JVA, Incorporated
1319 Spruce Street
Boulder, CO 80302
303.444.1951
info@jvajva.com

www.jvajva.com

June 14, 2023

El Paso County
Planning & Community Development
2880 International Circle, Suite 110
Colorado Springs, CO 80910-3127
Attn: Kari Parsons

RE: Cherokee Metropolitan District
Lift Station No. 1 Additions
Site Development Plan – Drainage Memo

Dear Kari:

Cherokee Metropolitan District (CMD, the District) is planning to construct a new lift station and wet well to replace the existing system located at the District's Sand Creek Facility. The proposed project is in unincorporated El Paso County on Parcel No. 5418000080. The purpose of this drainage memo is to satisfy the requirements of the El Paso County Planning and Community Development division for the project referenced above.

GENERAL LOCATION AND PROJECT DESCRIPTION

The proposed improvements are located at the District's Sand Creek Facility at 6657, 6677 East Platte Avenue in unincorporated El Paso County and at approximately 38°N 50' 10.5" latitude, 104°W 42' 24.7" longitude. The 53.38-acre property (Parcel No. 5418000080) is immediately south of Platte Avenue on the east bank of East Sand Creek and is bordered by the Peterson Space Force Base on its south and west sides. The Sand Creek Facility property is a Political Subdivision with zoning designations of RR-5 (residential rural district) and CAD-0 (commercial airport overlay district).

The site consists of the District's former wastewater treatment lagoon and has several unlined and lined lagoons, as well as a headworks facility. The current equalization pond functions as the wet well for the existing lift station (LS1A). It is open to the air and oversized, which results in a long retention time and odor issues. LS1A is an underground vault-type lift station and is a confined space that is difficult to access and maintain. To address the current issues, the District proposes to replace the LS1A with a new equalization wet well and lift station Lift Station 1B (LS1B). The function and capacity of the Sand Creek Facility does not change and LS1A will remain online for redundancy purposes.

The improvements consist of clearing and grubbing, construction of a new lift station building and covered wet well, utility connections, site grading, and demo of an accessory structure. No public right-of-way exists within the project limits.

EXISTING DRAINAGE FACILITIES

The 53.38-acre site consists mainly of the District's open air lagoons, of which collect the majority of on-site runoff. There are six lagoons, varying in size and depth which collect and detain the majority of on-site flows. The National Wetlands Inventory classifies several of the ponds as Freshwater Pond habitat, however, these are existing wastewater lagoons and do not serve as the habitat described. Since the existing lagoons utilize pumps to convey flows from the lagoons through the wastewater treatment system located on the north side of the site, the lagoons do not ultimately discharge off-site.

The area proposed for development will span two historic basins, H1 and H2. Both basins have similar runoff coefficients of 0.50. Due to the size of the site and basin, the proposed wet well at LS1B, there is no change to the runoff coefficient.

SOILS

A Natural Resources Conservation Service (NRCS) soils map and recent geotechnical report is attached to this memo. The map indicates the soil as Type "B" soil (moderate infiltration rate) for the site. Soils on-site are indicated to be loamy sands and coarse sands.

FLOODPLAIN DESIGNATION

The property is not located in the 100-year floodplain. The National Flood Hazard Insurance map for the site is attached to the end of this memo.

PROPOSED DRAINAGE MODIFICATIONS

Proposed drainage will generally maintain the same as the existing conditions. No impact to current drainage patterns is anticipated with this project. All drainage at the new lift station will be diverted into the north and south lagoon, where runoff currently flows, which maintain the historic patterns. These are existing wastewater holding lagoons and do not discharge into existing drainageway. They have since been discontinued for on-going use due to odor issues, but still act as an emergency overflow for the District. However, the lagoon are not designed or intended to drain back into existing drainageways. Any excess water is cycled through the treatment facilities. No permanent BMPs are proposed for this project. Construction of the new lift station involves grading within the limits of the Sand Creek Facility.

The area of land disturbance is less than an acre (approximately 0.6 acres). The overall imperviousness of the disturbance area will remain essentially the same with a slight increase due to the proposed Lift Station (LS1B) and wet well building, however, they are not signification enough to affect the runoff coefficient. The facility will meet International Building Code (IBC) to provide positive drainage away from the facility, directed to grass lined swales, which will ultimately drain north and into the lagoon. All runoff in the disturbance area will sheet flow to the north and into the existing lagoon.

Erosion and sediment control best management practices (BMPs) are generally shown on the plans. All BMPs are to conform to El Paso County *Engineering Criteria Manual (ECM), Land Development Code (LDC),* which generally refers methods of Mile High Flood Districts latest edition of *Urban Storm Drainage Criteria Manual, Volume 3 – Stormwater Quality*.

SUMMARY

Proposed drainage characteristics will generally remain the same as existing, with the exception of modification made for the installation of LS1B and proposed wet well facility. Drainage will be directed around the proposed facility, however, will match historic patterns which generally utilize the existing lagoons, therefore no detention facilities are proposed.

Sincerely,
JVA, INCORPORATED

By: ______________________
Nathan Skalak, P.E.
Project Manager

Enclosure: Drainage Exhibit
Vicinity Map
Soil Map
Geotechnical Report

CC: Jeff Munger, Cherokee Metropolitan District
Amy Lathen, Cherokee Metropolitan District

Design Engineer's Statement:

The attached drainage plan and report were prepared under my direction and supervision and are correct to the best of my knowledge and belief. Said drainage report has been prepared according to the criteria established by the County for drainage reports and said report is in conformity with the applicable master plan of the drainage basin. I accept responsibility for any liability caused by any negligent acts, errors or omissions on my part in preparing this report.

COLORADO REGISTERED MICHAEL KATALINICH 49620 PROFESSIONAL ENGINEER

Michael Katalinich, P.E. #49620 — 6/15/2023
Date

Owner/Developer's Statement:

I, the owner/developer have read and will comply with all of the requirements specified in this drainage report and plan.

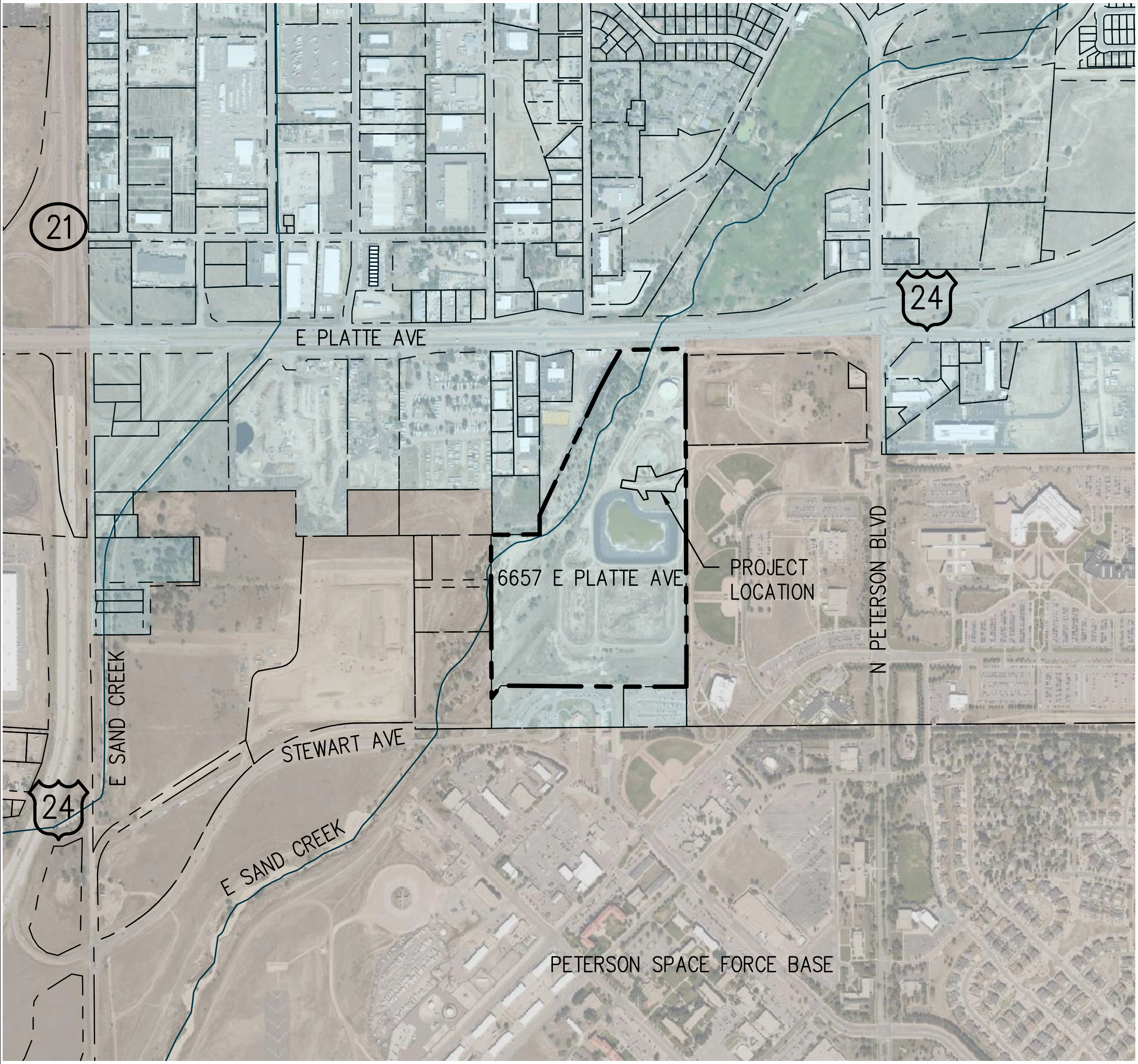
Amy Lathen, District Manager — 6-15-23
Cherokee Metropolitan District — Date
6250 Palmer Bark Blvd
Colorado Springs, CO 80915

El Paso County:

Filed in accordance with the requirements of the Drainage Criteria Manual, Volumes 1 and 2, El Paso County Engineering Criteria Manual and Land Development Code as amended.

County Engineer / ECM Administrator Date

Conditions:



LEGEND

UNINCORPORATED EL PASO COUNTY

CITY OF COLORADO SPRINGS

N

1000 0 1000 2000

SCALE IN FEET

CMD VICINITY MAP
PARCEL NO. 5418000080
OCTOBER 2022

JVA, Inc.
1319 Spruce Street
Boulder, CO 80302
303.444.1951
www.jvajva.com
Boulder • Fort Collins • Winter Park
Glenwood Springs • Denver

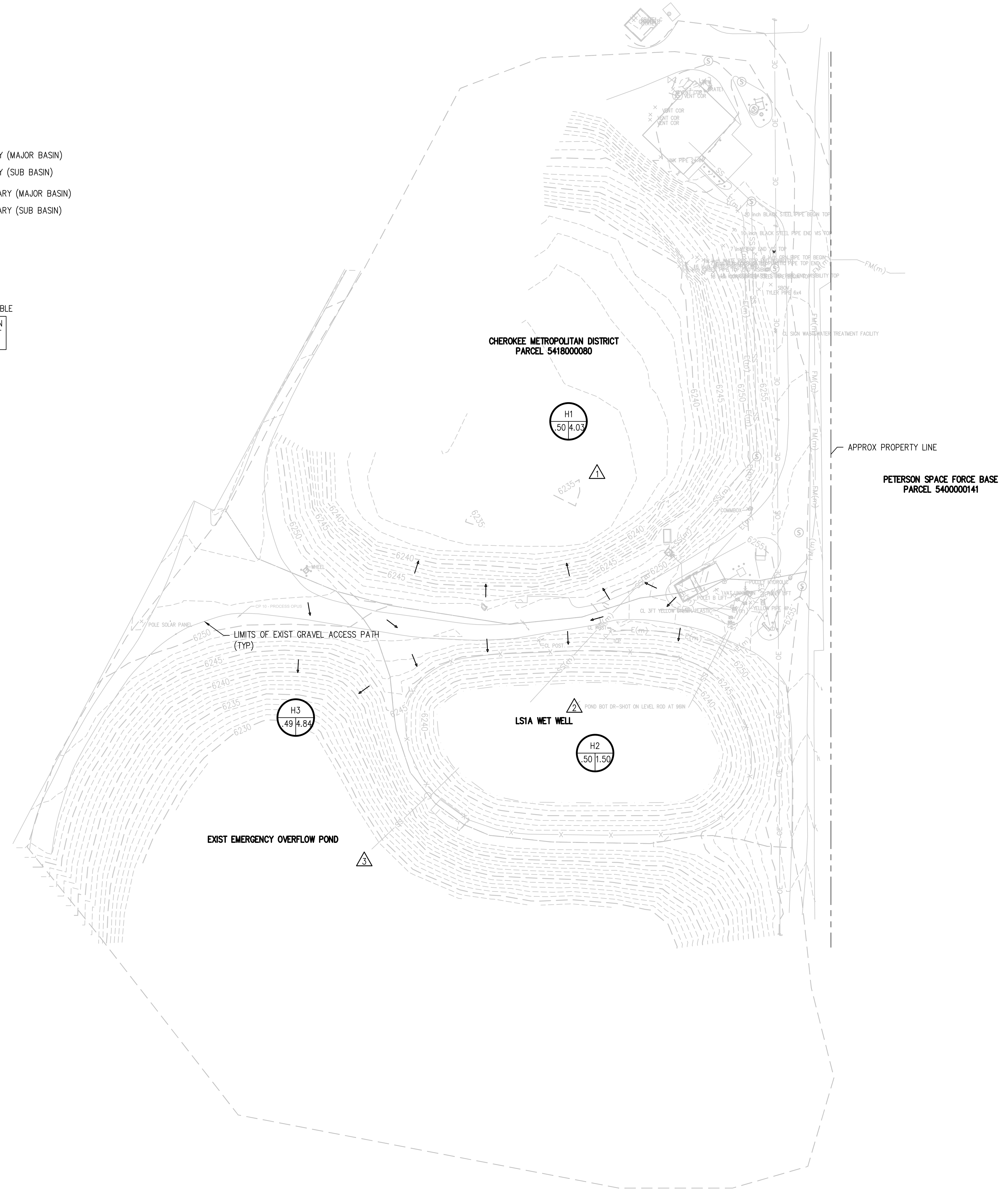
N:\1164e – Cherokee Metro District – LS1 Additions\Drawings\Exhibits-Figures\El Paso County Permitting\1164e – Vicinity Map.dwg, 10/27/2022 – 1:01 PM, mht

FILE NO.: PPR-2254

DRAINAGE MAP LEGEND

- EXISTING INDEX CONTOUR
- EXISTING INTERMEDIATE CONTOUR
- PROPOSED INDEX CONTOUR
- PROPOSED INTERMEDIATE CONTOUR
- +03.3 EXISTING SPOT ELEVATION
- +03.54 PROPOSED SPOT ELEVATION
- HISTORIC DRAINAGE BASIN BOUNDARY (MAJOR BASIN)
- HISTORIC DRAINAGE BASIN BOUNDARY (SUB BASIN)
- DEVELOPED DRAINAGE BASIN BOUNDARY (MAJOR BASIN)
- DEVELOPED DRAINAGE BASIN BOUNDARY (SUB BASIN)
- DIRECTION OF FLOW (HISTORIC)
- DIRECTION OF FLOW (DEVELOPED)
- OVERFLOW PATH
- BASIN DESIGN POINT
- DRAINAGE BASIN IDENTIFICATION BUBBLE

A = DEVELOPED BASIN DESIGNATION
.50 = 100-YR RUNOFF COEFFICIENT
1.0 = AREA ACRES



SCALE IN FEET: 50 0 50 100

JVA CONSULTING ENGINEERS
JVA, Inc. 1319 Spruce Street
Boulder, CO 80302 303.444.1951
www.jvajva.com
Boulder • Fort Collins • Winter Park
Glenwood Springs • Denver

NO.	DATE	DES'D	DWN	REVISION DESCRIPTION

DESIGNED BY:	NAS
DRAWN BY:	MHT
CHECKED BY:	JJM
JOB #:	1164e
DATE:	OCT 2022

CHEROKEE METROPOLITAN DISTRICT
LIFT STATION NO. 1B
EL PASO COUNTY, COLORADO

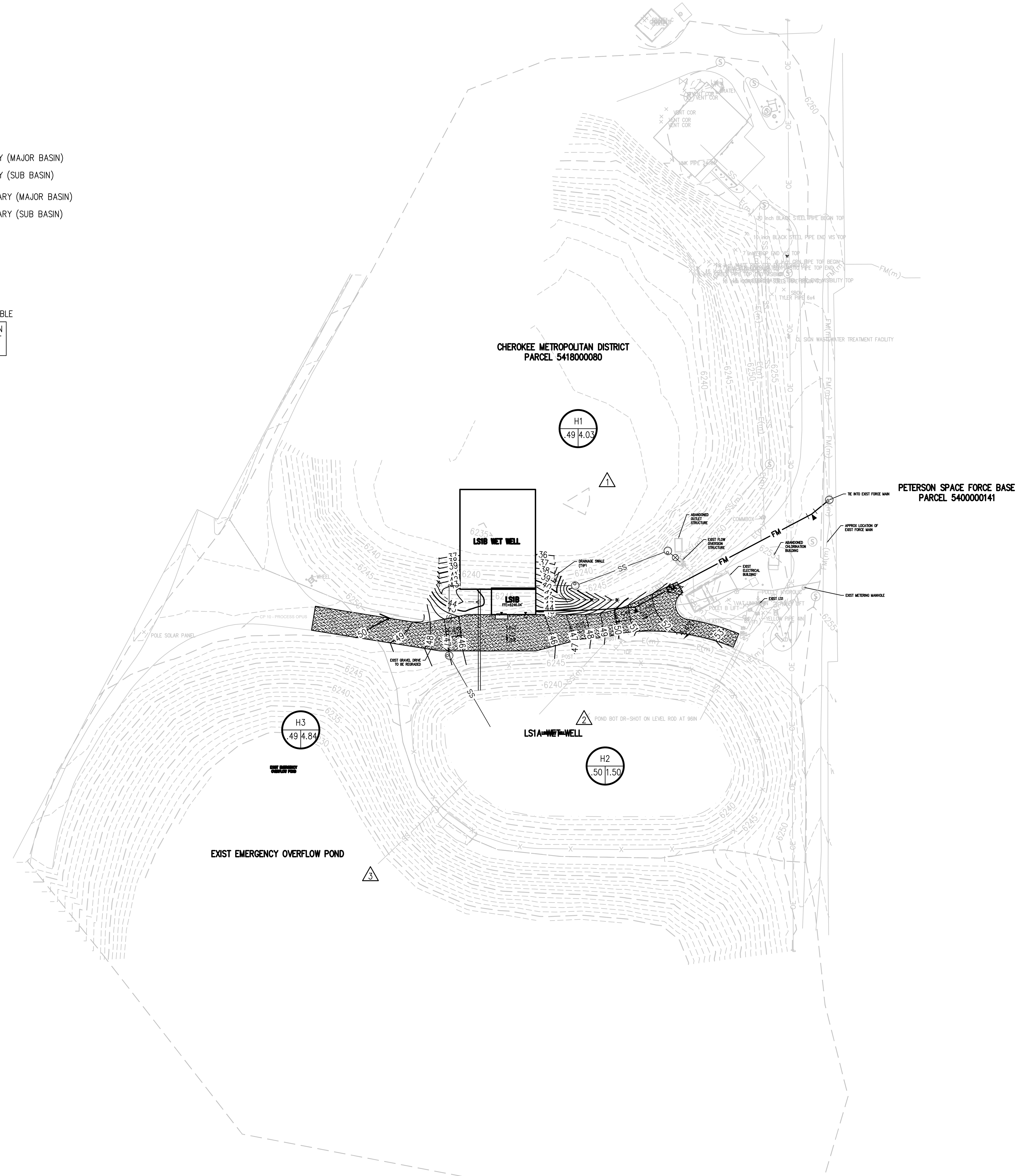
HISTORIC DRAINAGE

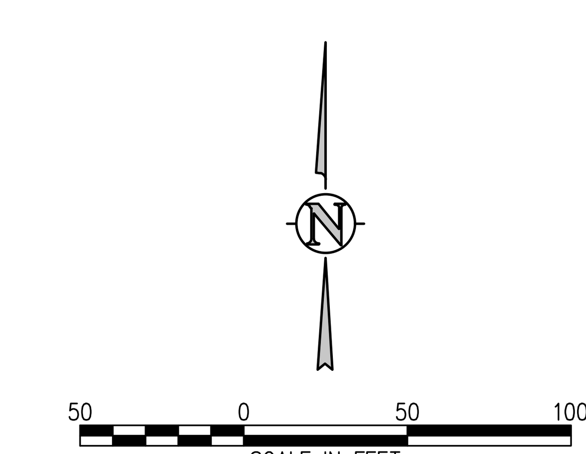
SHEET NO.
FIG-1

DRAINAGE MAP LEGEND

- EXISTING INDEX CONTOUR
- EXISTING INTERMEDIATE CONTOUR
- PROPOSED INDEX CONTOUR
- PROPOSED INTERMEDIATE CONTOUR
- +03.3 EXISTING SPOT ELEVATION
- +03.54 PROPOSED SPOT ELEVATION
- HISTORIC DRAINAGE BASIN BOUNDARY (MAJOR BASIN)
- HISTORIC DRAINAGE BASIN BOUNDARY (SUB BASIN)
- DEVELOPED DRAINAGE BASIN BOUNDARY (MAJOR BASIN)
- DEVELOPED DRAINAGE BASIN BOUNDARY (SUB BASIN)
- DIRECTION OF FLOW (HISTORIC)
- DIRECTION OF FLOW (DEVELOPED)
- OVERFLOW PATH
- BASIN DESIGN POINT
- DRAINAGE BASIN IDENTIFICATION BUBBLE

A = DEVELOPED BASIN DESIGNATION
.50 = 100-YR RUNOFF COEFFICIENT
1.0 = AREA ACRES





JVA CONSULTING ENGINEERS
JVA, Inc. 1319 Spruce Street
Boulder, CO 80302 303.444.1951
www.jvajva.com
Boulder • Fort Collins • Winter Park
Glenwood Springs • Denver

NO.	DATE	DES'D	DWN	REVISION DESCRIPTION

DESIGNED BY:	MK
DRAWN BY:	MHT
CHECKED BY:	JJM
JOB #:	1164e
DATE:	NOVEMBER 2022

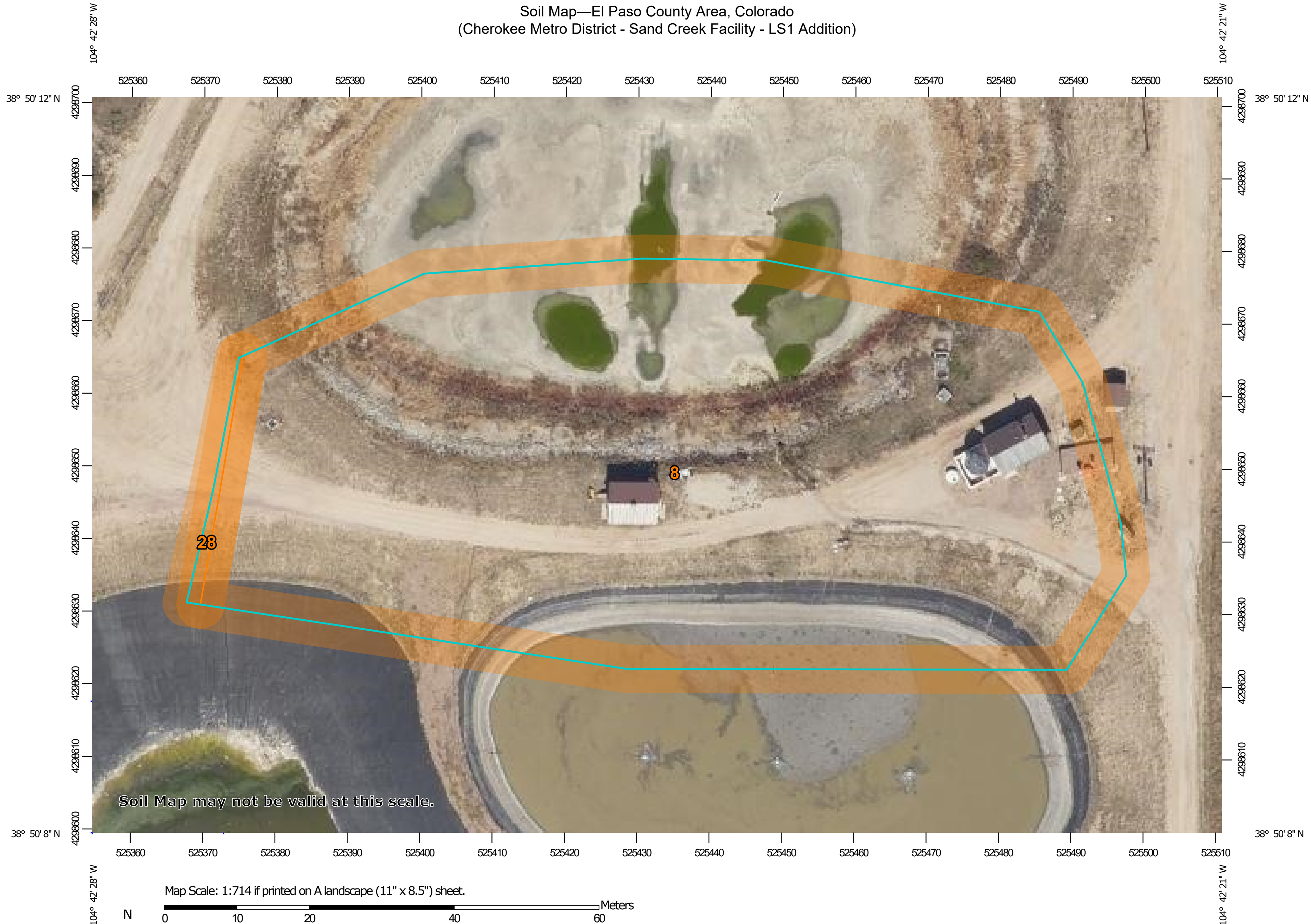
LIFT STATION NO. 1B
CHEROKEE METROPOLITAN DISTRICT
EL PASO COUNTY, COLORADO

DEVELOPED DRAINAGE

SHEET NO.
FIG-2

N:\1164e – Cherokee Metro District – LS1 Additions\Drawings\1164e – C1.0 – DEV.dwg, 11/1/2022 – 1:45 PM, nos

Soil Map—El Paso County Area, Colorado
(Cherokee Metro District - Sand Creek Facility - LS1 Addition)



Map Scale: 1:714 if printed on A landscape (11" x 8.5") sheet.

N

Meters: 0, 10, 20, 40, 60

Feet: 0, 30, 60, 120, 180

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 13N WGS84

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Map Unit Polygons

Soil Map Unit Lines

Soil Map Unit Points

Special Point Features

Blowout

Borrow Pit

Clay Spot

Closed Depression

Gravel Pit

Gravelly Spot

Landfill

Lava Flow

Marsh or swamp

Mine or Quarry

Miscellaneous Water

Perennial Water

Rock Outcrop

Saline Spot

Sandy Spot

Severely Eroded Spot

Sinkhole

Slide or Slip

Sodic Spot

Spoil Area

Stony Spot

Very Stony Spot

Wet Spot

Other

Special Line Features

Water Features

Streams and Canals

Transportation

Rails

Interstate Highways

US Routes

Major Roads

Local Roads

Background

Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: El Paso County Area, Colorado
Survey Area Data: Version 20, Sep 2, 2022

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Aug 19, 2018—Sep 23, 2018

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
8	Blakeland loamy sand, 1 to 9 percent slopes	1.5	99.4%
28	Ellicott loamy coarse sand, 0 to 5 percent slopes	0.0	0.6%
Totals for Area of Interest		**1.5**	**100.0%**

Kumar & Associates, Inc.®
Geotechnical and Materials Engineers
and Environmental Scientists

6735 Kumar Heights
Colorado Springs, CO 80918
phone: (719) 632-7009
fax: (719) 632-1049
email: kacolospgs@kumarusa.com
www.kumarusa.com

An Employee Owned Company

Office Locations: Denver (HQ), Parker, Colorado Springs, Fort Collins, Glenwood Springs, and Summit County, Colorado

GEOTECHNICAL ENGINEERING STUDY
CHEROKEE METROPOLITAN DISTRICT
TREATMENT POND UPGRADES
EAST PLATTE AVENUE AND PETERSON BOULEVARD
COLORADO SPRINGS, COLORADO

Prepared By:
Arben F. Kalaveshi, P.E.

COLORADO REGISTERED PROFESSIONAL ENGINEER
ARBEN F. KALAVESHI
45569
08-11-2022

Reviewed By:

Duane P. Craft, P.E.

Prepared For:

Mr. Michael Katalinich
JVA, Incorporated
1319 Spruce Street
Boulder, Colorado 80302

Project No: 22-2-146

August 11, 2022

TABLE OF CONTENTS

SUMMARY

1. Two borings were drilled to evaluate the subsurface conditions at this site. The general subsurface profile consisted of silty to clayey sand fill underlain by native well graded sand with silt, in turn underlain by fat clay.

2. Groundwater was encountered in both borings during drilling, but because Boring 2 had been backfilled after drilling, it was not measured in that boring on our return visit seven days later. The water level showed little change in Boring 1 when the second measurement was made, and we anticipate a similar groundwater condition in Boring 2. The elevation of groundwater was determined to be 6224 feet when the final measurement was made in Boring 1, based on the elevations shown on the contour map provided to us.

   Groundwater is expected to fluctuate over time due to seasonal and climatic factors as well as the use of adjacent unlined ponds.

3. We understand the new pump station will have a base elevation of about 6224 feet to match the elevation of the existing wet well. If this is the case, the structure will be constructed at or near the groundwater level, and also at or near the layer of fat clay encountered in our field exploration. Although the very moist clay was not found to possess significant swell potential at it's current moisture content, these soils are known to exhibit significant volume change with changes in moisture content, and will offer poor support to shallow foundations when overly moist. Considering the proximity of the base of the structure to these conditions, we recommend the use of deep foundations for the construction of the proposed pump station and equalization tank.

4. Because bedrock is anticipated to be deep at this site, helical pier foundations bearing in the underlying soils would be a suitable foundation option for the support of structures that will be constructed near the elevation of the existing water table. We anticipate it will be possible to achieve an ultimate capacity of at least 10 ksf at nominal depths of about 20 feet by using the appropriate size and number of bearing plates.

5. Shallow foundations can be considered for structures with base elevations at least 5 feet above the existing groundwater elevation, but the existing fill will need to be removed, moisture conditioned, and recompacted. Footings bearing on suitable compacted fill can be designed for a maximum allowable net bearing pressure of 2,500 psf.

PURPOSE AND SCOPE OF STUDY

This report presents the results of a geotechnical engineering study for the improvement of the existing water treatment facility at this site. We have conducted the study in general accordance with the scope of work in our Proposal No. C22-149, dated April 15, 2022 for the purpose of providing recommendations for site grading and foundations.

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 in the report.

PROPOSED CONSTRUCTION

A new pump station and equalization tank will be constructed near the south bank of the northmost water treatment pond at the existing water treatment facility. The new structure will be about 20 to 22 feet deep to match the elevation of the existing wet well. Based on the contour map provided to us for this study, we anticipate that the elevation of the base of the structure will be about 6224 feet. The plan area of the new structures will be less than about 6,000 square feet. If locations or conditions are significantly different from those described above or depicted in this report, we should be notified to reevaluate the recommendations contained herein.

SITE CONDITIONS

The project area is located at the existing water treatment facility west of North Peterson Boulevard, and south of East Platte Boulevard. The property is roughly triangular, and borders Peterson Space Force Base on its south and west sides. The northwest side of the site is bordered by the East Fork of Sank Creek. Within the property, an above-ground water storage tank is located at the north end, along with several single story buildings. Six water treatment ponds are located on the remainder of the site, and this project will be located at the south side of the northmost pond.

SUBSURFACE CONDITIONS

Information on the subsurface conditions was obtained by drilling two exploratory borings at the approximate locations shown on Fig. 1. The borings were drilled on July 15, 2022 with solid stem continuous flight power auger. Graphic logs of the borings are presented on Fig. 2, and the corresponding legend and notes are presented on Fig. 3. The results of laboratory testing performed on selected soil samples from the borings are presented on Figs. 4 and 5, and are summarized on Table I. The laboratory testing was conducted in general accordance with applicable ASTM standards.

The general subsurface profile consisted of silty to clayey sand fill underlain by native well graded sand with silt, in turn underlain by fat clay. The fill extended to depths of about 1½ to 7 feet below the ground surface, and the underlying sands extended to depths of 9 to 23 feet, or elevations of roughly 6223 to 6224 feet based on the contour maps provided to us for this project. The following subsurface descriptions are of a generalized nature to highlight the major stratification features in the borings drilled for this study. The boring logs should be reviewed for more detailed information.

The existing fill encountered during our exploration consisted of silty sand with some zones of clayey sand. This material was fine to coarse grained, moist, and light to dark brown. The vertical and lateral extents of the fill were not determined in the scope of our study, but based on the layout of this site, it appears that the berm at the edge of the pond is fill, and native soils are anticipated at depths greater than a few feet below the basin elevation. A sample of fill selected for testing exhibited slight compression upon wetting under a surcharge pressure of 1 ksf.

The native alluvial sand found below the fill was well graded sand with silt, and was fine to coarse grained with gravel, medium dense, moist to wet, and brown to light brown.

The fat clay found below the sand included a fine grained sand fraction, was medium stiff to stiff, very moist, and brown. A sample of fat clay selected for testing exhibited slight expansion upon wetting under a surcharge pressure of 1 ksf.

Groundwater was encountered in both borings during drilling, but was only measured in Boring 1 on our return visit seven days later because Boring 2 had been backfilled after drilling. The water level showed little change in Boring 1 when the second measurement was made, and we anticipate a similar groundwater condition in Boring 2. The elevation of groundwater was determined to be at 6224 feet when the final measurement was made in Boring 1, based on the elevations shown on the contour map provided to us. The measured groundwater depths and inferred elevations are presented in the following table.

Groundwater is expected to fluctuate over time due to seasonal and climatic factors as well as the use of adjacent unlined ponds.

Boring	Groundwater Depth (feet)		Inferred Groundwater Elevation (feet)
	At time of drill	Final measurement	
1	23	21.9	6224
2	7.5	*	6225

*: No follow-up water measurements were made at this location.

GEOTECHNICAL CONSIDERATIONS

We understand that the proposed pump station structure will have a base elevation of about 6224 feet. If this is the case, the structure will be constructed at or near the groundwater level, and also at or near the layer of fat clay encountered in our field exploration. Although the very moist clay was not found to possess significant swell potential at its current moisture content, these soils are known to exhibit significant volume change with changes in moisture content, and will offer poor support to shallow foundations when overly moist. Considering the proximity of the base of the structure to these conditions, we recommend the use of deep foundations for the construction of the proposed pump station and equalization tank.

Shallow foundations may be feasible at this site for structures founded at elevations at least 5 feet above groundwater, but the existing fill should be removed, moisture conditioned, and recompacted where it is found below the construction envelope.

FOUNDATIONS

As discussed section above, we recommend deep foundations for structures that will be constructed near the existing water level, but shallow foundations may be considered for structures constructed at higher elevations. Recommendations for both helical pier foundations and shallow footing foundations are provided below. Other deep foundation options such as caissons were also considered for this project, but may be difficult to construct due to the anticipated depth of bedrock. We can provide recommendations for such an alternative upon request, but additional subsurface exploration would be required.

Helical Pier Foundations: Helical pier foundations are feasible at this site and should result in relatively small disturbance of the in place infrastructure.

The axial design load of helical piers should be determined in general accordance with the current International Building Code (IBC), which states the allowable axial design load, P_a, should be determined as follows:

$P_a = 0.5\ P_u$, where P_u (the ultimate load) is the least value of:

1. Sum of the areas of the helical bearing plates times the ultimate bearing capacity of the soil or rock comprising the bearing stratum.

2. Ultimate capacity determined from well-documented correlations with installation torque.

3. Ultimate capacity determined from load tests.

4. Ultimate capacity of pile shaft.

5. Ultimate capacity of pile couplings.

6. Sum of the Ultimate axial capacity of helical bearing plates affixed to pile.

Items 1 through 3 are related to the geotechnical capacity of the piers; Items 4 through 6 are related to the structural capacity and should be evaluated by the structural engineer. The owner and structural designer should be aware that certain proprietary helical pier systems have been subjected to acceptance testing administered by the International Code Council (ICC), while other systems provided by specialty contractors may be fabricated according to designs by registered professional engineers. The certified systems have documentation that addresses many of the structural capacity issues, while the non-certified systems require structural design by an engineer. Many of the lighter-duty helical pile systems available, with working capacities on the order of 50 kips or less, are certified, which can simplify the design and submittal process. However, higher capacity systems, where single piers may have working capacities of 200 kips or more, sometimes referred to as screw piles, are often designed and fabricated and are not certified, manufactured systems.

Based on consideration of bearing capacity theory and published correlations of boring penetration resistance values with ultimate bearing capacity, we recommend an ultimate bearing capacity of 10 ksf for a helical pile embedded in the native clays. We anticipate it will be possible to achieve adequate capacities at nominal depths of about 20 feet by using the appropriate size and number of bearing plates. Nominal depths should be measured from the topmost bearing plate.

Helical piers are typically very slender foundation elements with a low capacity for resisting lateral loads. Lateral restraint of a helical pile foundation system is normally provided through the use of passive pressure on pile caps or foundation walls, or through the use of battered piers. It is normally assumed that a battered pile can be designed for the same axial load as a vertical pile, with the lateral restraint being provided by the horizontal component of the battered pile. Helical piers are often assumed to have tension capacities similar to the axial compressive capacity, although that should be evaluated through load testing or otherwise addressed by the specialty contractor's submittal.

Acceptance of helical pile installation should be based on attaining a specified torque in the recommended bearing stratum determined in accordance with correlations of installation torque to capacity based on calibrated torque measurements and axial load test data. In our opinion, the ultimate bearing capacity recommended above may be exceeded if supported by adequate site-specific load test data. If site-specific load tests are not performed, the specialty helical pile contractor's submittal should contain torque-to-capacity data for their pile system in similar soil conditions. If that information cannot be provided, site-specific load tests should be performed in accordance with ASTM D 1143.

We recommend that a qualified helical pile specialty contractor be retained to provide the required design submittal and to provide and install the helical piers. The project design should include a performance specification indicating required capacities, structural requirements, and submittal requirements. At a minimum, the submittal should be required to contain information supporting capacity determination, a description of equipment and installation procedures that will ensure penetration to the required depths, and acknowledgement that the helical bearing plates will be installed into the recommended bearing stratum, as well as all necessary information to satisfy the requirements of the project structural designer.

We should be retained to review the contractor's submittal, and to provide installation observation including monitoring depths and general conformance with the plans and specifications. Our observation and testing services will be intended to document that all of the helix bearing plates on the piers are installed into an adequate bearing stratum.

Footing Foundations: The design and construction criteria presented below should be observed for footing foundations. Construction details should be considered when preparing project documents.

1. Shallow foundations should bear on suitable fill that has been moisture conditioned and compacted in controlled lifts, and should have a base elevation at least 5 feet above the existing groundwater level. The specifications for fill materials along with a discussion regarding reuse of the on-site materials and compaction criteria are presented in the "Site Grading" section of the report. Wherever possible, a proof roll should be conducted at the base of the footing level to confirm suitable bearing conditions using a heavily loaded vehicle. Areas with significant deflection should be stabilized prior to fill placement. Detailed recommendations for stabilization can be found in the "Excavation Considerations" section.

2. We recommend a maximum net allowable bearing pressure of 2,500 psf for footings placed on properly compacted fill. This value may be increased by a factor of 1/3 for transient loading.

3. 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.

4. We estimate total movement for footings designed and constructed as discussed in this section will be on the order of 1 inch or less.

5. Spread footings should have a minimum width of 16 inches for continuous footings and 24 inches for isolated pads.

6. The lateral resistance of a footing placed on properly compacted structural fill material will be a combination of the sliding resistance of the foundation 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 0.35. Passive pressure against the sides of the foundation may be calculated using an allowable equivalent fluid unit weight of 180 pcf for compacted backfill.

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

8. Areas of existing fill, soft, loose, or otherwise unsuitable material encountered within the foundation excavation should be removed and replaced with properly compacted structural fill.

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

FLOOR SLABS

The existing fill materials may be reused to support lightly to moderately loaded slab-on-grade construction if they are moisture conditioned and compacted to the requirements listed in the “Site Gradin” section of this report. To reduce the effects of some differential movement, the following measures should be taken.

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

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

3. 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.

4. 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.

5. 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 but, 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	95

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, 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 to dense granular soils overlying medium stiff to stiff clays. Based on shear wave velocities for the subgrade materials estimated from the sampler penetration testing resistance values, the site subsurface profile meets the requirements for Site Class D.

Based on the subsurface profile, the anticipated ground conditions, and considering that the project area is in a region with relatively low seismic activity, liquefaction is not a design consideration. Using the USGS National Earthquake Hazard Reduction Program online database, the following unfactored risk-targeted maximum considered earthquake (MCE_R) probabilistic ground motion values are reported for the project site.

Spectral Acceleration	Acceleration Coefficient
S_s (0.2 Sec. Period)	0.174
S_1 (1.0 Sec. Period)	0.060

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 a representative sample 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.

SURFACE DRAINAGE

Providing proper surface drainage, both during construction and after the construction has been completed, is very important for acceptable performance of the facility. 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 or slab subgrades should be avoided during construction.

2. Care should be taken when compacting around the foundation walls to avoid damage to the structure.

3. The ground surface surrounding the exterior of the structure 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. Excessive landscape irrigation should be avoided within 10 feet of the foundation walls.

EXCAVATION CONSIDERATIONS

Excavation of the overburden soils should be possible with conventional heavy-duty equipment. We recommend temporary excavation slopes in the soils be constructed in accordance with OSHA regulations. In accordance with OSHA criteria, the on-site soils classify as an OSHA Type C soil. Per OSHA criteria, unless excavations are shored, temporary unretained excavations should have slopes no steeper than the following for each soil type encountered:

Type A…………………..$^{3}/_{4}$:1 (H:V)
Type B……………………1:1 (H:V)
Type C……………….1½:1 (H:V)

A properly braced excavation or the use of a trench box should be used where the indicated unretained slopes cannot be accommodated. 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. 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 all decisions regarding excavation slopes.

We anticipate that groundwater may be a consideration during excavation, and a dewatering system will likely be necessary. A dewatering system consisting of trenches flowing into sumps where the water can be discharged using pumps will likely be adequate, but a more robust system such as well points may be necessary if high volumes of water are encountered.

Subgrade Stabilization: If they are encountered, unstable soils may be stabilized by scarifying/ripping the subgrade and allowing them to dry, or by over-excavation and replacement of the subgrade with suitable, imported, angular, well-graded materials if they are encountered. 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.

SITE GRADING

We recommend the following criteria be used when preparing the site grading plans.

Removal and Replacement Requirements: Where existing fill is encountered, we recommend that it be removed from below the proposed foundations and slabs. All removed materials should be replaced with suitable fill that has been moisture conditioned and compacted as described in the subsection below.

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

1. *Fill Within the Proposed Development*: The on-site soils, minus any clays will be suitable for reuse. Clays may be used within areas where no structures or movement sensitive construction will occur, and low permeability will not impact the intended use. Import soils if used within the construction envelope, should consist of a non-expansive soil, consisting of a minus 2-inch material that has a maximum of 35 percent passing the No. 200 sieve, and a maximum plasticity index of 15. New fill should extend down from the edges of the foundations at a minimum 1:1 horizontal to vertical projection.

2. *Utility Trench Backfill*: Material excavated from the utility trenches may be used for backfill provided it does not contain unsuitable material or particles larger than 2 inches.

3. *Material Suitability*: All fill material should be free of vegetation, brush, sod, and other deleterious substances. The geotechnical engineer should evaluate the suitability of all proposed fill materials prior to placement.

4. *Subgrade Preparation*: The ground surface shall be stripped of vegetation/organics, loose soils, or any other unsuitable materials prior to fill placement. The resulting ground surface should be scarified to a depth of 12 inches; moisture conditioned as necessary and compacted in a manner specified below for the subsequent layers of fill. Loose or unstable soils shall be removed, where present, to provide a stable platform prior to placement of fill.

Compaction Requirements: 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)
Structure Footprint	98%
Exterior Flatwork, Fill placed for Site Grading	95%
Foundation Wall Backfill	95%
Landscape and Other Misc. Overlot Fill Areas	95%
Compaction of granular materials should be achieved at a moisture content within +/- 2% of the optimum moisture content. Cohesive materials should be compacted at a moisture content between 0 and +3 percent of optimum.	

If large volumes of relatively clean sand are placed, the use of a Modified Proctor (ASTM D1557) may be considered. If this is the case, soils should be compacted to 95 percent of the maximum dry density as determined by the Modified Proctor within the structure footprint, and 90 percent elsewhere.

Soils should be placed in loose lifts no thicker than 12 inches, or thinner as appropriate for the compaction equipment utilized.

DESIGN AND SUPPORT SERVICES

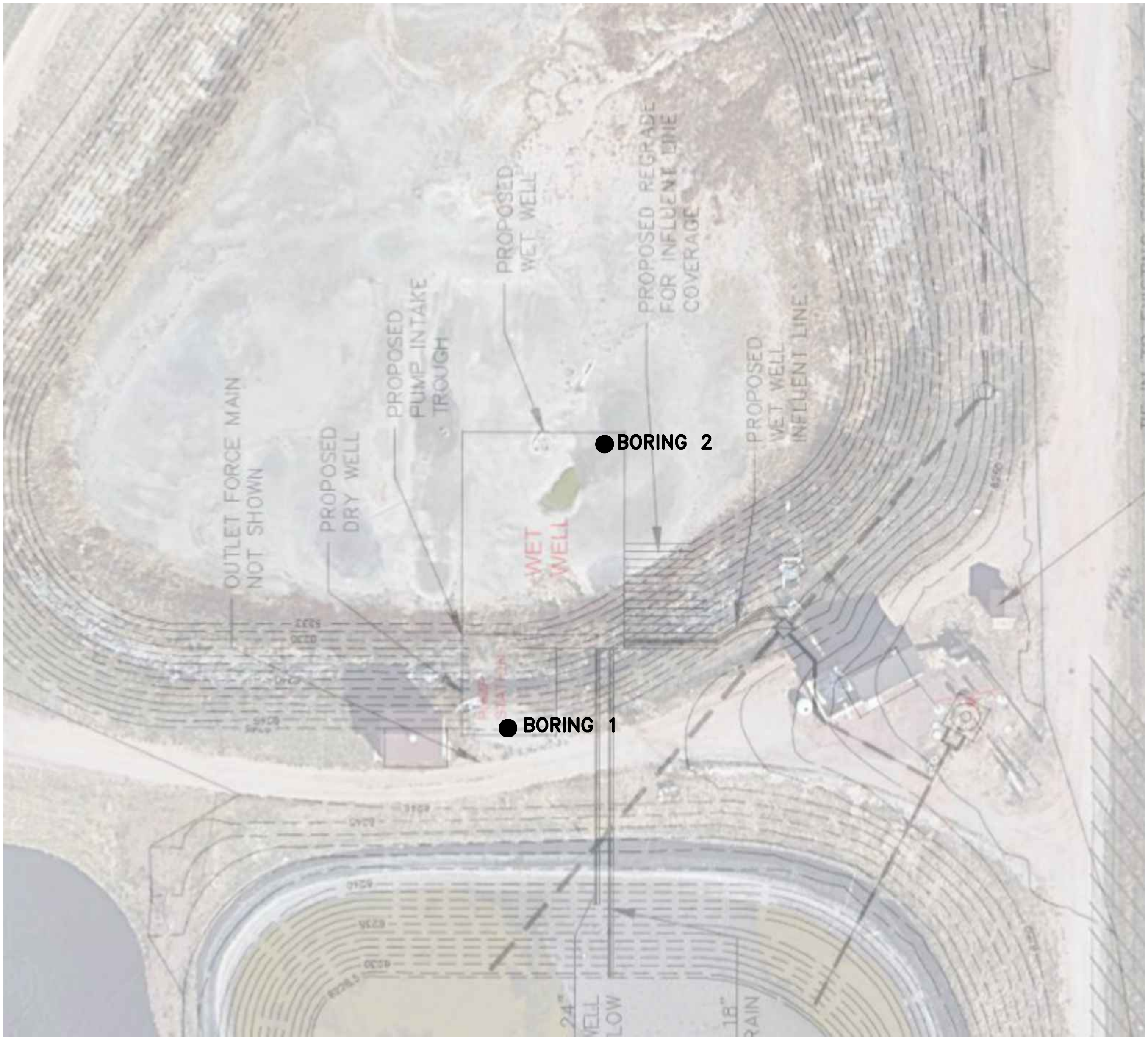
Kumar & Associates, Inc. should be retained to review the project plans and specifications for conformance with the recommendations provided in our report. We are also available to assist the design team in preparing specifications for geotechnical aspects of the project, and performing additional studies,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

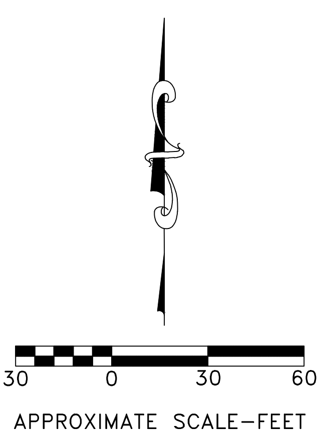
We recommend that Kumar & Associates, Inc. be retained to provide observation and testing services to document that the intent of this report and the requirements of the plans and specifications are being followed during construction, and to identify variations in subsurface conditions from those encountered in this study so that we can re-evaluate our recommendations, if needed.

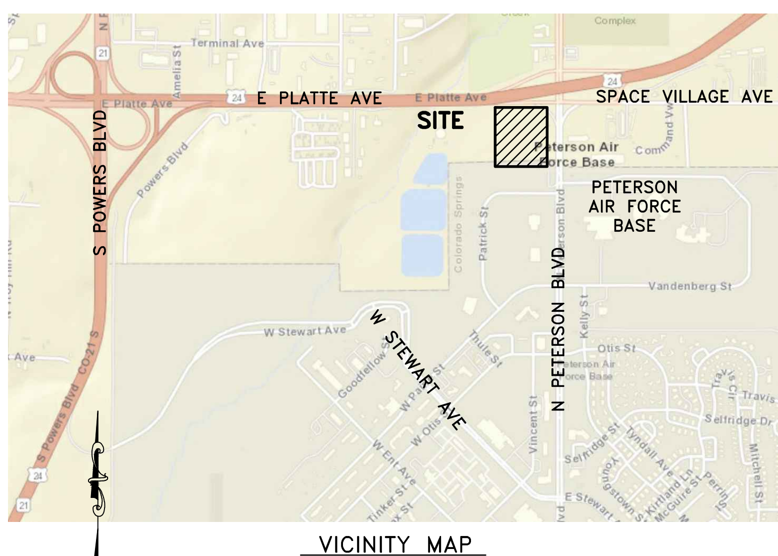
LIMITATIONS

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.

AFK/th



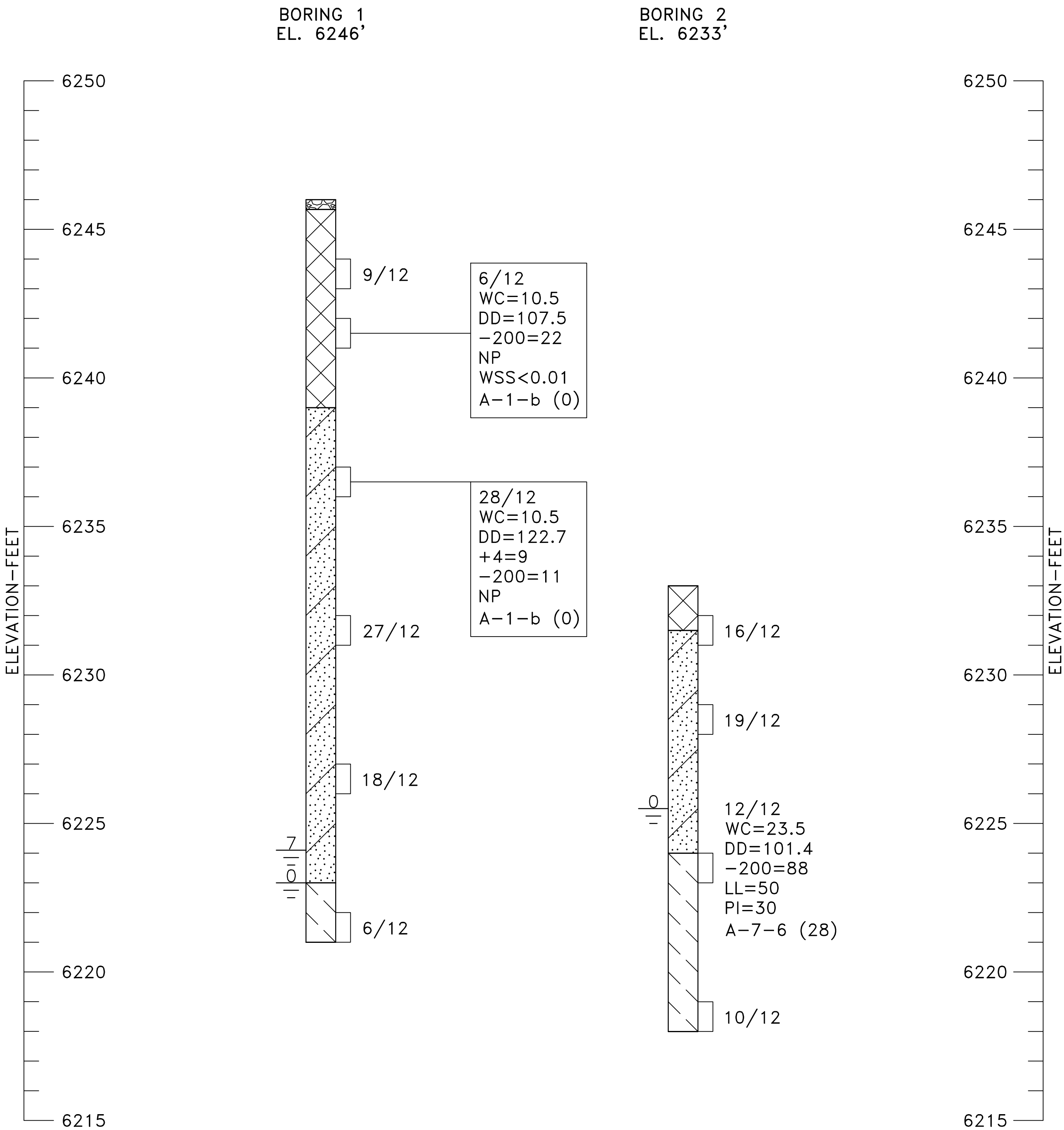




VICINITY MAP
NOT TO SCALE

22-2-146	Kumar & Associates	LOCATION OF EXPLORATORY BORINGS	Fig. 1

August 11, 2022 - 08:48am
V:\Projects\2022\22-2-146 New Equalization Tank and Pump Station Dry Pit for the Cherokee Metro District\Drafting\222146-01.dwg



22-2-146	Kumar & Associates	LOGS OF EXPLORATORY BORINGS	Fig. 2

August 11, 2022 - 08:48am
V:\Projects\2022\22-2-146 New Equalization Tank and Pump Station Dry Pit for the Cherokee Metro District\Drafting\222146-02 to 03.dwg

LEGEND

GRAVEL SURFACING.

FILL: SILTY SAND (SM) WITH SOME ZONES OF CLAYEY SAND (SC), FINE TO COARSE GRAINED, MOIST, LIGHT TO DARK BROWN.

WELL GRADED SAND WITH SILT (SW–SM), FINE TO COARSE GRAINED WITH GRAVEL, MEDIUM DENSE, MOIST TO WET, LIGHT BROWN TO BROWN.

FAT CLAY (CH), FINE GRAINED SAND FRACTION, MEDIUM STIFF TO STIFF, VERY MOIST, BROWN.

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

9/12 DRIVE SAMPLE BLOW COUNT. INDICATES THAT 9 BLOWS OF A 140–POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE THE SAMPLER 12 INCHES.

0 DEPTH TO WATER LEVEL AND NUMBER OF DAYS AFTER DRILLING MEASUREMENT WAS MADE.

NOTES

1. THE EXPLORATORY BORINGS WERE DRILLED ON JULY 15, 2022 WITH A 4–INCH–DIAMETER CONTINUOUS–FLIGHT POWER AUGER.

2. THE LOCATIONS OF THE EXPLORATORY BORINGS WERE MEASURED APPROXIMATELY BY HANDHELD GPS DEVICE.

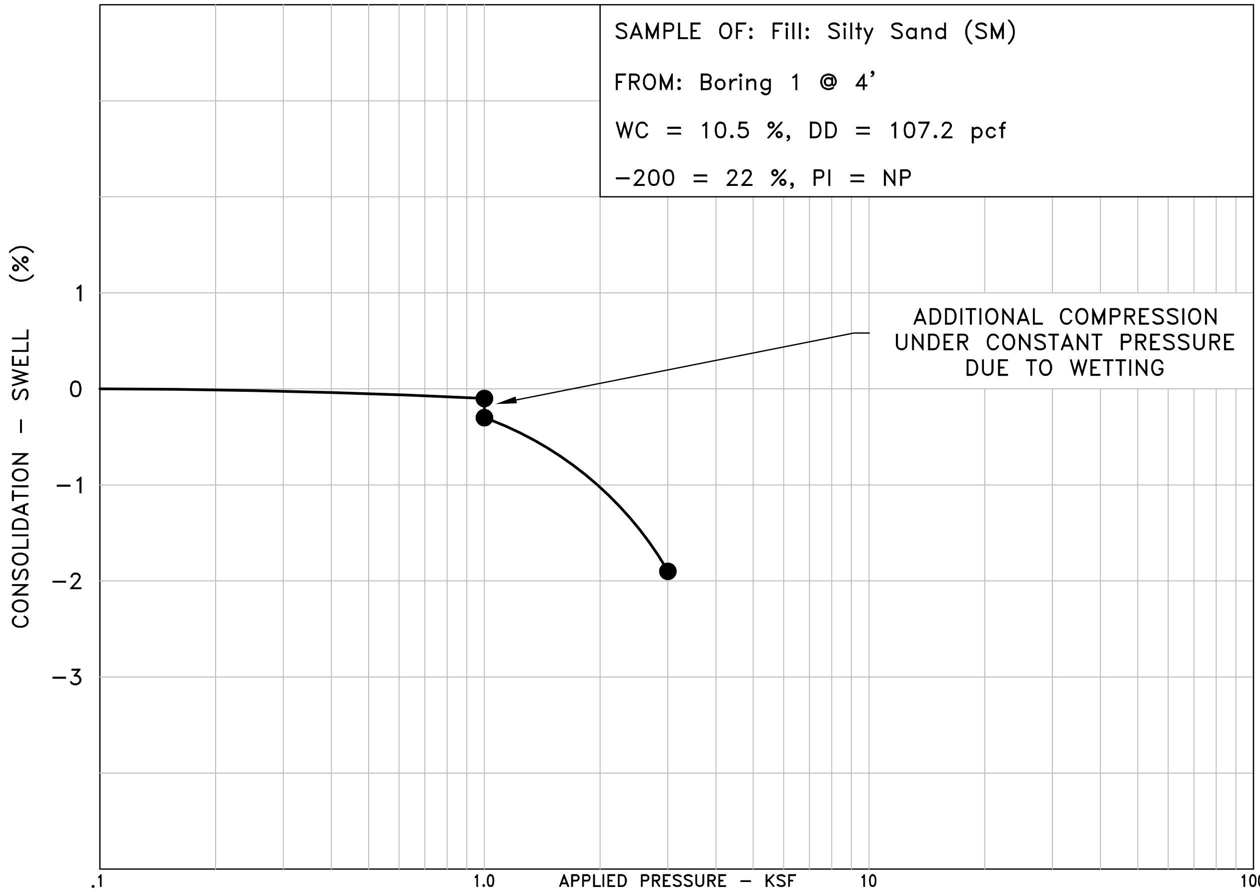
3. THE ELEVATIONS OF THE EXPLORATORY BORINGS WERE OBTAINED BY INTERPOLATION BETWEEN CONTOURS ON THE SITE PLAN PROVIDED.

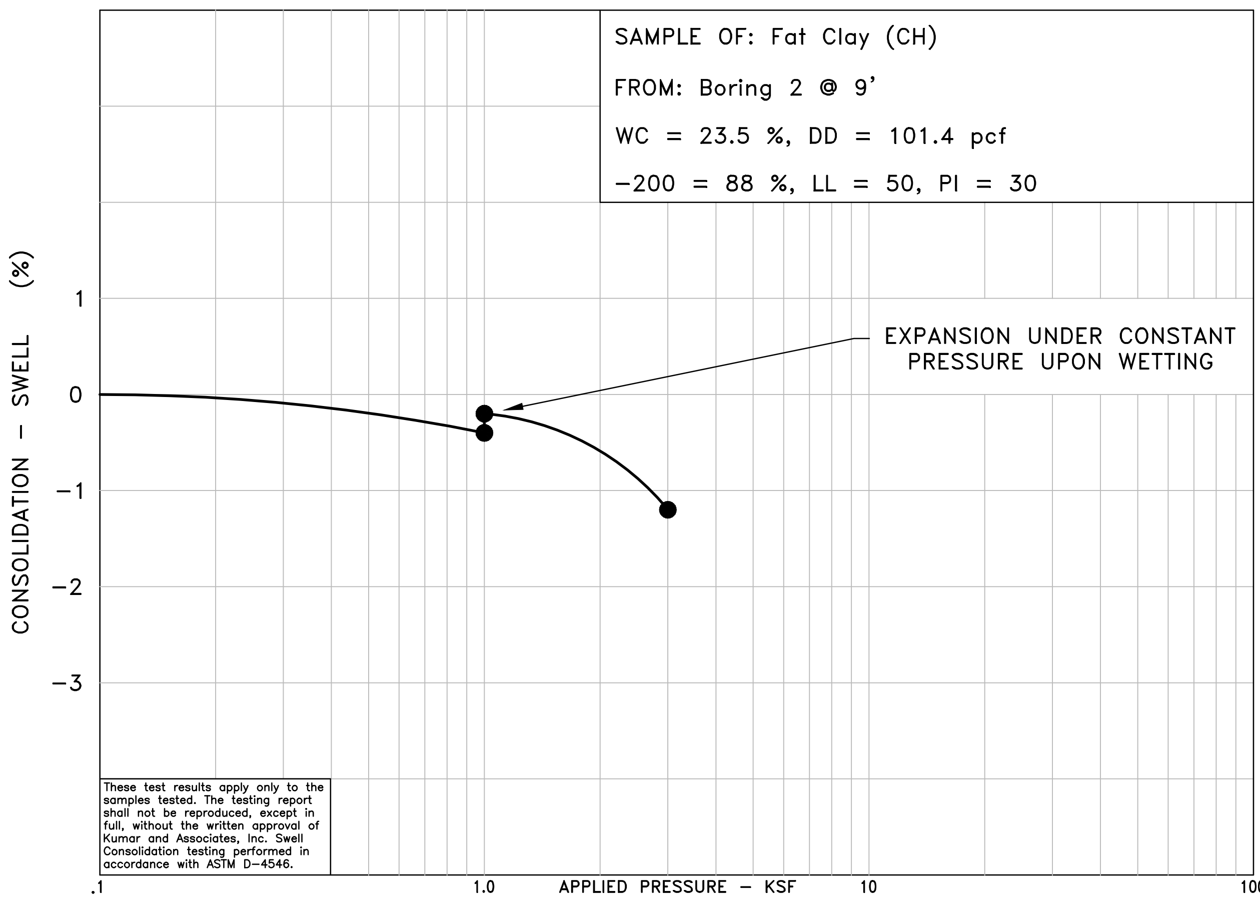
4. THE EXPLORATORY BORING LOCATIONS AND ELEVATIONS SHOULD BE CONSIDERED ACCURATE ONLY TO THE DEGREE IMPLIED BY THE METHOD USED.

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.

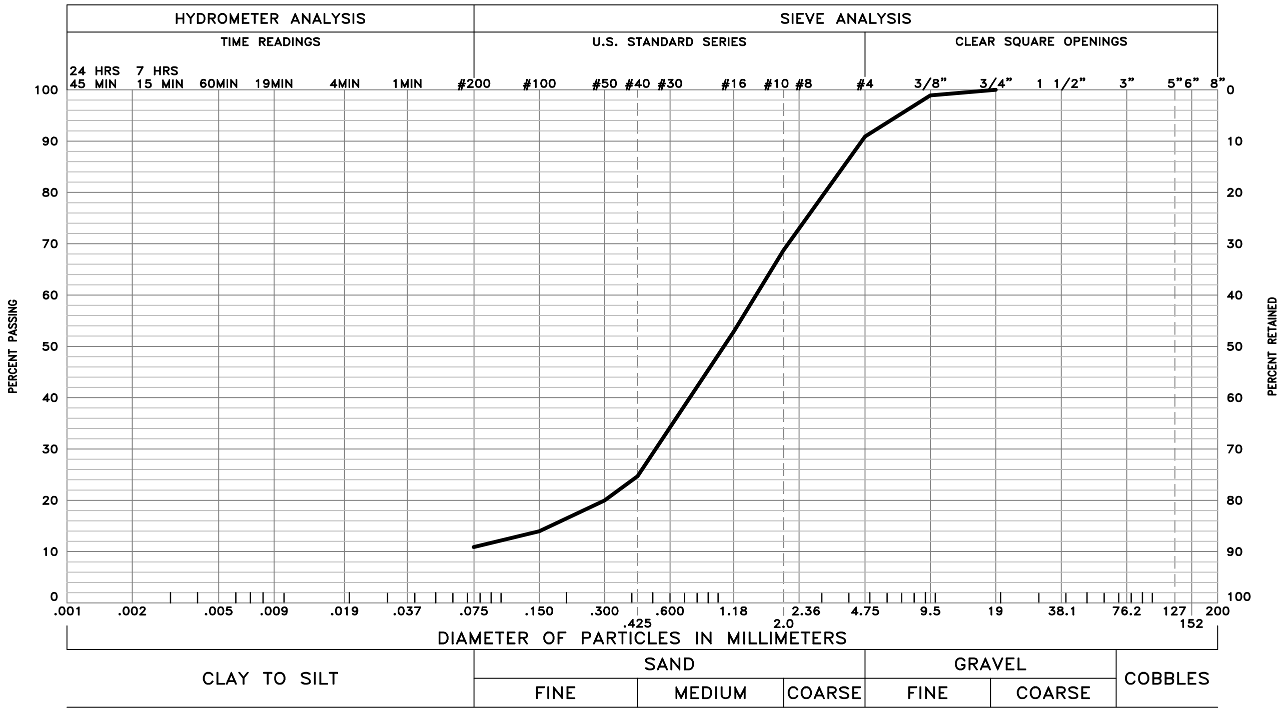
7. LABORATORY TEST RESULTS:
   WC = WATER CONTENT (%) (ASTM D2216);
   DD = DRY DENSITY (pcf) (ASTM D2216);
   +4 = PERCENTAGE RETAINED ON NO. 4 SIEVE (ASTM D6913);
   –200= PERCENTAGE PASSING NO. 200 SIEVE (ASTM D1140);
   LL = LIQUID LIMIT (ASTM D4318);
   PI = PLASTICITY INDEX (ASTM D4318);
   NP = NON–PLASTIC (ASTM D4318);
   WSS = WATER SOLUBLE SULFATES (%) (CP–L 2103);
   A-1-b (0) = AASHTO CLASSIFICATION (GROUP INDEX) (AASHTO M 145).





August 11, 2022 - 08:48am
V:\Projects\2022\22-2-146 New Equalization Tank and Pump Station Dry Pit for the Cherokee Metro District\Drafting\222146-04.dwg

22-2-146	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULTS	Fig. 4



GRAVEL 9 % SAND 80 % SILT AND CLAY 11 %

LIQUID LIMIT – PLASTICITY INDEX NP

SAMPLE OF: Well Graded Sand with Silt (SW–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.

22–2–146	Kumar & Associates	GRADATION TEST RESULTS	Fig. 5

Kumar and Associates, Inc.

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

Project No.: 22-2-146

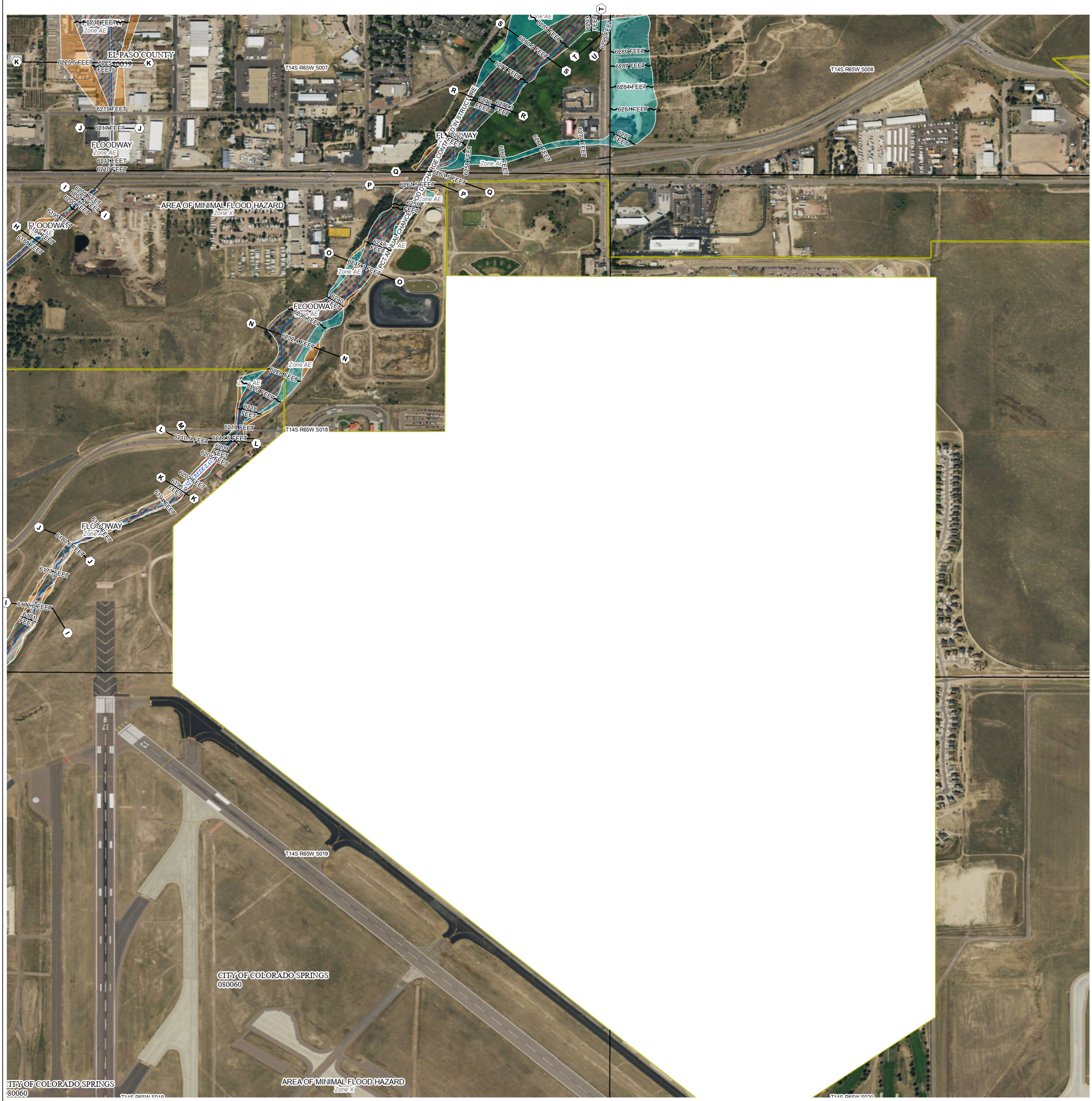
Project Name: Cherokee Metro

Date Sampled: 7/15/22

Date Received: 7/18/22

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	4	7/29/22	10.5	107.2			22		NP	<0.01	A-1-b (0)	Fill: Silty Sand (SM)
1	9	7/29/22	10.5	122.7	9	80	11		NP		A-1-b (0)	Well Graded Sand with Silt (SW-SM)
2	9	7/29/22	23.5	101.4			88	50	30		A-7-6 (28)	Fat Clay (CH)

104°43'7.96"W 38°50'44.11"N



104°41'14.61"W 38°48'38.43"N

FLOOD HAZARD INFORMATION

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR DRAFT FIRM PANEL LAYOUT

SPECIAL FLOOD HAZARD AREAS	Without Base Flood Elevation (BFE) *Zone A, V, A99*
	With BFE or Depth *Zone AE, AO, AH, VE, AR*
	Regulatory Floodway
OTHER AREAS OF FLOOD HAZARD	0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
	Future Conditions 1% Annual Chance Flood Hazard *Zone X*
	Area with Reduced Flood Risk due to Levee See Notes *Zone X*
	Area with Flood Risk due to Levee *Zone D*
OTHER AREAS	NO SCREEN Area of Minimal Flood Hazard *Zone X*
	Effective LOMRs
	Area of Undetermined Flood Hazard *Zone D*
GENERAL STRUCTURES	Channel, Culvert, or Storm Sewer
	Levee, Dike, or Floodwall
OTHER FEATURES	B 20.2 Cross Sections with 1% Annual Chance
	17.5 Water Surface Elevation
	8 Coastal Transect
	Coastal Transect Baseline
	Profile Baseline
	Hydrographic Feature
	513 Base Flood Elevation Line (BFE)
	Limit of Study
	Jurisdiction Boundary

NOTES TO USERS

For information and questions about this Flood Insurance Rate Map (FIRM), available products associated with this FIRM, including historic versions, the current map date for each FIRM panel, how to order products, or the National Flood Insurance Program (NFIP) in general, please call the FEMA Map Information eXchange at 1-877-FEMA-MAP (1-877-336-2627) or visit the FEMA Flood Map Service Center website at https://msc.fema.gov. Available products may include previously issued Letters of Map Change, a Flood Insurance Study Report, and/or digital versions of this map. Many of these products can be ordered or obtained directly from the website.

Communities annexing land on adjacent FIRM panels must obtain a current copy of the adjacent panel as well as the current FIRM Index. These may be ordered directly from the Flood Map Service Center at the number listed above.

For community and countywide map dates, refer to the Flood Insurance Study Report for this jurisdiction.

To determine if flood insurance is available in this community, contact your Insurance agent or call the National Flood Insurance Program at 1-800-638-6620.

Basemap information shown on this FIRM was provided in digital format by the United States Geological Survey (USGS). The basemap shown is the USGS National Map: Orthoimagery. Last refreshed October, 2020.

This map was exported from FEMA's National Flood Hazard Layer (NFHL) on **11/11/2022 3:48 PM** and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time. For additional information, please see the Flood Hazard Mapping Updates Overview Fact Sheet at https://www.fema.gov/media-library/assets/documents/118418

This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The basemap shown complies with FEMA's basemap accuracy standards. This map image is void if the one or more of the following map elements do not appear: basemap imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date.

SCALE

Map Projection:
GCS, Geodetic Reference System 1980;
Vertical Datum: NAVD88

For information about the specific vertical datum for elevation features, datum conversions, or vertical monuments used to create this map, please see the Flood Insurance Study (FIS) Report for your community at https://msc.fema.gov

N

1 inch = 500 feet 1:6,000

0 250 500 1,000 1,500 2,000 Feet

0 50 100 200 300 400 Meters

FEMA

National Flood Insurance Program

NATIONAL FLOOD INSURANCE PROGRAM

FLOOD INSURANCE RATE MAP

PANEL **754** OF **1275**

Panel Contains:

COMMUNITY	NUMBER	PANEL
PETERSON AIR FORCE BASE	08FED	0754
CITY OF COLORADO SPRINGS	080060	0754
EL PASO COUNTY	080059	0754

MAP NUMBER
08041C0754G
EFFECTIVE DATE
December 07, 2018