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GEOTECHNICAL ENGINEERING STUDY
PROPOSED IMPROVEMENT PROJECT
WASTEWATER TREATMENT FACILITY
CHEROKEE METROPOLITAN DISTRICT
DRENNAN ROAD AND MILNE ROAD
EL PASO COUNTY, COLORADO

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SUMMARY

1. This report was originally issued on April 29, 2020 and has been subsequently revised to include additional discussion about the sampler barrel used and how the measured blow counts correlate to other samplers. The other information presented has not been modified from that included in our originally issued report.

2. Eleven exploratory borings were drilled for the proposed improvement project. Below a thin layer of topsoil, aeolian sands extended to the maximum depths explored ranging from 15 to 35 feet. The soils were similar but were categorized in two distinct groups. The majority of the soils were poorly to well graded sand with varied amounts of silt and included up to about 20 percent fines. The remainder of the soils were clayey to silty-clayey sands, and generally included more fines. In general, the aeolian sands found throughout the site were medium dense, dry to slightly moist, and light brown in color.

3. Groundwater was not encountered at the time of drilling or when measured several days later. Fluctuations in the groundwater level may occur with time, but groundwater is expected to be relatively deep in this area.

4. The improvements will include several structures constructed at different elevations throughout the site. We understand that mat foundations are the preferred foundation type for the proposed structures and will be suitable at this site based on the subsurface conditions encountered. Deep foundations may also be feasible but would require additional exploration at greater depths to suitably characterize the subsurface conditions. Due to the presence of non-cohesive sands with few fines, drilled piers may be difficult to construct, and would require casing and/or slurry construction methods.

5. Based on the subgrade conditions encountered and the estimated traffic information, we recommend the following pavement sections:

Area	Composite Asphalt over Base Course (inches)	Portland Cement Concrete over Base Course (inches)
Paved Drive Areas	5 over 6	7 over 6

We recommend delivery truck lanes, trash pickup areas, and any areas where truck turning movements are concentrated be paved with 7 inches of Portland cement concrete (PCC). The use of a flexible pavement in these areas could result in rutting/shoving of the pavement due to the concentrated wheel loads. To develop a stable subgrade, a minimum section of 6 inches of aggregate base course material should be placed below PCC pavement sections.

PURPOSE AND SCOPE OF STUDY

This report presents the results of a geotechnical engineering study for the proposed improvement project at the Cherokee Metropolitan District's Wastewater Treatment Facility. The study was conducted in general accordance with the scope of work in our Proposal No. C20-113R, dated January 30, 2020 for the purpose of providing recommendations for foundations, grading, and other geotechnical considerations related to the proposed structures, as well as pavement thickness design and infiltration characteristics based on percolation testing.

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 the proposed construction are included herein.

PROPOSED CONSTRUCTION

We understand that this project will consist of the construction of several new structures, including a new headworks building and grit chamber, four new membrane tanks, a dissolved solids removal facility, and a new splitter box. Access roads will be constructed near the proposed and existing structures. Additionally, five new evaporation ponds will be constructed in the undeveloped area surrounding the facility. The new ponds will have a combined area of just under 70 acres. The finished grade will be similar to that existing, but significant excavation is expected to accommodate some of the structures with deeper bearing elevations. If the proposed construction, locations, grades, or conditions are significantly different than those described above or depicted in this report, we should be contacted to reevaluate our recommendations.

SITE CONDITIONS

The project is located at the existing operational treatment facility located near the intersection of Drennan Road and Milne Road in El Paso County, Colorado. This is a sparsely populated area about 12 miles east of the Colorado Springs Airport, and the surrounding land includes acreage lots and undeveloped land. The site is relatively flat with some gently rolling hills. A Shallow gulley is located within the northwest portion of the site and flows southeast, then south towards and parallel to the outside fence along the west edge of the existing facility. A detention pond is located on the south side of the site in the location of percolation test holes P-1 and P-2. Both areas were dry at the time of our study. Vegetation consists of field grasses and weeds.

SUBSURFACE CONDITIONS

The field exploration for the project was conducted on March 31, 2020. Eleven exploratory borings were drilled at the locations shown on Fig. 1 to explore subsurface conditions and estimate groundwater levels. Approximate locations of the exploratory borings were determined by pacing from the site features. Logs of the exploratory borings are presented on Figs. 2 and 3, and a legend and notes are presented on Fig 3.

The borings were advanced through the overburden soils and underlying bedrock with 4-inch diameter continuous flight augers. The borings were logged by a representative of Kumar & Associates, Inc.

Samples of the soils and bedrock materials were taken with a 2-inch I.D. California Barrel Sampler. The sampler was driven into the various strata with blows from a 140-pound hammer falling 30 inches. This test is similar to the standard penetration test described by ASTM Method D 1586. Penetration resistance values, when properly evaluated, indicate the relative density or consistency of the soils. Depths at which the samples were taken and the penetration resistance values are shown on the Logs of Exploratory Borings, Figs. 2 and 3.

The Standard Test Method for Standard Penetration Test (SPT) and Split Barrel Sampling of Soils described by ASTM relies on the use of a standard split barrel spoon sampler (SPT sampler) which has a sampler shoe with nominal inner and outer diameters of $1^{3}/_{8}$ inches and 2 inches respectively. The California barrel sampler used in our investigation (and commonly used in the Front Range area) has an inner and outer sampler shoe diameter of 2 inches and 2.5 inches, respectively.

The interaction between the soil or bedrock sampled and the impact energy applied is complex, and there are several variables that can affect the number of blows required to achieve a predetermined depth of penetration. One major factor affecting this is the cross-sectional diameter of the sampler shoe being used. A commonly used input energy correction equation originally proposed by Don Burmister in the 1948 ASTM Proceedings relates non-standard sampler diameters to SPT samplers. This equation discounts the increase in skin friction accompanying the increase in sampler barrel diameter and increasing sampler depth, and instead only considers the energy input, and the inner and outer diameters of the sampler barrel used. Burmister's Correlation is as follows:

$$N^* = (N_R) \cdot \left(\frac{W \cdot H}{4200}\right) \cdot \left(\frac{2.109}{(D_O)^2 - (D_i)^2}\right)$$

Where

N_R = Recorded blow count
W = Weight of hammer used (pounds)
H = Drop height (inches)
D_O = Outer diameter of sampler barrel used
D_i = Inner diameter of sampler barrel used
N^*= The blow count reported as the equivalent SPT value

Although the efficiency of energy transfer between the hammer and the soil or bedrock being sampled is variable, a large number of correlations for soil parameters are based on a "standard" energy ratio of 60 percent (N_{60}), the historical average ratio for a manual hammer. To correlate the blow counts obtained from differing hammer systems, it is a common industry practice to use the following relationship:

$$N_{60} = N_R \cdot \left(\frac{E_A}{E_{60}}\right)$$

Where

N_R = Recorded blow count
E_A = Actual energy transfer
E_{60} = Referenced energy transfer for manual hammers
N_{60} = Equivalent blow count for 60 percent energy transfer from hammer

Based on information published by the U.S. Bureau of Reclamation and the Central Mine Equipment Company (CME), as well as measurements conducted by geotechnical drill rig operator/owners in the Front Range area, an actual energy transfer in the range of 65 to 85 percent is commonly obtained for CME auto-hammer systems such as the one used in our exploration. This value is heavily dependent on actual conditions at the time of use including the condition of the equipment being used, and the levelness of the setup. For the purpose of this discussion, an assumed energy transfer of 70 percent has been considered.

Because our field exploration utilized a standard weight hammer and drop height, the first term in the Burmister correlation becomes a value of 1. Based on our use of a California Barrel Sampler with an inner and outer cross-sectional diameter of 2 inches and 2.5 inches, respectively, the Burmister Correlation to an equivalent SPT sampler value is 0.94. Assuming an energy transfer of 70 percent, the correlation to an SPT sampler with an energy transfer of 60 percent is 1.17. Considering both of these factors together, a correlation of 1.1 is the calculated factor used to convert California barrel sample blow counts using a CME auto-hammer to SPT blow counts referencing a manual hammer (N_{60}). Because this correlation is very close to a value of 1, and in consideration of the other numerous factors that can affect blow count values, the blow counts presented herein assume a 1:1 correlation.

Measurements of the water level were made in the borings by lowering a weighted tape measure into the open hole shortly after completion of drilling and again several days after drilling.

The results of laboratory tests performed on selected samples obtained from the borings are shown to the right of the logs on Figs. 2 and 3, and are summarized in Table I. Samples obtained from the exploratory borings were visually classified in the laboratory by the project engineer and samples were selected for laboratory testing. Laboratory testing included index property tests, such as moisture content (ASTM D 2216), dry unit weight, grain size analysis (ASTM D 422), and liquid and plastic limits (ASTM D 4318). A swell-consolidation test (ASTM D 4546, Method B) was conducted on a selected sample of the near-surface soil to determine the compressibility or swell characteristics under loading and when submerged in water. Detailed results of the swell test can be found on the attached Fig. 4. The percentage of water-soluble sulfates was determined in general accordance with AASHTO T-290.

Below a thin layer of topsoil, aeolian sands extended to the maximum depths explored ranging from 15 to 35 feet. The soils were similar but were categorized in two distinct groups. The majority of the soils were poorly to well graded sand with varied amounts of silt and included up to about 20 percent fines. These soils were fine to coarse grained and included trace amounts of gravel in some areas. The remainder of the soils were clayey to silty-clayey sands, and generally included more fines. The aeolian sands found throughout the site were dry to slightly moist, and light brown in color. In general, the sands were medium dense, but ranged from loose to dense.

Groundwater was not encountered at the time of drilling or when measured several days later. Fluctuations in the groundwater level may occur with time, but groundwater is expected to be relatively deep in this area.

GEOTECHNICAL ENGINEERING CONSIDERATIONS

The proposed construction at this site includes several structures to be constructed at varied depths. The following information has been provided to us by the client:

Structure	Foundation Depth (ft.)
Membrane and Filtrate basins	4
TRF (Filter)	Shallow*
Splitter Box	13
Grit Basin	15 to 20
Headworks	5

*: The TRF Building is a pre-engineered structure, and is assumed to have a shallow foundation meeting the requirements for frost depth coverage.

In general, the soils encountered throughout the site will be suitable for the support of lightly to moderately loaded mat foundations. Deep foundations may also be feasible but would require additional exploration at greater depths to suitably characterize the subsurface conditions. Due to the presence of non-cohesive sands with few fines, drilled piers may be difficult to construct, and would require casing.

FOUNDATION RECOMMENDATIONS

<u>Footing Foundations:</u> The design and construction criteria presented below should be observed for a footing foundation system. The construction details should be considered when preparing project documents.

1. Footings constructed over the native soils prepared as specified in the “Site Grading and Earthwork” section can be designed for an allowable soil bearing pressure of 3,000 psf.

2. Based on our experience with similar projects, we estimate a low risk of total foundation settlement beyond about 1 inch for foundations designed and constructed in accordance with the recommendations provided herein.

3. Continuous footings should be at least 16 inches wide and isolated pads at least 24 inches wide.

4. Exterior footings and footings beneath unheated areas should be provided with adequate soil cover above their bearing elevation for frost protection. Placement of foundations at least 30 inches below the exterior grade is typically used in this area.

5. Continuous foundation walls should be reinforced top and bottom to span an unsupported length of at least 10 feet.

6. The lateral resistance of a spread footing placed on the on-site sands will be a combination of the sliding resistance of the footing on the foundation materials and passive earth pressure against the side of the footing. Resistance to sliding at the bottoms of the footings may be calculated based on an allowable coefficient of friction of 0.30. Passive pressure against the sides of the footings may be calculated using an allowable equivalent fluid unit weight of 180 pcf.

7. Earthwork recommendations for spread footing foundations is presented in the "Earthwork and Site Grading" section of this report.

8. A representative of the geotechnical engineer should observe all footing excavations prior to fill and concrete placement.

Mat Foundations: The bearing capacity presented in the "Footing Foundations" subsection above can be applied to mat foundations as well. Additional considerations for the use of mat foundations are presented below.

The rigidity of the mats will be dependent on their dimensions, column spacing, rigidity of the superstructure and the modulus of subgrade reaction of the supporting soils. We recommend mats be analyzed to determine if the rigidity assumption is valid.

If the mats cannot be considered rigid, the soil pressure distribution should be computed using a method which models the soil-structure interaction, such as the beam on an elastic foundation procedure. The following moduli of vertical subgrade reaction may be used for materials that have been prepared and/or placed in accordance with the specifications provided in the "Site Grading and Earthwork" section. The values presented are for a 1 ft. x 1 ft. square plate and should be corrected for the shape and size of the actual slab.

Material	Uncorrected modulus of Subgrade Reaction (pci)
Prepared Onsite Sand	125
Class 6 Aggregate Base Course (min 12-inch thick)	200

FLOOR SLABS

Based on the moisture-volume change characteristics of the materials encountered, we believe slab-on-ground construction may be used at this site. It should be noted that some differential movement of the slab is likely to occur and may result in cracking and other distress even if the movement is within the range considered normal for this type of construction. If this risk cannot be tolerated, the use of structural slabs constructed over a crawlspace should be considered.

To reduce the effects of some differential movement, the following measures should be taken.

1. The native or fill materials below the slab should be prepared in accordance to the criteria presented in the “Site Grading and Earthwork” section of the report.

2. Floor slabs should be separated from all bearing walls and columns with expansion joints which allow unrestrained vertical movement.

3. Floor slabs should not extend beneath exterior doors or over foundation grade beams, unless saw cut at the beam after construction.

4. Floor slab control joints should be used to reduce damage due to shrinkage cracking. The appropriate joint spacing is dependent on slab thickness, concrete aggregate size and slump, and should be consistent with recognized guidelines such as those of the Portland Cement Association (PCA) or American Concrete Institute (ACI). The joint spacing and any requirements for slab reinforcement should be established by the designer based on experience and the intended slab use.

5. If moisture-sensitive floor coverings will be used, mitigation of moisture penetration into the slabs, such as by use of a vapor barrier, may be required. If an impervious vapor barrier membrane is used, special precautions will be required to reduce potential differential curing problems which could cause the slabs to warp. Section 302.1R of the ACI Manual of Concrete Practice addresses this topic.

6. All plumbing lines should be tested before operation. Where plumbing lines or other slab protrusions enter through the floor, a positive bond break should be provided. Flexible connections should be provided for slab-bearing mechanical equipment.

The precautions and recommendations itemized above will not prevent the movement of floor slabs if the underlying materials are subjected to moisture increases, and experience settlement. However, the precautions should reduce the damage if such movement occurs.

FOUNDATION AND RETAINING WALLS

Foundation walls and retaining structures should be designed for the lateral pressure generated by the backfill, which is a function of the degree of rigidity of the retaining structure and the type of backfill material used. Cantilevered retaining structures that can deflect sufficiently to mobilize the active earth pressure condition may be designed using the active equivalent fluid pressure (EFP) presented below. Retaining structures that are not expected to deflect should be designed using the at-rest EFP.

Condition	Soil Type	Equivalent Fluid Pressure (pcf)	
		Active	At-rest
Unsubmerged	Granular	40	60
Submerged	Granular	85	96

All foundation and retaining structures should be designed for appropriate hydrostatic and surcharge pressures such as adjacent footings, traffic, construction materials and equipment. The unsubmerged pressures recommended above assume drained conditions behind the walls. Both conditions assume 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. If no underdrain system is installed, and the structure is waterproofed, the values presented for submerged soils should be used to determine the EFP. Retaining walls may be designed using the values presented for unsubmerged soils if adequate drainage is provided to prevent the buildup of hydrostatic pressure. This can be accomplished using an underdrain or weep holes.

Care should be taken not to over-compact the backfill or use large equipment near walls, since this could cause excessive lateral pressure on the wall. Some settlement of deep foundation wall backfill should be expected, even if the material is placed correctly, and could result in distress to structures, pavements, or flatwork constructed on the backfill.

The lateral resistance of foundation or retaining wall footings can be found in Item 6 of the “Footing Foundations” subsection within the “Foundations” section above.

SEISMIC DESIGN CRITERIA

The generalized subsurface profile encountered within the proposed building footprint consisted of medium dense granular soils extending at least 30 feet deep. Based on the estimated weighted average shear wave velocity within the upper 100 feet, a Site Class D has been assigned per the 2015 International Building Code (IBC). The Colorado Front Range is located in an area of relatively low seismic activity, and soil liquefaction is not a design consideration. Using the USGS National Earthquake Hazard Reduction Program online database, the following probabilistic ground motion values are reported for the project site.

Spectral Acceleration	Acceleration Coefficient 2 percent in 50 Years
S_S (0.2 Sec. Period)	0.158
S_1 (1.0 Sec. Period)	0.057

The design of the site facilities should conform to the seismic requirements specified in the local building code.

WATER SOLUBLE SULFATES

The concentration of water-soluble sulfates measured in samples obtained from the exploratory borings was less than 0.01 percent. Water soluble sulfate concentrations of 0.10 percent or less represent a Class 0 severity of exposure to sulfate attack on concrete exposed to these materials. The degree of attack is based on a range of Class 0 to Class 3 severity of exposure as presented in ACI 201. Based on this information and our experience with similar materials, we believe sulfate resistant cement will not be required for concrete exposed to the onsite soils. Concrete containing Type I/II cement is commonly used in this area, should be considered for this project due to its ready availability.

SITE GRADING AND EARTHWORK

Excavated areas should be backfilled with suitable materials. Existing soils, with the exception of any deleterious material such as construction debris or concentrations of organic matter can be reused, provided they are adequately moisture conditioned and compacted.

Fill Material Specifications: The following material specifications are presented for fills on the project site.

1. *Non-expansive Fill*: Imported non-expansive fill material should have a maximum of 50% passing the No. 200 sieve, a maximum Liquid Limit of 30, and a maximum Plasticity Index

of 10. Imported or on-site fill materials not meeting the above liquid limit and plasticity index criteria may be acceptable provided the maximum percentage passing the No. 200 sieve specified above, and the swell criteria outlined in Item 5 below are satisfied, though it is unlikely that swelling soils will be encountered at this site.

2. *Pavement Areas:* Same as Item No. 1 above.

3. *Pipe Bedding Material*: Pipe bedding material should be a free draining, coarse-grained sand and/or fine gravel. The on-site natural granular alluvial sands anticipated to be available for use as fill include materials with relatively high fines content that may not be suitable for pipe bedding.

4. *Utility Trench Backfill*: Materials excavated from the utility trenches may be used for trench backfill above the pipe zone fill provided they do not contain unsuitable material or particles larger than 4 inches.

5. *Material Suitability*: Unless otherwise defined herein, all fill material should be non- to low-swelling, free of vegetation, brush, sod, trash and debris, and other deleterious substances, and should not contain rocks or lumps having a diameter of more than 4 inches. A fill material should be considered non-expansive if the swell potential under a 200 psf surcharge pressure does not exceed ½ percent when a sample remolded to 95 percent of the standard Proctor (ASTM D 698) maximum dry density at optimum moisture content is wetted.

<u>Compaction Requirements</u>: A representative of the geotechnical engineer should observe fill placement operations on a full-time basis. We recommend the following minimum compaction criteria be used on the project.

Area	Percentage of Standard Proctor Maximum Dry Density (ASTM D 698)
Building Pads/Structure Footprints	98%
Pavement Areas/Exterior Flatwork	95%
Foundation Wall Backfill	95%
Landscape and Other Misc. Overlot Fill Areas	95%
Compaction of fill materials should be achieved at a moisture content between +/-2% of optimum for granular soils.	

EXCAVATION CONSIDERATIONS

In our opinion, the soils encountered in the exploratory borings drilled for this study can be excavated with conventional construction equipment. All excavations should be made in accordance with OSHA, state and local requirements. The contractor should follow appropriate safety precautions. In accordance with OSHA guidelines, the soils will classify as Type B and C materials, but should be considered as Type C due to their variability. Per OSHA criteria, unless excavations are shored, temporary unretained excavations in Type C materials should have slopes no steeper than 1½:1 (H:V). Flatter slopes will be required where groundwater seepage is encountered. OSHA regulations require that excavations greater than 20 feet in depth be designed by a professional engineer. Actual soil conditions should be verified at the time of excavation. If soils different from those indicated in this report are encountered, the OSHA soil type may vary, and the required cut slopes may need to be adjusted. The contractor's "competent person" should make decisions regarding excavation slopes.

Based on the subsurface conditions encountered, we do not expect dewatering to be necessary for the excavation depths anticipated.

SURFACE DRAINAGE

Proper surface drainage is very important for acceptable performance of structures and pavements 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 the building. 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, slab, and pavement subgrades should be avoided during construction.

2. Exterior backfill should be compacted according the specifications listed in the "Site Grading and Earthwork" Section of this report.

3. The ground surface surrounding the exterior of the building should be sloped to drain away from the foundation in all directions. We recommend a minimum slope of 6 inches in the first 10 feet in unpaved areas. Site drainage beyond the 10-foot zone should be designed to promote runoff and reduce water 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.

4. Ponding of water should not be allowed on backfill material or within 10 feet of the foundations, whichever is greater.

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

6. Low intensity drip irrigation, and limited vegetation should be used in the area around the structures, which should result in a low risk of excessive settlement related to wetting. The risk could be further reduced by eliminating landscape irrigation within about 15 to 20 feet of the building and limiting irrigation elsewhere on site.

PAVEMENT DESIGN

Subgrade Materials: The materials tested near the surface in the area of the proposed drive lane extension include A-1-b, A-2-4, and A-4 soils with a maximum group index of 0 in accordance with the American Association of State Highway Transportation Officials (AASHTO) classification. This material is generally rated as good to fair for subgrade support. Based on the laboratory test results and our experience with similar subgrade soils, an R-value of 20 and a resilient modulus of 4,940 psi were assumed for design of flexible pavements, and a corrected subgrade modulus of 150 pci was assumed for rigid pavements.

Design Traffic: The traffic loading information for the planned pavement areas was not provided to us at the time of our study, but we have assumed traffic will consist of maintenance vehicles and occasional single-unit delivery trucks. Based on the relatively low anticipated traffic volume, a design Equivalent Daily 18-kip Load Application (EDLA) of 15, which represents a 20-year Equivalent 18-kip Single Axle Load ($ESAL_{18}$) of 109,500 was estimated. If it is determined that actual traffic is significantly different from that estimated, we should be contacted to reevaluate the pavement thickness design.

Pavement Sections: The recommended sections were determined using the 1993 AASHTO pavement design procedures. Based on the subgrade conditions encountered and the estimated traffic information, we recommend the following pavement sections:

Area	Composite Asphalt over Base Course (inches)	Portland Cement Concrete over Base Course (inches)
Paved Drive Areas	5 over 6	7 over 6

We recommend delivery truck lanes, trash pickup areas, and any areas where truck turning movements are concentrated be paved with 7 inches of Portland cement concrete (PCC). The use of a flexible pavement in these areas could result in rutting/shoving of the pavement due to the concentrated wheel loads. To develop a stable subgrade, a minimum section of 6 inches of aggregate base course material should be placed below PCC pavement sections.

Subgrade Preparation: Just prior to paving, the exposed subgrade soils should be thoroughly scarified and well mixed to a depth of at least 12 inches, moisture conditioned, and compacted according to the specifications in the “Site Grading and Earthwork” Section of this report.

Proof Roll: Before paving, the subgrade should be proof rolled with a heavily loaded, pneumatic-tired vehicle. The vehicle should have a gross vehicle weight of at least 50,000 pounds with a loaded single axle weight of 18,000 pounds and a tire pressure of 100 psi. Areas which deform excessively under heavy wheel loads are not stable and should be removed and replaced with suitable material to achieve a stable subgrade prior to paving or placement of base course.

Subgrade Stabilization: Areas of unstable subgrade soils may be encountered during subgrade preparation for construction of the new pavement. Unstable foundation soils may be stabilized by over-excavation and replacement of the subgrade with suitable, imported, angular, well-graded materials. Other alternatives include the use of Type 2 biaxial geogrid reinforcement in combination with a layer of Class 6 aggregate base course. It has been our experience that the use of a crushed concrete product meeting a Class 6 gradation can perform well when trying to achieve stabilization. Specific stabilization requirements should be evaluated at the time of construction.

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

Maintenance: Periodic maintenance of paved areas is critical to achieve the design life of the pavement. Crack sealing should be performed annually as new cracks appear. Chip seals, fog seals, or slurry seals applied at approximate intervals of 3 to 5 years are usually necessary for asphalt. As conditions warrant, it may be necessary to perform patching and structural overlays at approximate 10-year intervals.

Pavement Materials: The asphalt should consist of a mixture of aggregate, filler and asphalt cement established by a qualified engineer. Asphalt cement with a performance grade of 58-28 is commonly used in this area and will likely be acceptable for this project. The use of an asphalt cement with a performance grade of 64-22 may also be considered but will be more prone to thermal cracking. The appropriate asphalt cement content should be determined by the mix design.

Aggregate Base Course should conform to the requirements of AASHTO M147 and to Section 703.03 of the Colorado Department of Transportation (CDOT) Standard Specifications for Road and Bridge Construction and should meet Class 6 grading and quality as defined by the CDOT specifications. The aggregate should have a minimum R-value of 78. The asphalt and aggregate base course also should meet any other applicable El Paso County asphalt paving specifications, including compaction standards for these materials.

Concrete pavement should meet the requirements of a Class P Mix, per Section 601 of the CDOT Standard Specifications, and should be based on a mix design established by a qualified engineer. The concrete should contain joints not greater than 12 to 15 feet on centers. The joints should be hand formed, sawed or formed by pre-molded filler. The joints should be at least 1/4 of the slab thickness. Expansion joints should be provided at the end of each construction sequence and between the concrete slab and adjacent structures. Expansion joints where required, should be filled with a ½ inch-thick asphalt impregnated fiber. Concrete should be cured by protecting against loss of moisture, rapid temperature changes and mechanical injury for at least three days after placement. The concrete sections presented above are assumed to be unreinforced. Providing dowels at construction joints would help reduce the risk of differential movements between panel sections. Providing a grid mat of deformed rebar or welded wire mesh within the concrete pavement section would assist in mitigating corner breaks and differential panel movements. If a rebar mat is installed, we recommend that the bars be placed in the lower half of the pavement section. A structural engineer should evaluate the placement and spacing of rebar if needed.

PERCOLATION TESTING

Percolation tests were performed at the south, west, and north ends of the site in accordance with a modified percolation test procedure whereby the infiltration rate is determined by applying a reduction factor to the measured percolation rate to account for the exfiltration occurring through the sides of the percolation hole. The measured percolation rates and associated infiltration rates of the soils tested are presented below. The locations of the percolation test holes are presented on Fig. 1. The measurements were made at 30-minute intervals. The values presented reflect the average of the last three readings made at each test hole.

Percolation Test ID	Average Percolation Rate (inches per hour)	Correction Factor	Infiltration Rate (inches per hour)	Hydrologic Soil Group
P-1	4.20	6.87	0.59	B
P-2	12.3	5.57	2.21	A
P-3	21.2	7.32	2.89	A
P-4	20.7	8.10	2.56	A
P-5	1.65	9.38	0.17	C
P-6	1.80	9.77	0.18	C

Based on the measured infiltration rates and the subsurface conditions encountered, a hydrologic soil group of “B” was determined for the soils represented by percolation test holes P-1 and P-2 on the south end of the site. A soil group of “A” was determined for soils represented by percolation test holes P-3 and P-4 on the west end of the site, and soils represented by percolation test holes P-5 and P-6 on the north end of the site were determined to have a soil group of “C”.

The soil groups were determined in accordance with the criteria listed in the United States Department of Agriculture’s *National Engineering Handbook, Part 630: Hydrology*. Group A and B soils have a low to moderately low runoff potential when thoroughly wet. Group C Soils have moderately high runoff potential.

Though no groundwater was encountered in this study, a long-term water level survey was not conducted, and the depth to high groundwater could potentially be shallow, particularly after precipitation events. If groundwater is anticipated within about 4 feet of the final grade (unlikely at this site), a different soil hydrologic group should be considered.

DESIGN AND SUPPORT SERVICES

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

We recommend that Kumar & Associates, Inc. be retained to provide observation and testing services to document that the requirements of the plans and specifications are being followed during construction, and to identify possible variations in subsurface conditions from those encountered in this study.

LIMITATIONS

This study has been conducted for exclusive use by the client for geotechnical related design and construction criteria for the project. 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 or as described in the report, 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.

The scope of services for this project does not include any environmental assessment of the site or identification of contaminated or hazardous materials or conditions, or the presence, prevention or possibility of mold or other biological contaminants (MOBC) developing in the future. If the owner is concerned about the potential for such contamination, other studies should be undertaken.

AFK/bj

LEGEND:

● EXPLORATORY BORING LOCATION.

▲ PERCOLATION TEST LOCATION.

150 0 150 300

APPROXIMATE SCALE-FEET

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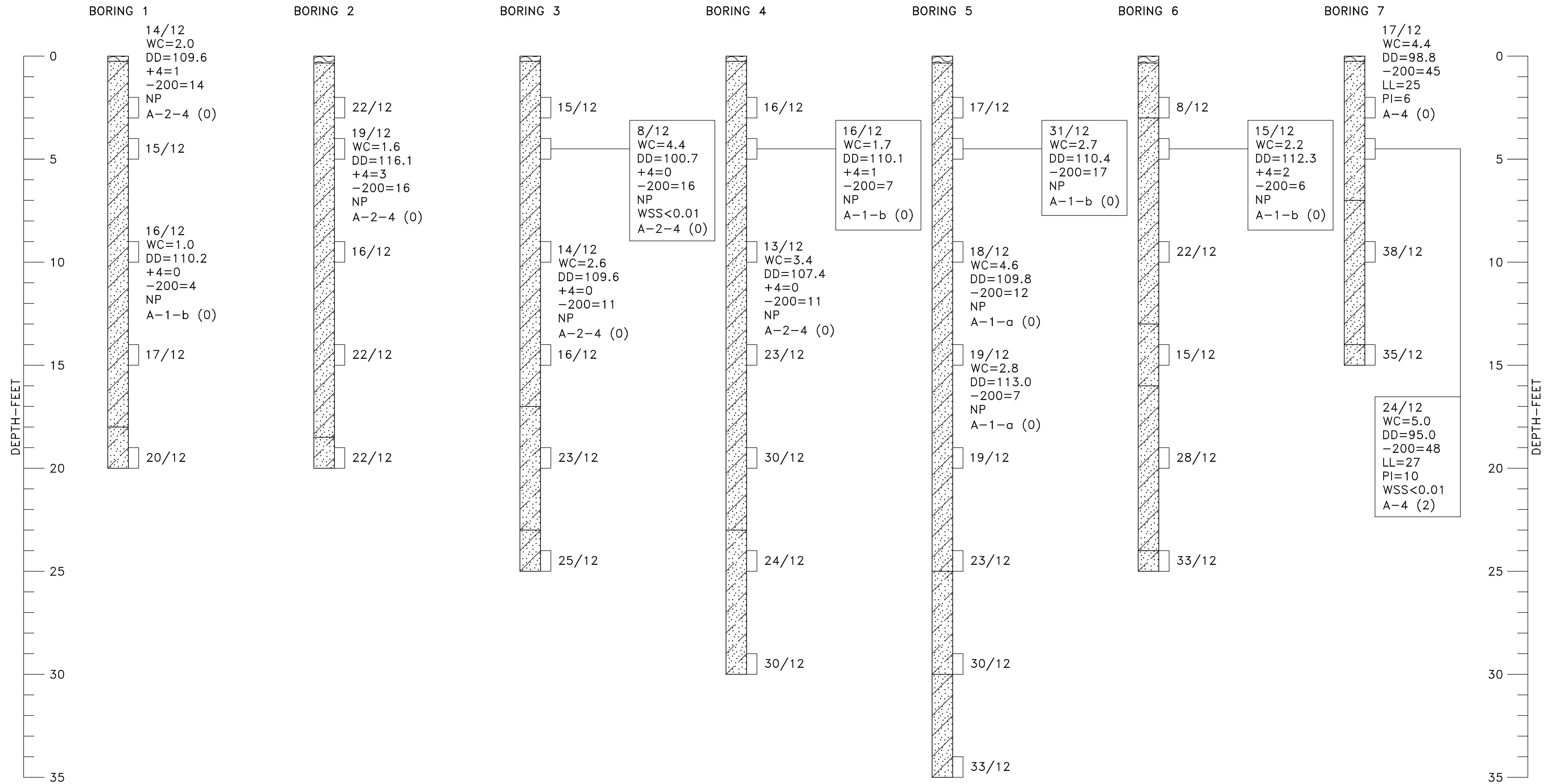
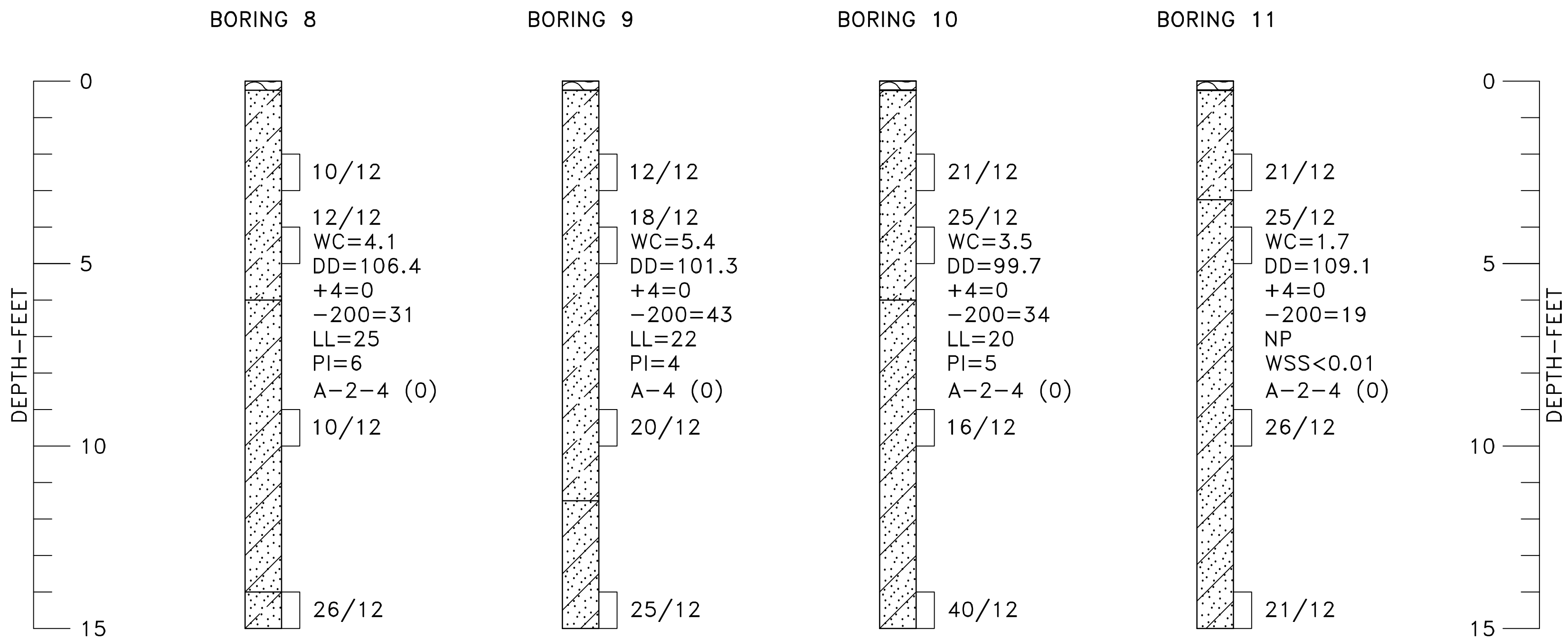
20-2-126	Kumar & Associates		LOCATION OF EXPLORATORY BORINGS	Fig. 1



20-2-126	Kumar & Associates		LOGS OF EXPLORATORY BORINGS	Fig. 2



LEGEND

TOPSOIL.

AEOLIAN SAND, INCLUDES CLAYEY AND SILTY−CLAYEY SAND (SC AND SC−SM), FINE TO MEDIUM GRAINED, LARGELY MEDIUM DENSE, BUT RANGING FROM LOOSE TO DENSE, DRY TO SLIGHTLY MOIST, LIGHT BROWN.

AEOLIAN SAND, INCLUDES POORLY TO WELL GRADED SAND WITH VARIED AMOUNTS OF SILT (SP, SW, SP−SM, SW−SM, SM), FINE TO COARSE GRAINED WITH TRACE AMOUNTS OF GRAVEL IN AREAS, SLIGHTLY MOIST, LARGELY MEDIUM DENSE, BUT RANGING FROM LOOSE TO DENSE, DRY TO SLIGHTLY MOIST, LIGHT BROWN.

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

14/12 DRIVE SAMPLE BLOW COUNT. INDICATES THAT 14 BLOWS OF A 140−POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE THE SAMPLER 12 INCHES.

NOTES

1. THE EXPLORATORY BORINGS WERE DRILLED ON MARCH 31, 2020 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.

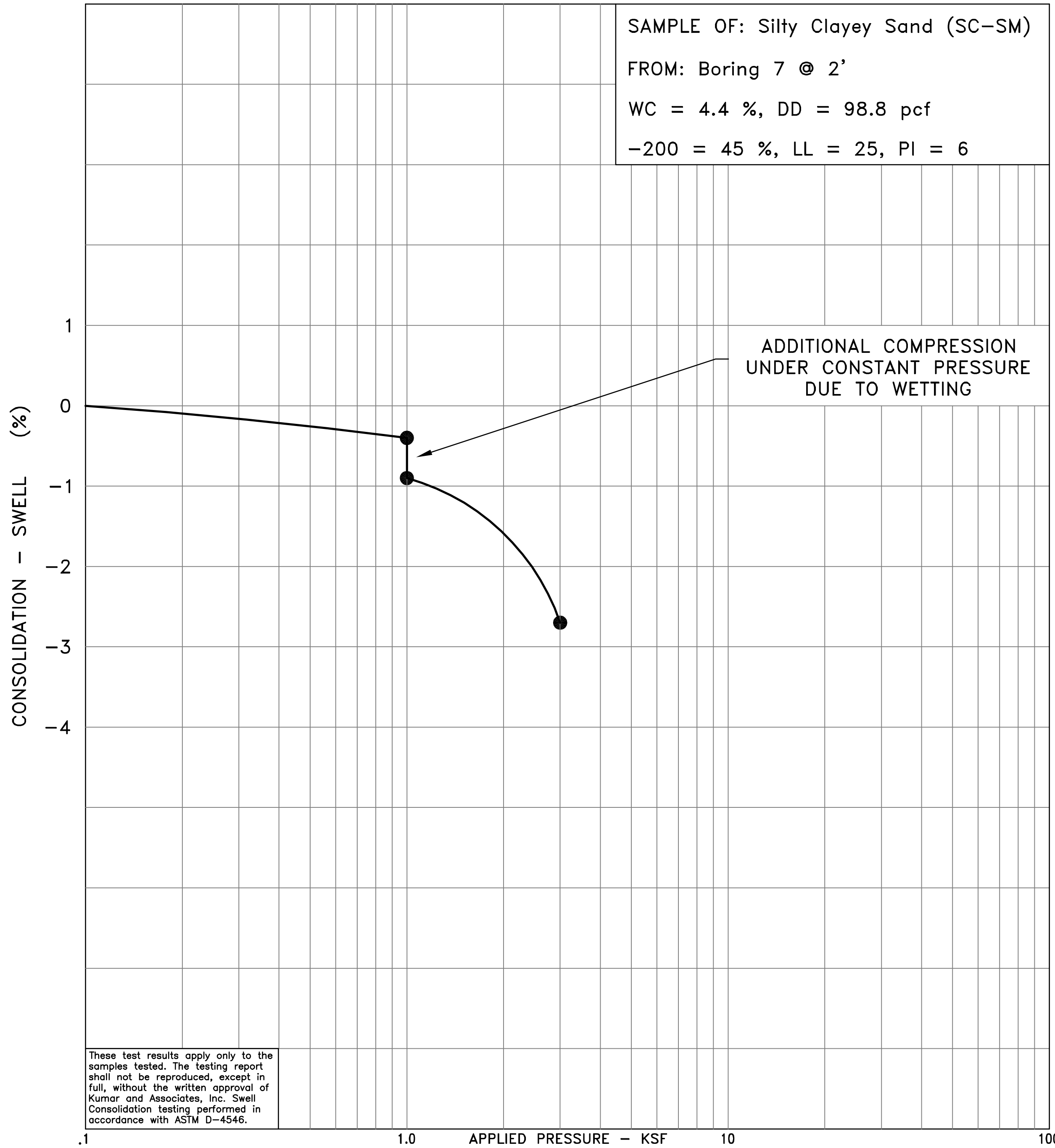
3. THE ELEVATIONS OF THE EXPLORATORY BORINGS WERE NOT MEASURED AND THE LOGS OF THE EXPLORATORY BORINGS ARE PLOTTED TO DEPTH.

4. THE EXPLORATORY BORING LOCATIONS SHOULD BE CONSIDERED ACCURATE ONLY TO THE DEGREE IMPLIED BY THE METHOD USED.

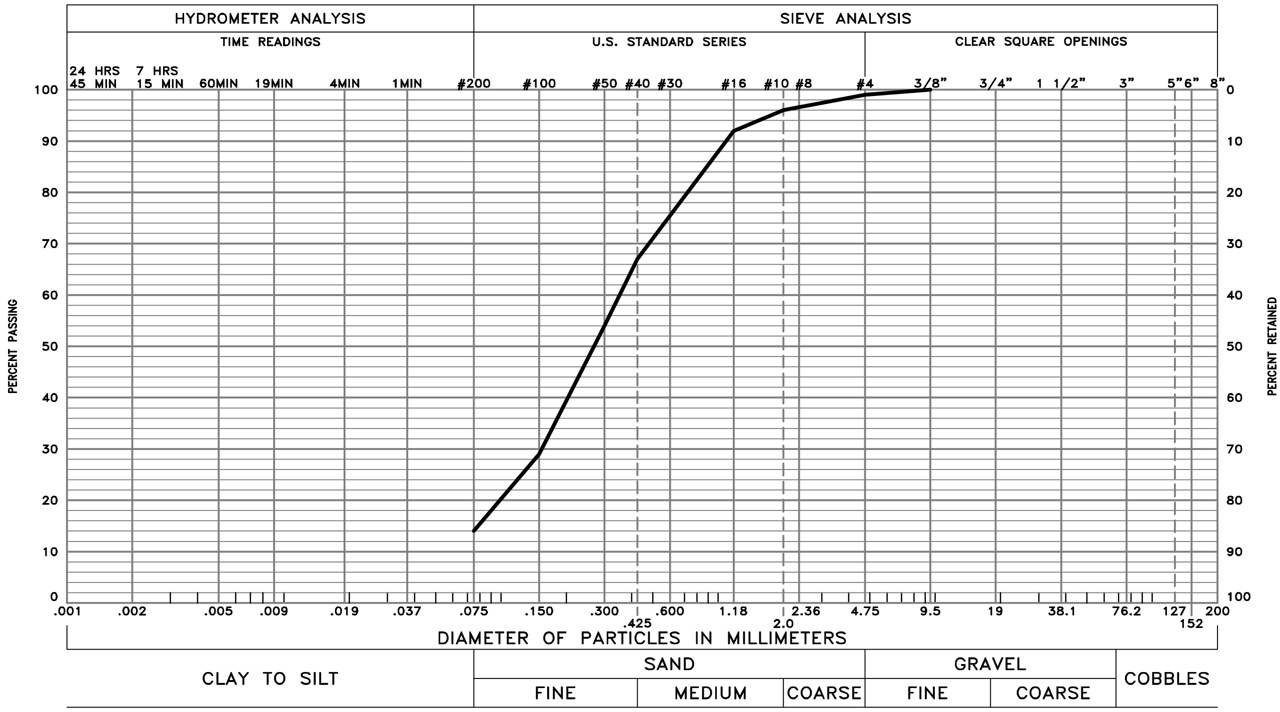
5. THE LINES BETWEEN MATERIALS SHOWN ON THE EXPLORATORY BORING LOGS REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN MATERIAL TYPES AND THE TRANSITIONS MAY BE GRADUAL.

6. GROUNDWATER WAS NOT ENCOUNTERED IN THE BORINGS AT THE TIME OF DRILLING OR WHEN CHECKED SEVERAL DAYS LATER.

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);
   NP = NON−PLASTIC (ASTM D 4318);
   WSS = WATER SOLUBLE SULFATES (%) (CP−L 2103);
   A−2−4 (0) = AASHTO CLASSIFICATION (GROUP INDEX) (AASHTO M145).



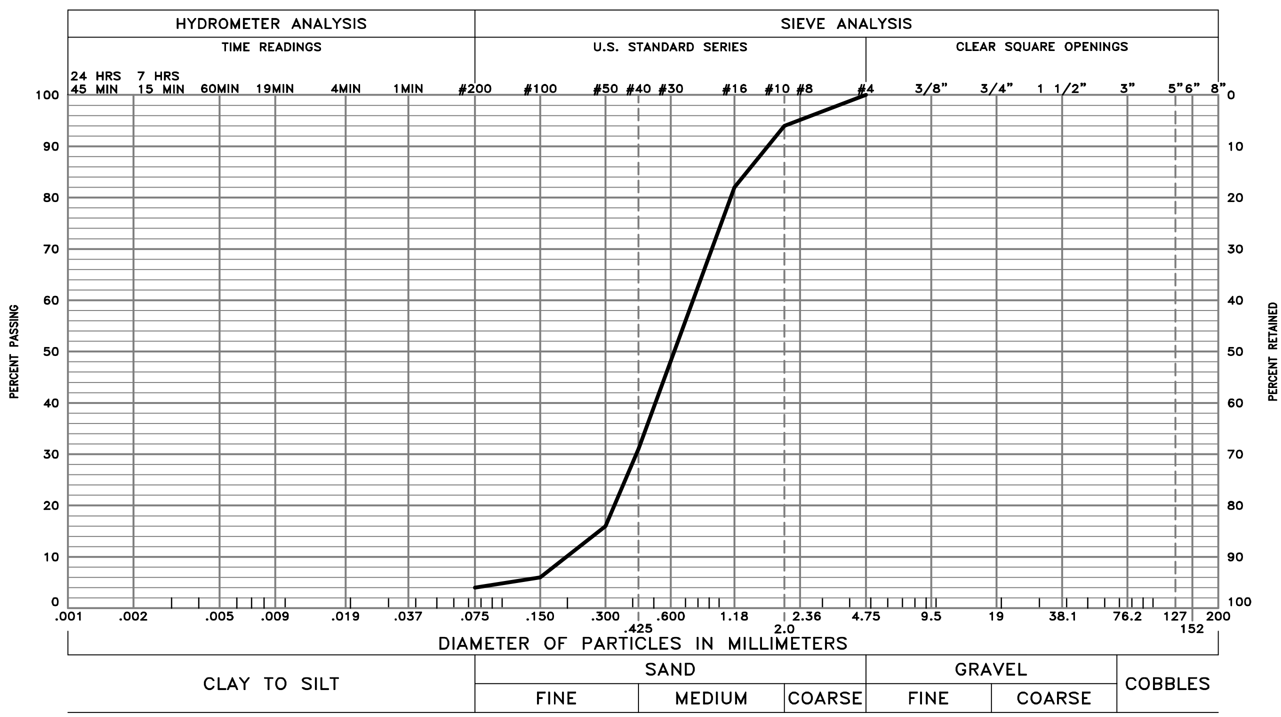
20–2–126	Kumar & Associates	SWELL–CONSOLIDATION TEST RESULT	Fig. 4



GRAVEL 1 % SAND 85 % SILT AND CLAY 14 %

LIQUID LIMIT PLASTICITY INDEX NP

SAMPLE OF: Silty Sand (SM) FROM: Boring 1 @ 2'



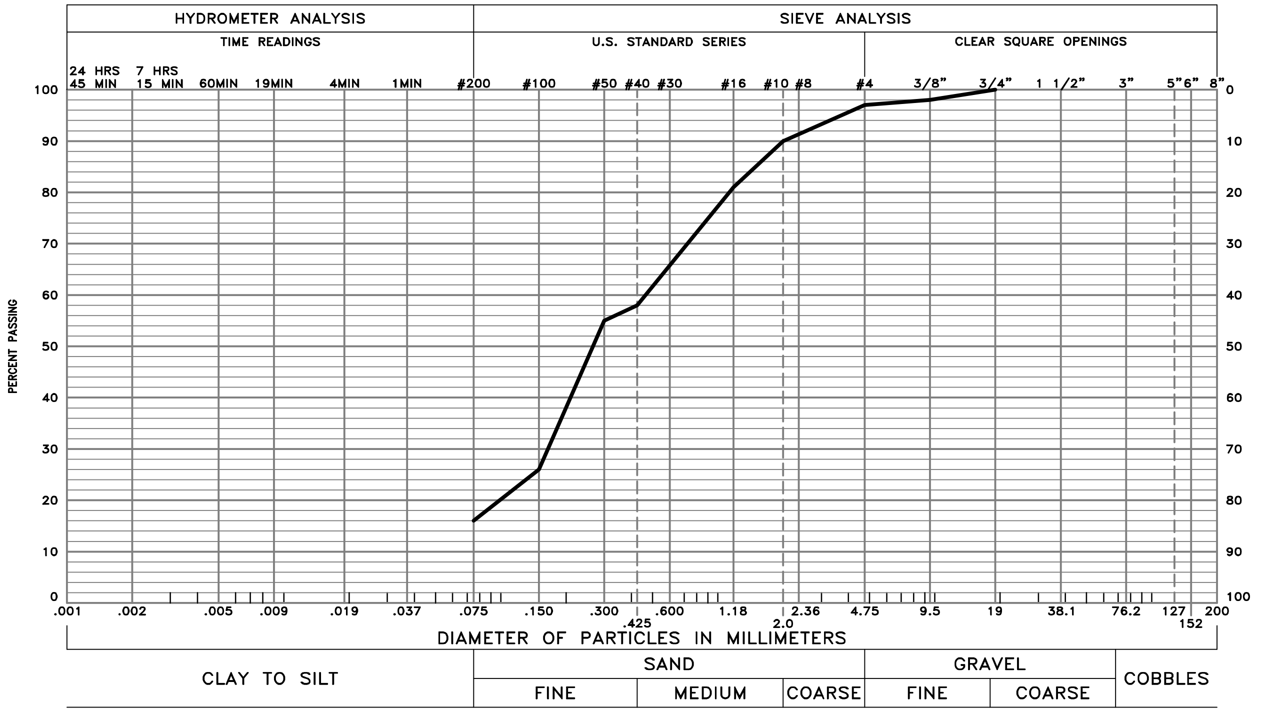
GRAVEL 0 % SAND 96 % SILT AND CLAY 4 %

LIQUID LIMIT PLASTICITY INDEX NP

SAMPLE OF: Poorly Graded Sand (SM) FROM: Boring 1 @ 9'

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.

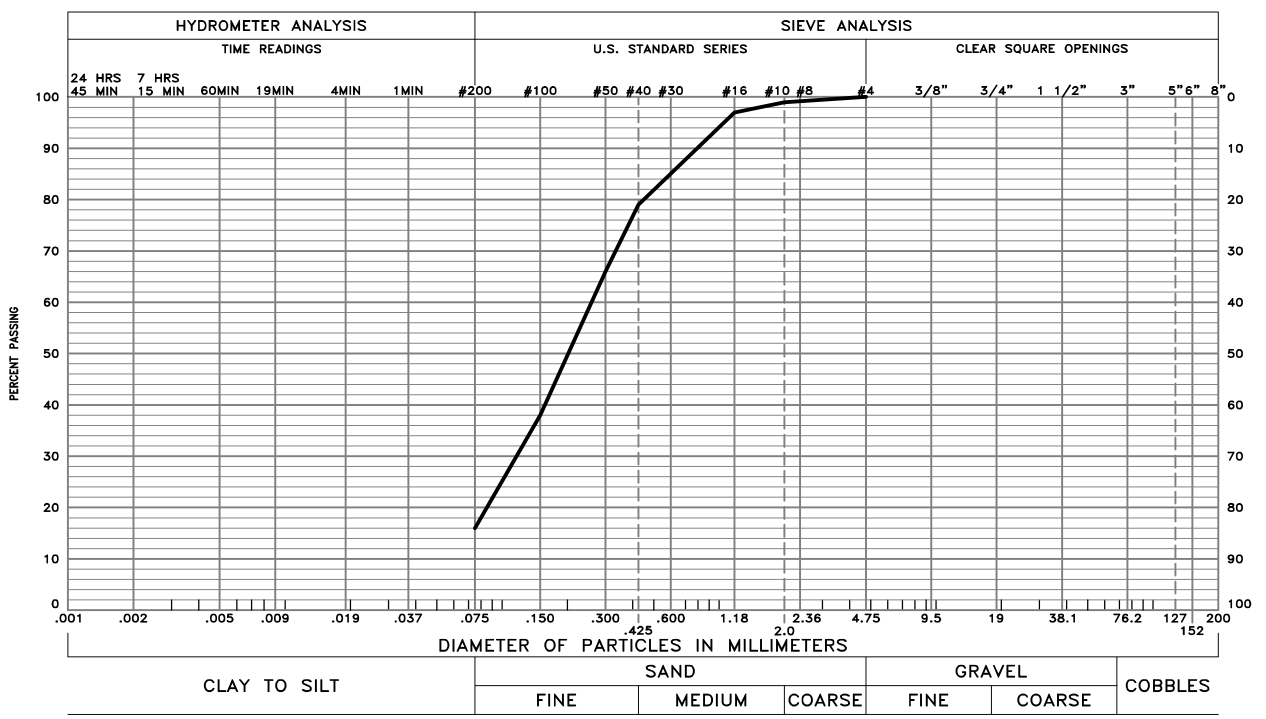
20-2-126	Kumar & Associates	GRADATION TEST RESULTS	Fig. 5



GRAVEL 3 % SAND 81 % SILT AND CLAY 16 %

LIQUID LIMIT PLASTICITY INDEX NP

SAMPLE OF: Silty Sand (SM) FROM: Boring 2 @ 4'

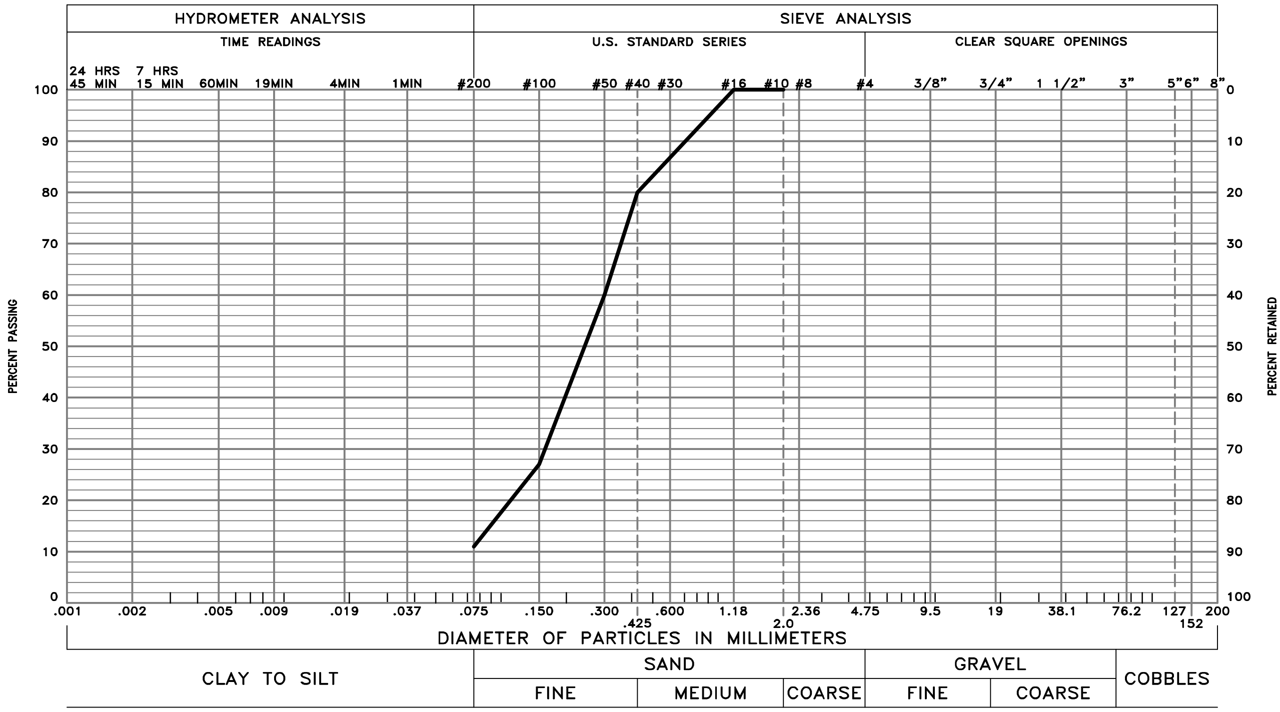


GRAVEL 0 % SAND 84 % SILT AND CLAY 16 %

LIQUID LIMIT PLASTICITY INDEX NP

SAMPLE OF: Silty Sand (SM) FROM: Boring 3 @ 4'

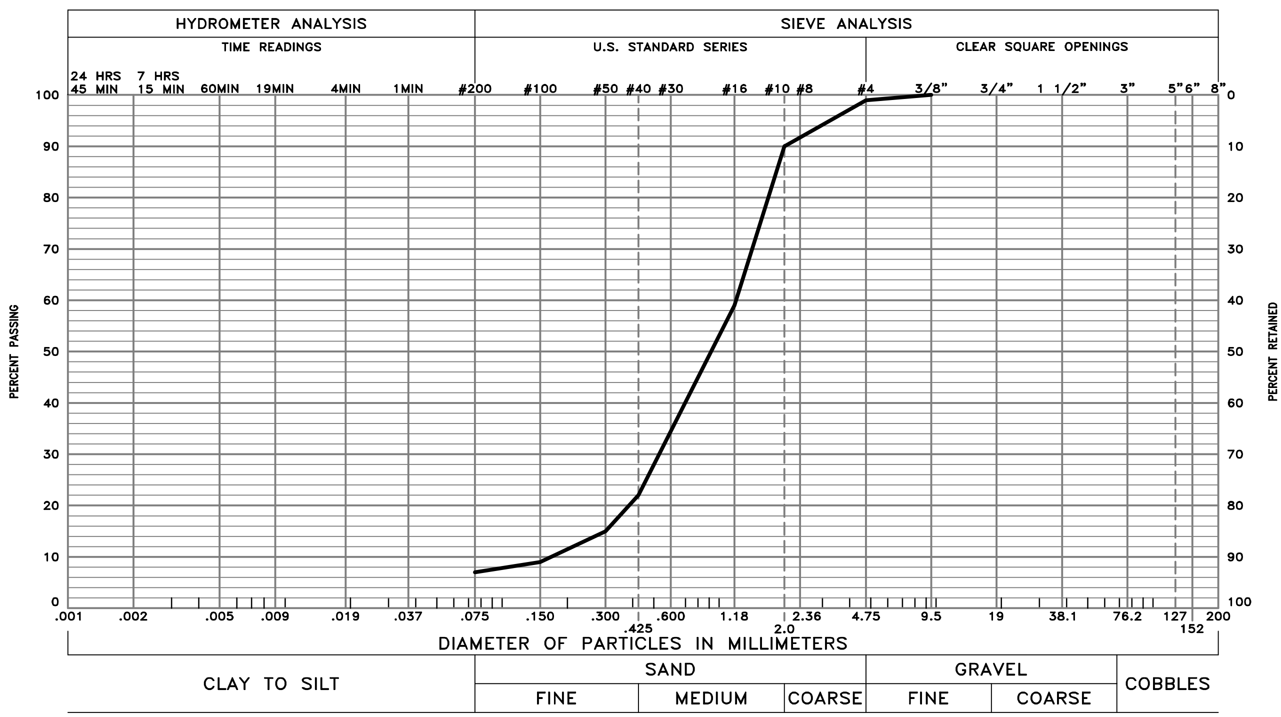
20-2-126	Kumar & Associates	GRADATION TEST RESULTS	Fig. 6



GRAVEL 0 % SAND 89 % SILT AND CLAY 11 %

LIQUID LIMIT PLASTICITY INDEX NP

SAMPLE OF: Poorly Graded Sand with Silt (SP-SM) FROM: Boring 3 @ 9'



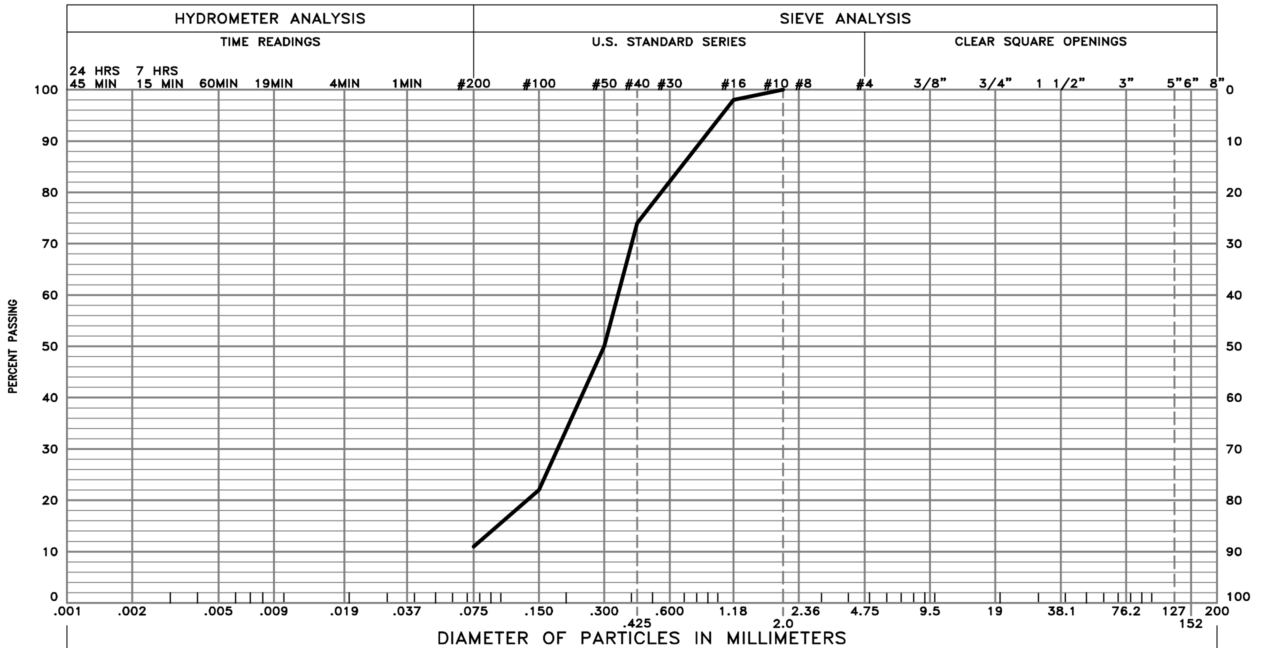
GRAVEL 1 % SAND 92 % SILT AND CLAY 7 %

LIQUID LIMIT PLASTICITY INDEX NP

SAMPLE OF: Poorly Graded Sand with Silt (SP-SM) FROM: Boring 4 @ 4'

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.

20-2-126	Kumar & Associates	GRADATION TEST RESULTS	Fig. 7

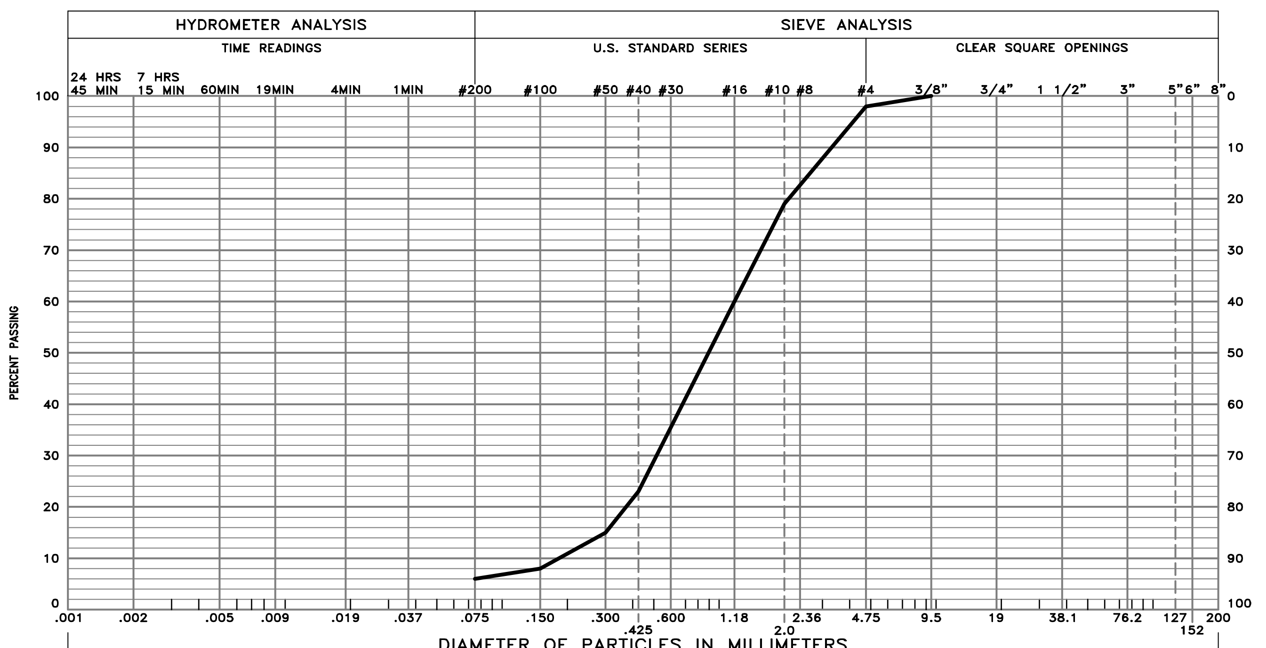


CLAY TO SILT	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

GRAVEL 0 % SAND 89 % SILT AND CLAY 11 %

LIQUID LIMIT PLASTICITY INDEX NP

SAMPLE OF: Poorly Graded Sand with Silt (SP-SM) FROM: Boring 4 @ 9'



CLAY TO SILT	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

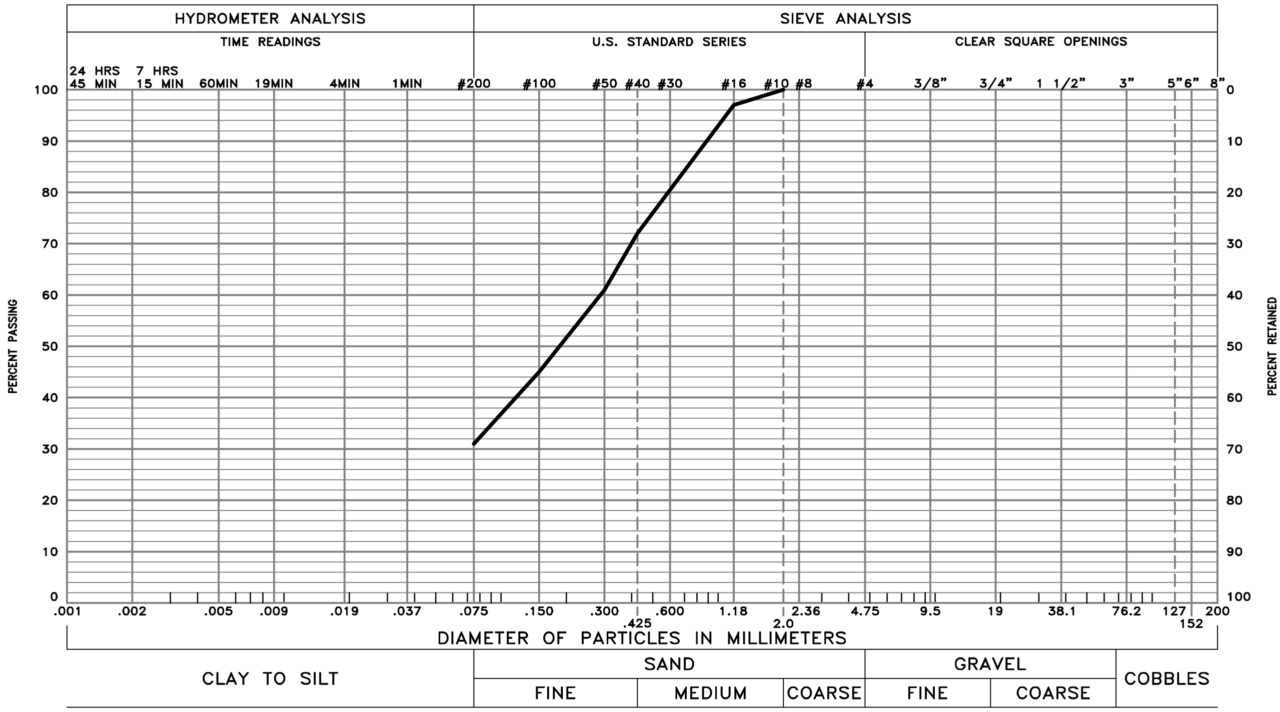
GRAVEL 2 % SAND 92 % SILT AND CLAY 6 %

LIQUID LIMIT PLASTICITY INDEX NP

SAMPLE OF: Well Graded Sand with Silt (SP-SM) FROM: Boring 6 @ 4'

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.

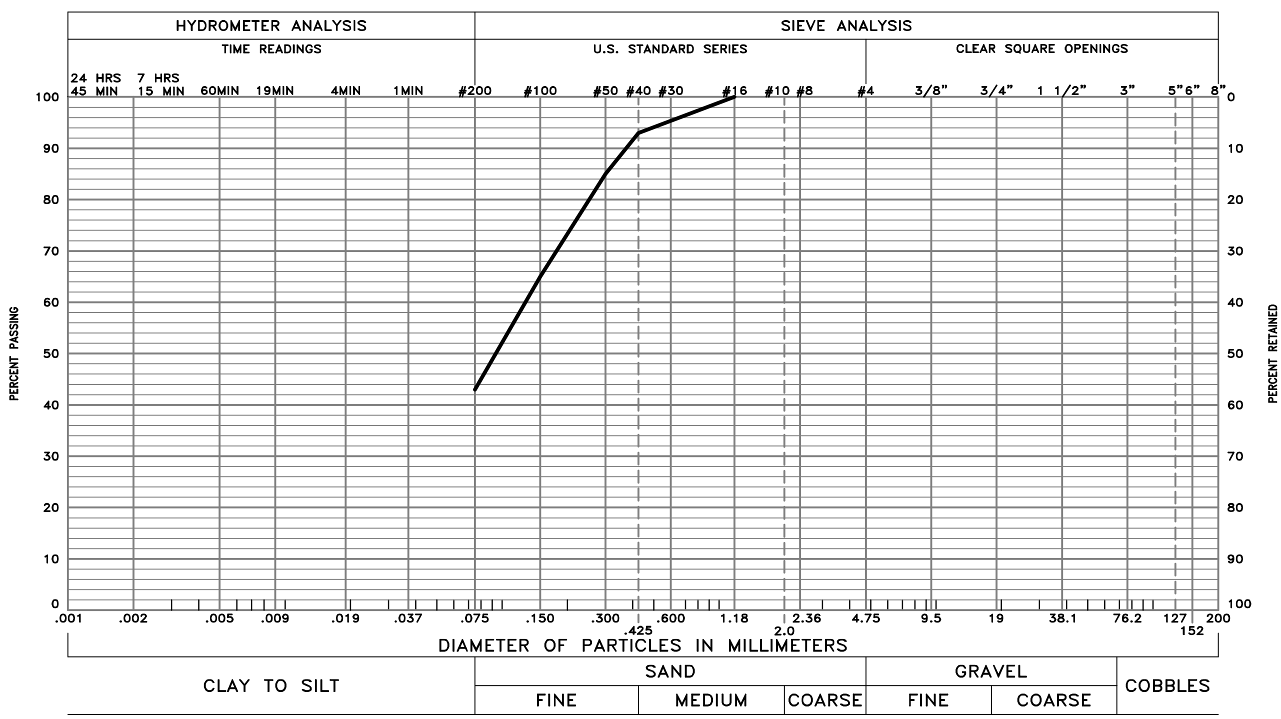
20-2-126	Kumar & Associates	GRADATION TEST RESULTS	Fig. 8



GRAVEL 0 % SAND 69 % SILT AND CLAY 31 %

LIQUID LIMIT 25 PLASTICITY INDEX 6

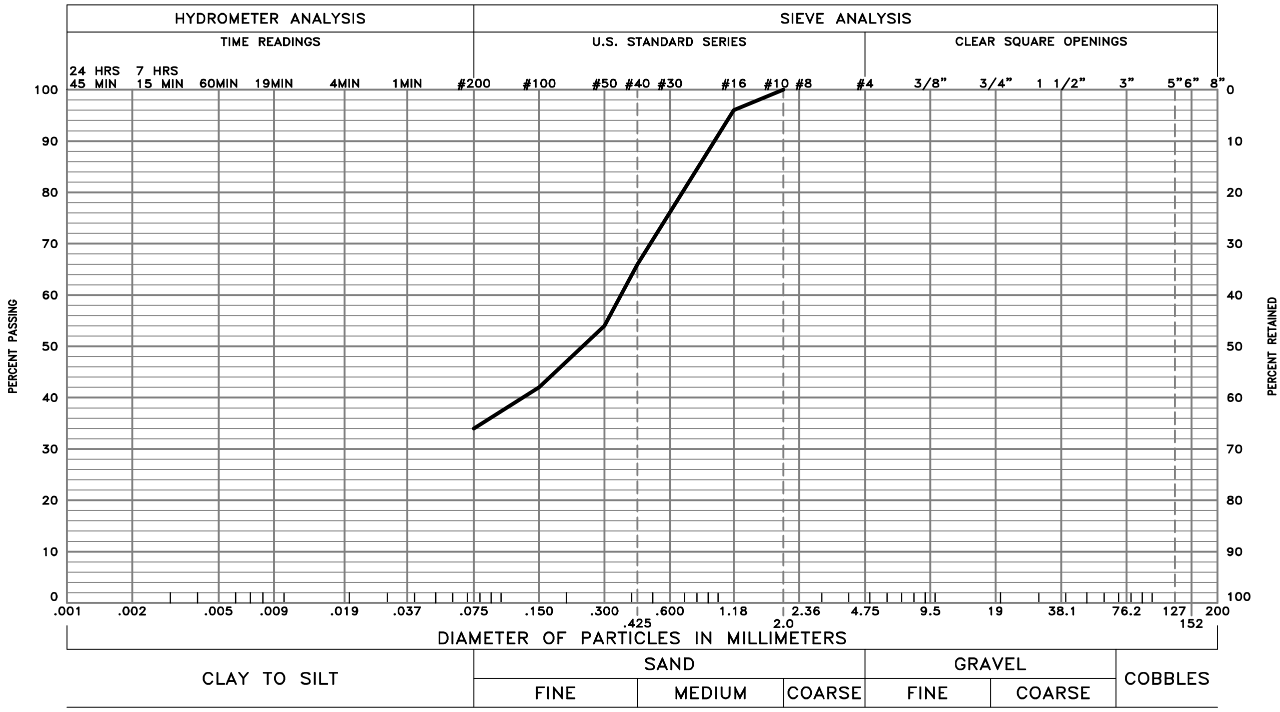
SAMPLE OF: Silty Clayey Sand (SC-SM) FROM: Boring 8 @ 4'



GRAVEL 0 % SAND 57 % SILT AND CLAY 43 %

LIQUID LIMIT 22 PLASTICITY INDEX 4

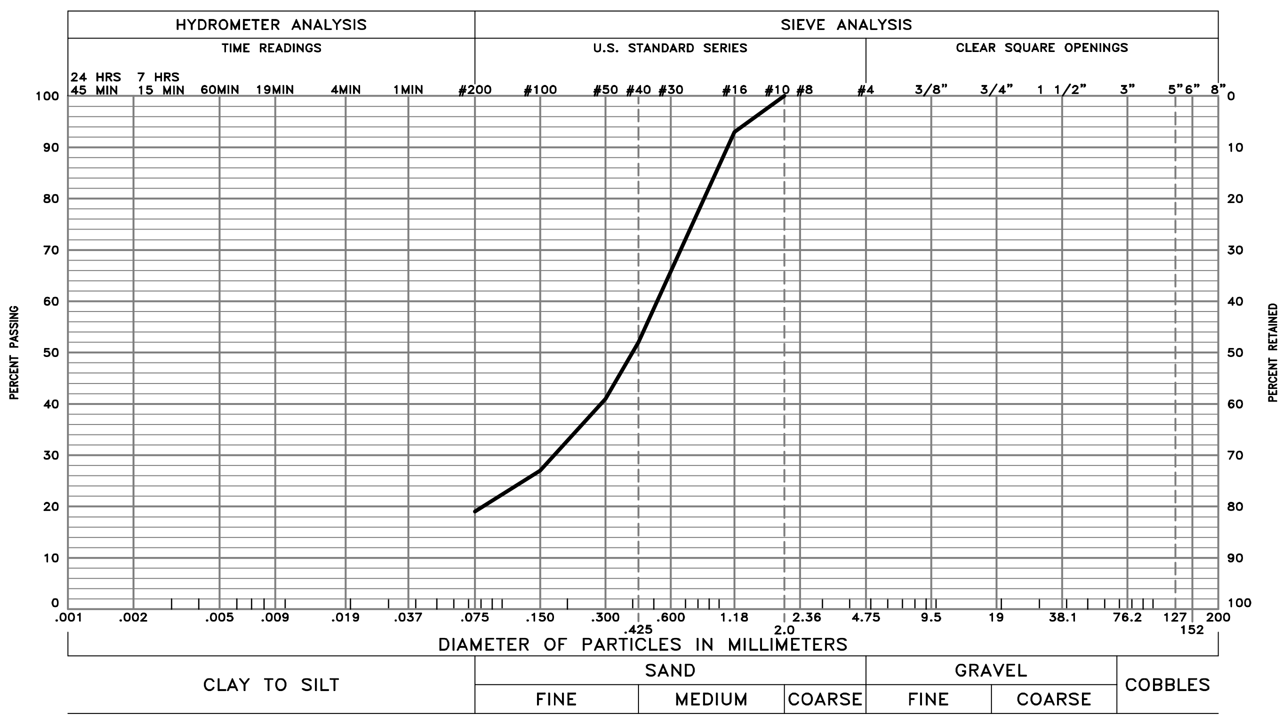
SAMPLE OF: Silty Clayey Sand (SC-SM) FROM: Boring 9 @ 4'



GRAVEL 0 % SAND 66 % SILT AND CLAY 34 %

LIQUID LIMIT 20 PLASTICITY INDEX 5

SAMPLE OF: Silty Clayey Sand (SC–SM) FROM: Boring 10 @ 4'



GRAVEL 0 % SAND 81 % SILT AND CLAY 19 %

LIQUID LIMIT PLASTICITY INDEX NP

SAMPLE OF: Silty Sand (SM) FROM: Boring 11 @ 4'

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.

20–2–126	Kumar & Associates	GRADATION TEST RESULTS	Fig. 10

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Kumar and Associates, Inc.

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

Project No.: 20-2-126
Project Name: Cherokee Metro
Date Sampled: 3/31/2020
Date Received: 4/6/2020

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 (Unified Soil Classification)
BORING	DEPTH (ft)				GRAVEL (%)	SAND (%)		LIQUID LIMIT	PLASTICITY INDEX			
1	2	4/16/20	2.0	109.6	1	85	14		NP		A-2-4 (0)	Silty Sand (SM)
1	9	4/16/20	1.0	110.2	0	96	4		NP		A-1-b (0)	Poorly Graded Sand (SP)
2	4	4/16/20	1.6	116.1	3	81	16		NP		A-2-4 (0)	Silty Sand (SM)
3	4	4/16/20	4.4	100.7	0	84	16		NP	<0.01	A-2-4 (0)	Silty Sand (SM)
3	9	4/16/20	2.6	109.6	0	89	11		NP		A-2-4 (0)	Poorly Graded Sand with Silt (SP-SM)
4	4	4/16/20	1.7	110.1	1	92	7		NP		A-1-b (0)	Poorly Graded Sand with Silt (SP-SM)
4	9	4/16/20	3.4	107.4	0	89	11		NP		A-2-4 (0)	Poorly Graded Sand with Silt (SP-SM)
5	4	4/16/20	2.7	110.4			17		NP		A-1-b (0)	Silty Sand (SM)
5	9	4/16/20	4.6	109.8			12		NP		A-1-a (0)	Poorly Graded Sand with Silt (SP-SM)
5	14	4/16/20	2.8	113.0			7		NP		A-1-a (0)	Poorly Graded Sand with Silt (SP-SM)
6	4	4/16/20	2.2	112.3	2	92	6		NP		A-1-b (0)	Well Graded Sand with Silt (SW-SM)

Kumar and Associates, Inc.

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

Project No.: 20-2-126
Project Name: Cherokee Metro
Date Sampled: 3/31/2020
Date Received: 4/6/2020

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 (Unified Soil Classification)
BORING	DEPTH (ft)				GRAVEL (%)	SAND (%)		LIQUID LIMIT	PLASTICITY INDEX			
7	2	4/16/20	4.4	98.8			45	25	6		A-4 (0)	Silty Clayey Sand (SC-SM)
7	4	4/16/20	5.0	95.0			48	27	10	<0.01	A-4 (2)	Clayey Sand (SC)
8	4	4/16/20	4.1	106.4	0	69	31	25	6		A-2-4 (0)	Silty Clayey Sand (SC-SM)
9	4	4/16/20	5.4	101.3	0	57	43	22	4		A-4 (0)	Silty Clayey Sand (SC-SM)
10	4	4/16/20	3.5	99.7	0	66	34	20	5		A-2-4 (0)	Silty Clayey Sand (SC-SM)
11	4	4/16/20	1.7	109.1	0	81	19		NP	<0.01	A-2-4 (0)	Silty Sand (SM)