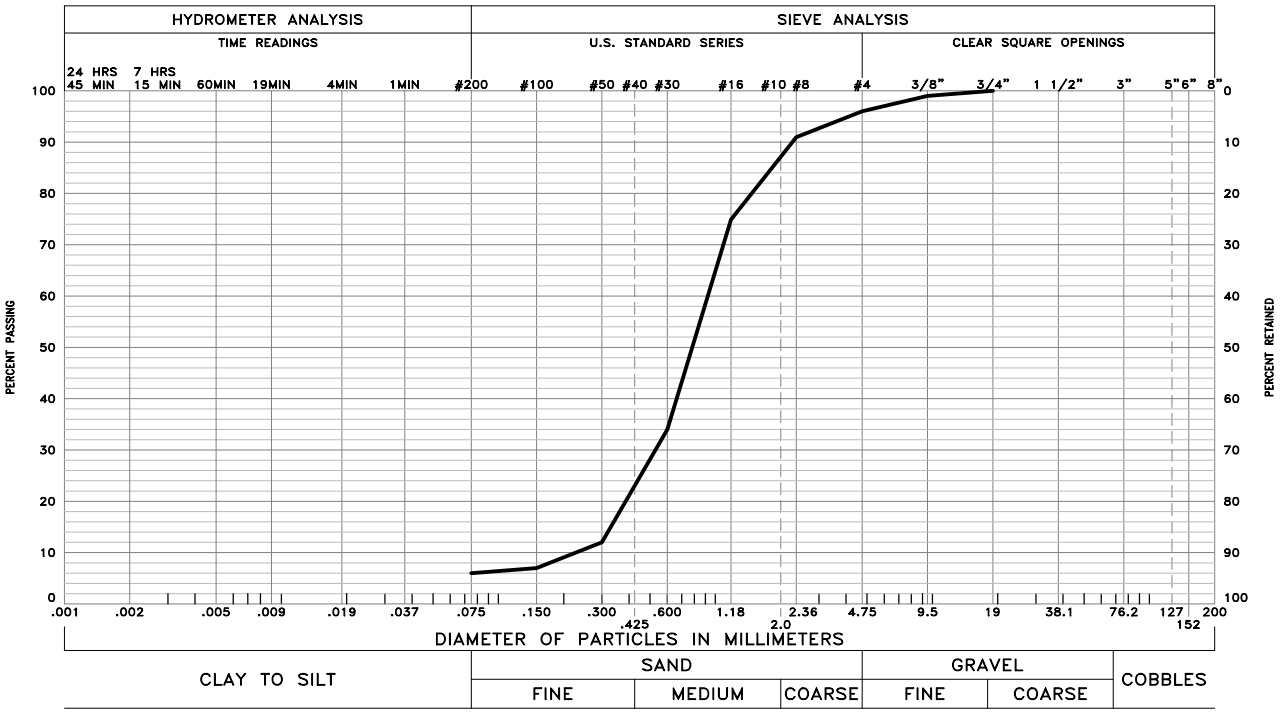


21−3−157	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULTS	Fig. 5





21-3-157	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULTS	Fig. 6



GRAVEL 3 % SAND 92 % SILT AND CLAY 5 %

LIQUID LIMIT – PLASTICITY INDEX –

SAMPLE OF: Poorly Graded Sand with Silt (SP–SM) FROM: Boring 1 @ 14'

These test results apply only to the samples which were tested. The testing report shall not be reproduced, except in full, without the written approval of Kumar & Associates, Inc. Sieve analysis testing is performed in accordance with ASTM D6913, ASTM D7928, ASTM C136 and/or ASTM D1140.

21–3–157	Kumar & Associates	GRADATION TEST RESULTS	Fig. 7

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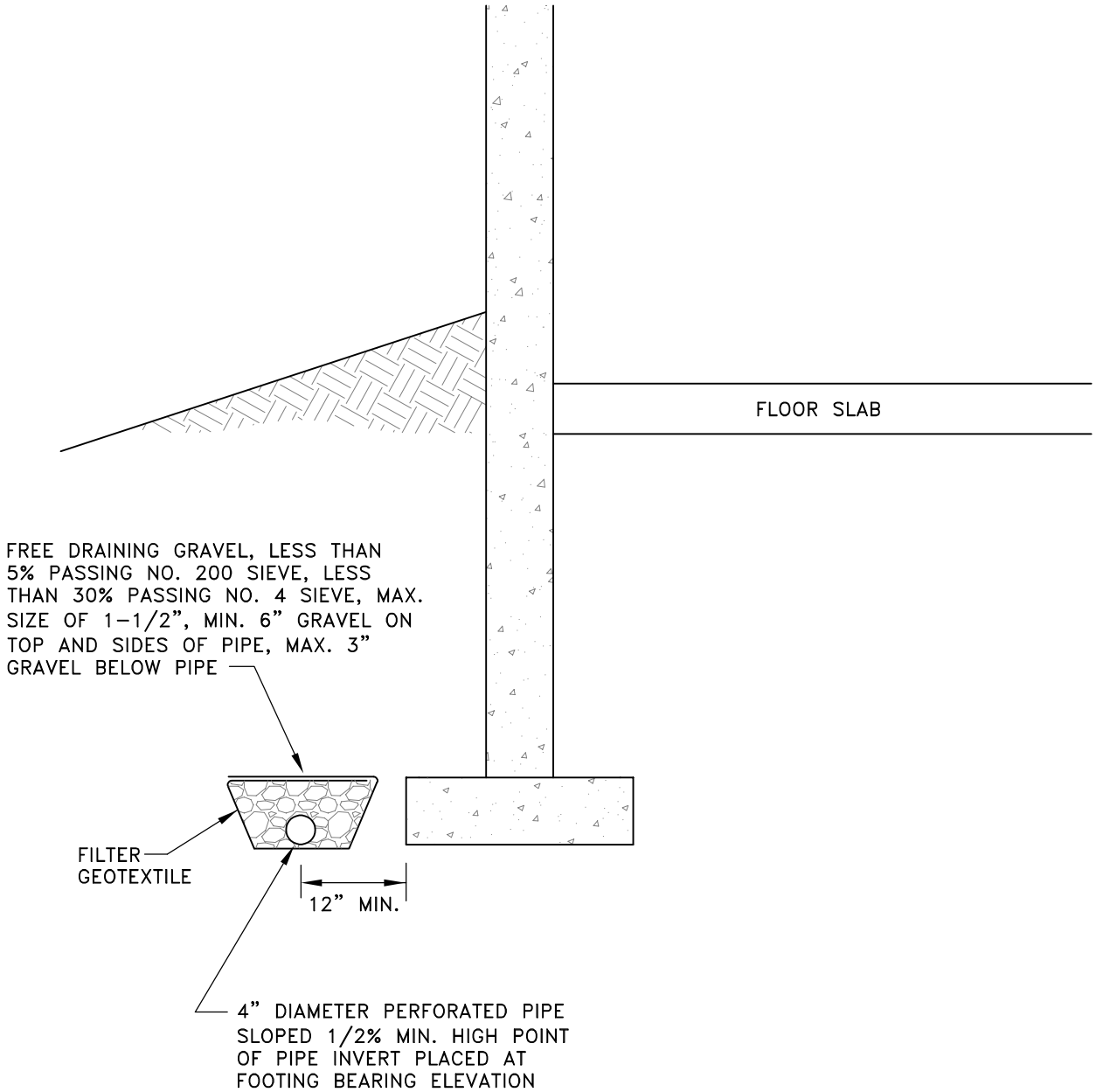
Table I
Summary of Laboratory Test Results

Project No.: 21-3-157
Project Name: Thornton Fire Station No. 7
Date Sampled: June 17, 2021
Date Received: June 18, 2021

Sample Location		Date Tested	Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation		Percent Passing No. 200 Sieve	Atterberg Limits		Water Soluble Sulfates (%)	AASHTO Classification (Group Index)	Soil or Bedrock Type
Boring	Depth (Feet)				Gravel (%)	Sand (%)		Liquid Limit (%)	Plasticity (%)			
1	4	6/21/21	15.6	114.3			64	30	15	0.02	A-6 (7)	Sandy Lean Clay (CL)
1	14	6/21/21	2.6		3	92	5					Poorly Graded Sand with Silt (SP-SM)
2	4	6/21/21	19.4	103.6			86	34	17		A-6 (14)	Lean Clay (CL)
2	9	6/21/21	15.3	111.9			66	35	20		A-6 (11)	Sandy Lean Clay (CL)
3	4	6/21/21	17.5	108.6	1	32	67	32	17		A-6 (9)	Sandy Lean Clay (CL)
3	24	7/22/21	14.5	114.6								Claystone Bedrock
3	29	7/22/21	13.7	118.3								Claystone Bedrock
4	4	6/21/21	18.8	106.1			79	36	20		A-6 (14)	Lean Clay with Sand (CL)
P-1	2	6/21/21	18.8	108.0			85	40	21		A-6 (18)	Lean Clay with Sand (CL)
P-2	4	6/21/21	17.2	112.4			69	32	17		A-6 (9)	Sandy Lean Clay (CL)

APPENDIX A

TYPICAL PERIMETER DRAIN DETAIL



21-3-157	Kumar & Associates	TYPICAL PERIMETER DRAIN DETAIL	Fig. 6