GEOTECHNICAL ENGINEERING REPORT

BOOSTER II - ELEVATED WATER STORAGE TANK

MULROY PROPERTY

SPRINGFIELD, MISSOURI

Prepared for:

CITY UTILITIES OF SPRINGFIELD
P.O. Box 551
Springfield, Missouri 65801

Prepared by:

- Geotechnical Services
- Environmental Services
- Material Testing Services

PALMERTON & PARRISH, INC.

Springfield, MO
4168 W. Kearney Springfield, MO 65803
Call 417.864.6000 Fax 417.864.6004
www.ppimo.com

PPI PROJECT NUMBER: 242540

April 28, 2017

PPI
PALMERTON & PARRISH, INC.

GEOTECHNICAL & MATERIALS ENGINEERS
MATERIALS TESTING LABORATORIES
ENVIRONMENTAL SERVICES

4168 W. Kearney Street.
Springfield, MO 65803
Ph: (417) 864-6000
www.ppimo.com

April 28, 2017

City Utilities of Springfield
P.O. Box 551
Springfield, Missouri 65801

Attn: Mr. Cody Marshall, P.E.
Email: cody.marshall@cityutiltities.net

RE: Geotechnical Engineering Report
Booster II – Elevated Water Storage Tank – Mulroy Property
Springfield, Missouri
PPI Project Number: 242540

Dear Mr. Marshall:

Attached, please find the report summarizing the results of the geotechnical investigation conducted for the above-referenced project. We appreciate this opportunity to be of service. If you have any questions, please don't hesitate to contact this office.

PALMERTON & PARRISH, INC.
By:

BRANDON ROSS PARRISH
NUMBER
PE-2010000852
4/28/17

Brandon R. Parrish, P.E.
Vice-President

PALMERTON & PARRISH, INC.
By:

Brad R. Parrish, P.E.
President

Submitted: One (1) Electronic .pdf Copy

BRP/BRP/TLA/jrh

cc: Mr. Kem Reed (kem.reed@cityutilities.net)
Mr. Clark McLemore (clark.mclemore@cityutilities.net)
Mr. James Okumu (james.okumu@cityutilities.net)

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EXECUTIVE SUMMARY

A Geotechnical Investigation was performed at the site planned for construction of the new Booster II Elevated Water Storage Tank located on the existing City Utilities of Springfield (CU) Mulroy Property in Springfield, Missouri. The new Water Storage Tank is anticipated to consist of a steel reservoir with a diameter ranging from 60 to 74 ft. and a concrete pedestal ranging from 36 to 40 ft. in diameter. The New Tank is anticipated to have a capacity of approximately 750,000 to 1 million gallons. Foundation loadings, both compressive and overturning are anticipated to be heavy. Minimal depths of cut and/or fill are anticipated to be required to provide finish subgrade elevation at the project site.

A total of five (5) sample/core borings were drilled around the perimeter and at the center of the proposed tank pedestal. Limestone bedrock was encountered within all borings at depths ranging from 14.8 to 24 ft. below the existing ground surface. Based upon the information obtained from the borings and subsequent laboratory testing, the site is suitable for construction of the proposed Water Storage Tank. Important geotechnical considerations for the project are summarized below. However, users of the information contained in the report must review the entire report for specific details pertinent to geotechnical design considerations.

- Depending upon the amount of tolerable settlement of the tank structure, the proposed structure may be supported upon shallow foundations bearing on medium stiff to stiff natural soils, rock fill, or deep foundations in the form of drilled piers bearing in limestone;
- Limestone bedrock was encountered within all boring locations at depths ranging from 14.7 to 24 ft. below the existing ground surface, indicating a pinnacled nature of the limestone bedrock. **Bedrock and depth to bedrock information are presented in Section 6.1 of this report**;
- The project site classifies as a Site Class C in accordance with Section 1613 of the 2012 International Building Code (IBC);

EXECUTIVE SUMMARY (CONTINUED)

- The project site has been used for agricultural purposes for many years and grass covered topsoil underlain by silty lean clays were encountered at the project site. These surficial lean clays containing little to no rock content are considered moisture sensitive and may undergo a loss of shear strength upon an increase in soil moisture or when disturbed by heavy construction equipment. These lean clays are anticipated to be removed during tank foundation installation. However, they will be exposed at the subgrade surface in areas surrounding the tank, as well as the Entrance Drive;

- Although no evidence of sinkholes were found on the immediate tank site, densely populated sinkholes were found to the north of the tank site along the Danforth Graben, an ancient inactive fault. Results of this 2015 study are depicted in Figure 3; and

- Palmerton & Parrish, Inc. should be retained for construction observation and construction materials testing. Close monitoring of subgrade preparation work is considered critical to achieve adequate foundation and subgrade performance.

GEOTECHNICAL ENGINEERING REPORT
BOOSTER II - ELEVATED WATER STORAGE TANK
MULROY PROPERTY
SPRINGFIELD, MISSOURI

1.0 INTRODUCTION

This is the report of the Geotechnical Investigation performed at the site planned for construction of the new Booster II Elevated Water Storage Tank located west of the Farm Road 116 and Mulroy Road intersection in Springfield, Missouri. This investigation was performed in general accordance with the proposal prepared by Palmerton & Parrish, Inc. (PPI) dated March 23, 2017, and performed under the Blanket Agreement for Geotechnical Consultant Services Contract between CU and PPI. The approximate site location is shown in the aerial photograph below for reference.

The purpose of the Geotechnical Investigation was to provide information for foundation design and construction planning and to aid in site development. Palmerton & Parrish Inc.'s (PPI) scope of services included field and laboratory investigation of the subsurface conditions in the vicinity of the proposed project site, engineering analysis of the collected data, development of recommendations for foundation design and construction planning, and preparation of this engineering report.

2.0 PROJECT DESCRIPTION

Item	Description
Site Layout	See Figure 1: Boring Location Plan
Structure	New elevated Water Storage Tank with an approximate reservoir diameter of 60 to 74 ft. and supported upon a concrete pedestal approximately 36 to 40 ft. in diameter. Tank height is unknown. The tank is anticipated to have a 750,000 to 1 million gallon storage capacity.
Foundation Loadings	Compressive and overturning loadings are anticipated to be heavy.
Grading	Minimal depths of cut and/or fill are anticipated to provide finish subgrade elevations outside the tank footprint, while several feet of cut is anticipated within the pedestal footprint to provide adequate bearing for the tank foundations.

3.0 SITE DESCRIPTION

Item	Description
Latitude/Longitude (± Center of Project Site)	37°14'25"N / -93°11'10"W
Ground Elevation at Center of Proposed Tank	1457.04
Existing Improvements	None known.
Current Ground Cover	Grass covered open field with occasional mature trees.
Existing Topography	Located on a ridge with elevations decreasing on all sides of the tank footprint.
Drainage Characteristics	Fair.

4.0 SUBSURFACE INVESTIGATION

Subsurface conditions were investigated through completion of five (5) sample/core borings and subsequent laboratory testing.

4.1 Subsurface Borings

The planned boring locations were selected and staked in the field by PPI using the center of tank staked in the field by CU prior to drill-rig mobilization. Boring 2 was located at the center of the proposed tank, while the remaining borings were drilled around the proposed pedestal perimeter assuming a pedestal diameter of 40 ft. All borings were extended to top of limestone bedrock with rock core obtained within Borings 1 through 3. Approximate boring locations are shown on Figure 1: Boring Location Plan.

Bedrock was encountered within all boring locations at depths ranging from 14.7 to 24 ft. below the existing ground surface. Logs of the borings showing descriptions of soil and rock units encountered, as well as results of field and laboratory tests and a “Key to Symbols” are presented in Appendix I. The Missouri One-Call System was notified prior to the investigation to assist in locating buried public utilities.

Borings were drilled April 4 through 7, 2017 using either 4.25-inch I.D. hollow stem augers powered by a CME-55 track-mounted drill-rig. Soil samples were collected at 2.5 to 5-ft. centers during drilling. Soil sample types included split spoon samples collected while performing the Standard Penetration Test (SPT) in general accordance with ASTM D1586 and thin walled Shelby tubes pushed hydraulically in advance of drilling in accordance with ASTM D1587. After auger refusal upon bedrock within Borings 1 through 3, rock coring procedures were implemented using a 2-inch I.D. diamond impregnated core bit. Please refer to Appendix II for general notes regarding boring logs and additional soil sampling information.

4.2 Laboratory Testing

Collected samples were sealed and transported to the laboratory for further evaluation and visual examination. Laboratory soil test results included the following:

- Moisture Content (ASTM D2216);
- Unconfined Compressive Strength (ASTM D 2166);

- Atterberg Limits (ASTM D4318); and
- Pocket Penetrometers.

Laboratory test results are shown on each boring log in Appendix I and are summarized in the following table. Grain size anlaysis results are presented in Appendix III. In addition to the above laboratory testing, one (1) one-dimensional consolidation test was performed in accordance with ASTM D2435 on one (1) fat clay soil specimen to determine compressibility characteristics. Results of the consolidation tests are presented in Appendix IV.

Boring	**Depth (ft.)**	**Liquid Limit (LL)**	**Plastic Limit (PL)**	**Plasticity Index (PI)**	**% Passing No. 200 Sieve**	**Moisture Content (%)**	**USCS Symbol**	**Cohesion (psf)**	$\frac{C_c}{1+e_o}$	**Compressibility**
1	0 to 2	48	17	31	-	27.6	CL-CH	-	-	-
1	13.5 to 15	86	21	65	-	37.3	CH	-	-	-
2	3.5 to 5	28	20	8	50	15.8	CL	-	-	-
4	8.5 to 10	85	22	63	-	36.4	CH	-	-	-
4	18.5 to 20	-	-	-	-	46.9	CH	2770	-	-
5	3.5 to 5	-	-	-	37.8	15.7	GC	-	-	-
5	8.5 to 9.8	88	21	67	-	35.7	CH	1610	0.09	Slightly Compressible
5	13.5 to 15	87	23	64	-	44.4	CH	2750	-	-

5.0 SITE GEOLOGY

The general site area is underlain at depth by the Mississippian Age Burlington Limestone Formation. This unit characteristically consists of coarse-grained gray limestone, which is nearly pure calcium carbonate. Isolated chert nodules and discontinuous chert layers are present throughout the formation. The upper surface of this limestone unit is generally irregular due to the effects of differential vertical weathering and solution activity. Limestone pinnacles, some of which are 10 to 15 ft. high, are common in the general area. In upland areas, overburden soils are usually composed of red clay and chert and are residual having developed from physical and chemical weathering of the parent limestone. The chert fragments were interbedded with the limestone, but are much more resistant to weathering and retain rock-like properties. The contact between comparatively unweathered bedrock and the residual soils is usually abrupt.

The general site area is located within the Ozarks Physiographic Region of Missouri, which is characterized by rugged to rolling hill terrain, meandering streams, and karst topography. Karst topography forms over areas of carbonate bedrock where groundwater has solutionally enlarged openings to form a subsurface drainage system. Springs, caves, losing streams and sinkholes are common in karst areas. Sinkholes are defined as a depression in the landscape with an internal drainage system.

Based upon a review of readily available topographic contour maps and a site visit performed by Mr. Joshua Elson, R.G. with PPI, on April 10, 2017, no karst features were identified on the surface of the subject property. However, based upon previous studies on the north and northeast adjacent properties performed by PPI in 2015, the general area is extremely karst-prone and densely populated with sinkholes adjacent to the Danforth Graben, an ancient inactive fault. The locations of the sinkholes observed and approximate fault location on the north adjacent property are presented in Figure 3.

CU should be aware that it is possible for karst features to be encountered at the project site during construction. If a karst feature is identified during site grading, PPI should be contacted immediately for evaluation on a case-by-case basis.

6.0 GENERAL SITE & SUBSURFACE CONDITIONS

Based upon subsurface conditions encountered within the borings drilled at the project site, generalized subsurface conditions are summarized in the table below. Soil stratification lines on the boring logs indicate approximate boundary lines between different types of soil and rock units based upon observations made during drilling. In-situ transitions between soil and some rock types are typically gradual.

Description	Borings	Approx. Depth to Bottom of Stratum	Material Encountered	Moisture	Consistency/ Density
Stratum 1	All	0.5 ft.	Topsoil	Moist	Soft
Stratum 2	All, Except 1	2 to 5.5 ft.	Lean Clay w/Varying Amounts of Chert	Moist	Medium Stiff to Very Stiff
Stratum 3	1	2 ft.	Lean to Fat Clay w/Trace Chert	Moist	Medium Stiff
Stratum 4	1, 4 & 5	6 to 7.5 ft.	Clayey Gravel	Moist	Dense to Very Dense
Stratum 5	All	Top of Limestone	Fat Clay w/Varying Amounts of Chert	Moist	Medium Stiff to Very Stiff
Stratum 6	All	Boring Completion	Limestone	-	Moderately Hard

6.1 Limestone

Limestone was encountered within all borings drilled at depths ranging from 14.7 to 24 ft. below the existing ground surface. Following auger refusal within Borings 1 through 3, rock coring was accomplished to a depth of 5 ft. into competent continuous limestone. The limestone was primarily logged as gray, coarse crystalline, slightly weathered, moderately hard, medium to thick bedded with scattered chert layers and nodules. The following table summarizes groundwater and rock coring information. Percent recovery and RQD values obtained during rock coring ranged from 98 to 100 and 62 to 93 percent, respectively, indicating fair to excellent rock quality. Please refer to the table below for bedrock information. Photographs of rock core are presented in Appendix V.

Boring	Depth to Limestone (ft.)	REC RQD	Groundwater Depth During Drilling/At Completion (ft.)
1	23[1] (29.4)	100 62	None
2	14.7[2] (24.7)	100 80	None
3	19	98 93	None
4	24	-	None
5	22.3	-	None

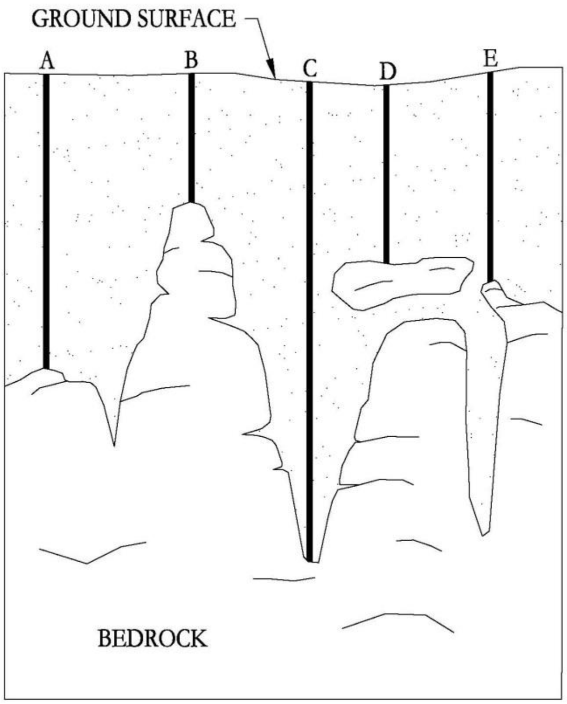
1. Clay layer encountered from 24 to 29.4 ft. Coring began at 29.4 ft.
2. Clay layer encountered from 15.8 to 24.7 ft. Coring began at 24.7 ft.

As indicated in the table above, depth to limestone was relatively inconsistent within all borings drilled. This differing depth to top of limestone as well as the potential "floater" or discontinuous bedrock (See Section 6.2 below) encountered within Borings 1 & 2 indicates a pinnacled nature of the limestone surface which is typical of the bedrock in the general site area. If deep foundations bearing in limestone bedrock are selected, several feet of limestone penetration should be anticipated to obtain a level rock bottom within some piers.

6.2 Auger Refusal

Auger refusal is defined as the depth below the ground surface at which a boring can no longer be advanced with the soil drilling technique being used. Auger refusal is subjective and is based upon the type of drilling equipment and types of augers being used, as well as the effort exerted by the driller. Several different auger refusal conditions are possible in the general site area. These conditions are represented graphically in the adjacent figure: (A) on the upper surface of continuous bedrock, (B) on rock "pinnacles", (C) in widened joints that may extend well below the surrounding bedrock surface, (D) slabs of unweathered rock suspended in the residual soil matrix, or "floaters", or (E) on the upper surface of discontinuous bedrock.

POSSIBLE AUGER REFUSAL CONDITIONS



THE BEDROCK CONDITIONS ILLUSTRATED ABOVE ARE FOR REFERENCE ONLY & DO NOT INDICATE CONDITIONS ENCOUNTERED AT THE PROJECT SITE.

Due to the possibility that some or all of these features exist at this project site, estimating the exact quantity of rock excavation is difficult. Linear interpolation of apparent bedrock elevations based upon the boring data is often used but can misrepresent actual rock removal quantities where such anomalies exist.

6.3 Groundwater

As presented in the table in Section 6.1, groundwater was not observed within the borings drilled above limestone on the date drilled. Due to the addition of coring water during the coring process, detection of a groundwater table below limestone could not be performed. Development of shallow perched groundwater is considered possible during wetter periods is considered possible especially at the overburden/bedrock interface. Groundwater levels should be expected to fluctuate with changes in site grading, precipitation, and regional groundwater levels. Groundwater may be encountered at shallower depths during wetter periods.

7.0 CONSTRUCTION AND DESIGN RECOMMENDATIONS

Based upon subsurface conditions encountered within the borings drilled and subsequent laboratory testing, the structure foundations may be designed as shallow or deep foundations. Shallow foundations consisting of a mat or ring-shaped footing should bear upon natural overburden soils or rock fill. Deep foundations in the form of drilled piers bearing in limestone bedrock may also be selected for foundation support. The alternate chosen for foundation support will depend upon the amount of settlement tolerable by the anticipated structure. Each foundation alternate and associated anticipated settlement are discussed in the below sections of this report.

8.0 EARTHWORK

It is anticipated that minimal depths of cut will be required to provide finish subgrade elevations across the site. The initial phase of site preparation should include clearing and grubbing of all organic and vegetative matter. Topsoil stripping on the order of 6 to 12 inches should be anticipated. Excavation depths on the order 4 ft. or more will be required for footing/mat foundation on natural overburden soils. It is recommended that all footing/mat subgrades be proof-rolled prior to foundation construction or placement of fill, if required, to assure a stable subgrade.

Proof-rolling consists essentially of rolling the ground surface with a loaded tandem axle dump truck or similar heavy rubber tired construction equipment and noting any areas which rut or deflect during rolling. All soft subgrade areas identified during proof-rolling should be undercut and replaced with compacted fill as outlined below. Proof-rolling, undercutting, and replacement should be monitored by a qualified representative of PPI. **The depth and areal extent of undercutting, if any, should be minimal but will be largely dependent upon the time of year and related soil moisture conditions. If construction is initiated during wetter spring or winter months, the requirement for undercutting soft surficial soils below normal topsoil stripping should be anticipated and reflected in contract documents. As previously mentioned, lean clays at the project site are moisture sensitive and may pose difficulties regarding subgrade stability and proper compaction.**

After evaluation by proof-rolling, and approval, the subgrade should be scarified to a depth of at least 8 inches, adjusted to within 2 percent of optimum moisture content and compacted to the specified density.

8.1 Rock Fill

If chosen, rock fill placed beneath foundations should be constructed using rock having maximum dimensions in excess of 4 inches, but no greater than 8 inches. Rock material should be placed in horizontal layers having a thickness of approximately the maximum size of the larger rock comprising the lift, but not greater than 8 inches. Rocks or boulders too large to permit placing in an 8-inch thick lift should be reduced in size as necessary to permit placement or not used in the compacted fill. Rock fill should not be dumped into place but should be distributed in horizontal lifts by blading and dozing in such a manner as to ensure proper placement into final position in the excavation. Finer material including rock fines and limited soil fines should be worked into the rock voids during this blading operation. Excessive soil and rock fine particles preventing interlock of cobble and boulder sized rock should be prohibited. **Rock fill should be compacted by a minimum of 4 passes by a large diameter self-propelled vibratory roller.**

The testing of rock fill quality should include the requirements that a representative of PPI be present daily, but not necessarily continuously during the placement of the fill to observe the placement of rock fill in order to determine fill quality and to observe that the contractors work sequence is in compliance with this specification. Progress reports indicative of the quality of the fill should be made at regular intervals to the Owner. If improper placement procedures are observed during the placement of the fill, the Geotechnical Engineer should inform the Contractor and no additional fill should be permitted on the affected area until the condition causing the low densities has been corrected and the fill has been reworked to obtain sufficient density.

Subgrade inspection and fill monitoring under controlled conditions is considered essential if footings/mat and interior slab are to be founded on fill.

Once rock backfill is complete, a 6-inch thick quarry produced crushed stone leveling pad should be constructed above rock fill, below the interior slab. The gradation of this material should conform to MODOT Type 1 aggregate or similar baserock material typically specified by tank designers and compacted to 100 percent Standard Proctor (ASTM D698) density. Additional moisture may be required to achieve this compaction.

In lieu of rock fill, stone backfill may also consist of MODOT Type 1 aggregate base compacted to at least 100% of maximum Standard Proctor Density. It should be noted that the use of this material will require greater effort during placement, as maximum lift thickness should not exceed 6 inches. Field density tests and documentation of adequate compaction for each lift will also be required if crushed stone fill is chosen.

8.2 Exterior Backfill

Earth fill to be placed above footings/mat to provide finish grade elevations around the tank foundation should consist of inorganic low plasticity lean clay with or without chert. Large size rock greater than 6 inches inhibits fill compaction and should be generally excluded from controlled fill areas. Fill should be placed in no greater than

8 inch loose lifts and compacted to at least 95 percent of maximum density as determined by Standard Proctor Procedures (ASTM D698) provided no future structures are planned. Soil moisture adjustment may be required to achieve specified compacted density. Adequate field density and moisture content tests should be performed to ensure compliance with project specifications. It is believed that soils removed from the undercut below topsoil should generally be suitable for reuse as exterior backfill provided soil moisture is near optimum. Topsoil is not considered adequate for structural fill and should be placed in landscape areas only.

8.3 Site Drainage

Surface flow from the surrounding topography should be diverted well away from the foundation perimeter. Rapid, efficient runoff away from the proposed foundations should also be provided.

8.4 Excavations

Based upon the subsurface conditions encountered during this investigation, the on-site soils located above limestone bedrock generally classify as OSHA Type B, which require a 1H:1V backslope. The Geotechnical Engineer should be notified for review of the excavated slopes to assist in determination of OSHA soil types. It should also be mentioned that the backslopes provided in OSHA regulations are limited to excavations not exceeding 20 ft. A professional engineer with knowledge and expertise in soils engineering is required for evaluation of excavated slopes deeper than 20 ft. Field verification of soil type and excavation safety concerns are the responsibility of the contractor.

9.0 FOUNDATIONS

Shallow and deep foundation recommendations are provided in the following sections. The foundation alternate chosen will be dependent upon the tolerable settlement allowed by the tank designer. A summary of anticipated settlements and foundation recommendations for shallow and deep foundations are presented in the following sections.

9.1 Foundation Alternate Settlement Summary

A summary of the computed and estimated settlements for each foundation alternate presented in the following sections are presented in the table below. Settlements for shallow foundations on natural soils are within the maximum allowable value for total settlement and differential settlement. Minimal, if any, total settlement and differential settlement is anticipated for a tank founded upon drilled piers to limestone. Depending upon the amount of tolerable settlement, shallow or deep foundations may be utilized.

Criteria	Maximum Allowable (from Previous CU Tank Projects)	Option 9.2 Shallow Foundation On Natural Soil	Option 9.3 Shallow Foundation On Rock Fill	Option 9.5 Drilled Piers to Limestone
Shallow Foundation Total Settlement	3.0”	2.0”	1.0”	-
Deep Foundation Total Settlement	0.75”	-	-	Less than maximum allowable
Differential Settlement	1.5”	0.5”	0.5”	Less than maximum allowable

Based upon the amount of foundation settlement tolerable by the proposed structure, foundation recommendations are provided in the following sections.

9.2 Shallow Foundations on Natural Soils

Based upon subsurface conditions encountered within all borings, and if anticipated settlements can be tolerated, installation of a shallow mat or ring wall type footings founded upon medium stiff to stiff natural foundation soils should provide satisfactory support for foundation loads anticipated. The footing/mat should be founded at least 3 feet below existing grade on stiff fat clay or chert. In addition, the footing/mat should be founded at least 2.5 feet below finish grade in order to limit the effects of frost penetration and seasonal variation in soil moisture. A footing/mat founded as outlined above may be sized using a net allowable bearing pressure of **4,000 psf.**

9.3 Shallow Foundations on Rock Fill

If the estimated settlements (total and differential) as presented in Section 9.1 cannot be tolerated, an alternate that may be considered in lieu of shallow foundations on natural soil is the partial removal and replacement of existing soils from within the proposed foundation area. Removing and replacing existing soils with rock fill reduces the potential for total and differential settlements. Under this alternate, existing soils beneath tank footings or mat should be removed to a minimum depth of 6 ft. below proposed foundation bottom. Soil removal should extend horizontally beyond footing or mat perimeter a distance of at least 5 ft. and the excavation backsloped to at least a 2V:1H. Undercuts should then be backfilled to provide, at a minimum, a 6 ft. thick rock fill pad below footing or mat bottoms.

Footings or a mat under this alternate should be founded at least 2.5 ft. below final exterior grade for frost protection. Footings or a mat founded as outlined above on rock fill may be sized using a net allowable bearing pressure of 6000 psf.

Settlements for shallow foundations on rock fill were computed assuming soil removal to a depth of 6 ft. below foundation bottom, and backfilling with rock fill. The rock fill was assumed to be 6 ft. thick and placed directly below footing bottoms. Predicted maximum total and differential foundation settlements under this alternate are approximately 1.0 inches and 0.5 inches, respectively. These settlement values are within the tolerable ranges discussed in Section 9.1 above.

9.4 Uplift Resistance

Resistance of shallow spread footings to uplift (Up) may be based upon the dead weight of the concrete footing structure (W_C) and the weight of soil backfill contained in an inverted cone or pyramid directly above the footings (W_S).

The weight of the concrete footing structure (W_C) may be computed using a concrete unit weight of 150 pounds per cubic foot (pcf).

The weight of the soil resistance (W_S) may be computed using a moist soil unit weight of 100 pcf, for on-site soils compacted to at least 95% of maximum dry density as determined by the Standard Proctor compaction test (ASTM D 698). The base of the cone or pyramid should be the top of the footing and the pyramid or cone sides should form an angle of 30 degrees with the vertical.

Allowable uplift capacity should be computed as the lesser of the two equations listed below:

$$U_P = (W_S/2.0) + (W_C/1.25);\ \text{or}$$

$$U_P = (W_S + W_C)/1.5$$

9.5 Drilled Piers to Limestone

In view of the heavy loads transmitted by the elevated water storage tank and if limited settlement cannot be tolerated by the foundations, piping, and connections, it is recommended that the water storage tank be supported upon deep foundations bearing in limestone bedrock.

Piers for this project should have straight shafts and should be founded at least 1 ft. into sound continuous limestone, or a depth required to resist uplift and lateral loading anticipated. **It should be emphasized that the upper few feet of the Burlington limestone can be quite irregular or pinnacled with clay seams, so the piering contractor should be capable of removing a few to several feet of limestone to provide a level pier bottom with adequate rock socket.** In addition, a large clay seam was encountered below a “floater” or discontinuous bedrock within Borings 1 and 2. Pier shafts in the area of Borings 1 and 2 should be founded in limestone below the clay seam. **It is recommended that prior to drilled pier construction, PPI, or the drilled pier contractor, drill probe borings into the shallow limestone in the area of Borings 4 and 5 to verify that the limestone is continuous, and is not a large “floater” or discontinuous bedrock as was encountered within Borings 1 and 2.**

Pier shafts should have a relatively flat pier bottom, plumb pier shaft and competent bearing rock consistent with the recommended bearing pressure and removal of essentially all groundwater prior to concrete placement. With pier installation as outlined in this report, an allowable end bearing pressure of 20 kips per sq. ft. (ksf) may be used in pier design.

A higher allowable end bearing pressure of 60 ksf can be used for pier design if proof-testing is performed as described below. The limestone in each pier bottom should be evaluated by drilling a 2-inch diameter probe boring to a depth of 1.5 times the pier diameter or 5 ft. below the pier bottom, whichever is greater. Probe borings should be scraped with a right angle chisel point (scratch tested) by a representative of PPI to verify the bearing rock is free of clay seams and/or cavities. Pier shafts should also be inspected prior to concrete placement to verify a relatively level pier bottom and plumb pier shaft. Please refer to Appendix VI for additional recommendations for inclusion in pier design specifications.

Pier drilling contractors should be prepared to provide casing capable of being screwed into the limestone and limiting groundwater inflow into pier shafts. Pier shafts should be dewatered to a maximum water depth of 2 to 3 inches prior to concreting or a tremie used for concrete placement.

Drilling a few additional feet into limestone to obtain sound bearing rock free of clay seams or layers has been the experience of our firm on similar projects within the Burlington Limestone formation. A common area of dispute on drilled pier projects is often related to the volume of rock removal or interpretation of a “rock clause” in the construction contract. A means of limiting this type of dispute which has been employed on past projects is to establish a plan pier bottom elevation based upon results of geotechnical borings or additional core borings drilled after pier layout is finalized. The piering contractor is then required to submit a base bid using plan pier bottom elevations plus a per foot unit price for pier drilling below plan bottom. An estimated footage of rock drilling below plan bottom elevations is then added to the base bid for comparison of bids. Elevation of pier bottoms is recorded upon pier inspection reports and used to determine extra for rock drilling based only on elevations. Another alternative is to pay for rock drilling as extra work but compare bids based upon an estimated quantity of rock drilling. Rock drilling must be well defined in piering specifications under this alternate.

Another area of dispute has been related to the preference of a piering contractor to over-drill the limestone bedrock to facilitate removal of the limestone core with extra payment for this rock drilling. This method of rock drilling often produces a 2 or 3 ft. rock socket when only a 1 ft. socket is required. **Pier specifications should clearly state that no payment will be made for deepening of piers below the required 1 ft. rock socket when performed for the contractor's convenience to facilitate rock removal**. **In any event, it is strongly recommended that PPI be retained to review the pier specifications and bid document for drilled piers prior to bid letting.**

9.5.1 Lateral Loadings for Drilled Piers

It is anticipated that resistance of a drilled pier foundation to lateral loading and the associated lateral deflection will be evaluated using finite difference computer models based on the horizontal modulus of subgrade reaction (K_h). The following values may be used in the analysis for this site. Please note the following table states to "ignore" lateral support for the depth from 0 to 1 pier diameter of 2.5 ft., whichever is shallower. This notation is intended to account for the fact that near-surface soils are significantly disturbed during drilled shaft excavation, greatly reducing the lateral support provided. Designers should use their judgment and make an appropriate reduction of soil strength parameters in this zone.

Depth	Static K_h (pci)	Cyclic K_h (pci)	Cohesion (S_u) ksf	ε_{50}
0-1 Shaft Diameter or 2.5 ft., whichever is shallower	Ignore	-	-	-
1 Shaft Diameter or 2.5 ft., whichever is shallower, to Top of Lower Limestone Unit	1000	400	1.5	0.005
Limestone (Not Proof Tested)	2000	800	4	0.0005
Limestone (Proof Tested)	>2000	>800	>50	0.0005

The above values were measured in the laboratory or based upon published correlations with soil strength and classification tests.

9.5.2 Uplift Resistance for Drilled Piers

For resistance to uplift, side friction may also be applied to the sidewalls, with the exception of the top 1 times the diameter of the drilled shaft or 2.5 ft. whichever is shallower. An allowable side friction of 400 psf may be used for the overburden soils down to top of limestone. An allowable side friction of 2.0 ksf may be used for shaft length embedded into limestone. However, due to the top of the limestone often being weathered, the upper 1 ft. of the rock socket should be ignored. Side friction may also be used to resist compressive loads.

10.0 SEISMIC CONSIDERATIONS

Code Used	Site Classification
2012 International Building Code (IBC)[1]	C
1. In general accordance with Section 1613 of the *2012 International Building Code.*	

10.1 Site Amplification Factor

According to AWWA D-100, a Soil Profile “A” should be used for stiff soil conditions where the soil depth is less than 200 feet and the soil types overlying rock are stable deposits of sands, gravels or stiff clays. Given the subsurface conditions encountered and the presence of shallow bedrock, a soil profile “A” should be used in tank design.

11.0 CONSTRUCTION OBSERVATION & TESTING

The construction process is an integral design component with respect to the geotechnical aspects of a project. Since geotechnical engineering is influenced by variable depositional and weathering processes and because we sample only a small portion of the soils affecting the performance of the proposed structures, unanticipated or changed conditions can be disclosed during grading. Proper geotechnical observation and testing during construction is imperative to allow the Geotechnical Engineer the opportunity to evaluate assumptions made during the design process. Therefore, we recommend that PPI be kept apprised of design modifications and construction schedule of the proposed project to observe compliance with the design concepts and geotechnical recommendations and to allow design changes in the event that subsurface conditions or methods of construction differ from those assumed while completing this study. We recommend that during construction all earthwork be monitored by a representative of PPI, including site preparation, placement of all engineered fill and trench backfill, and all foundation excavations as outlined below.

- An experienced Geotechnical Engineer or Engineering Technician of PPI should observe the subgrade throughout the proposed project site immediately following stripping to evaluate the native clay, identify areas requiring additional undercutting, and evaluate the suitability of the exposed surface for fill placement;

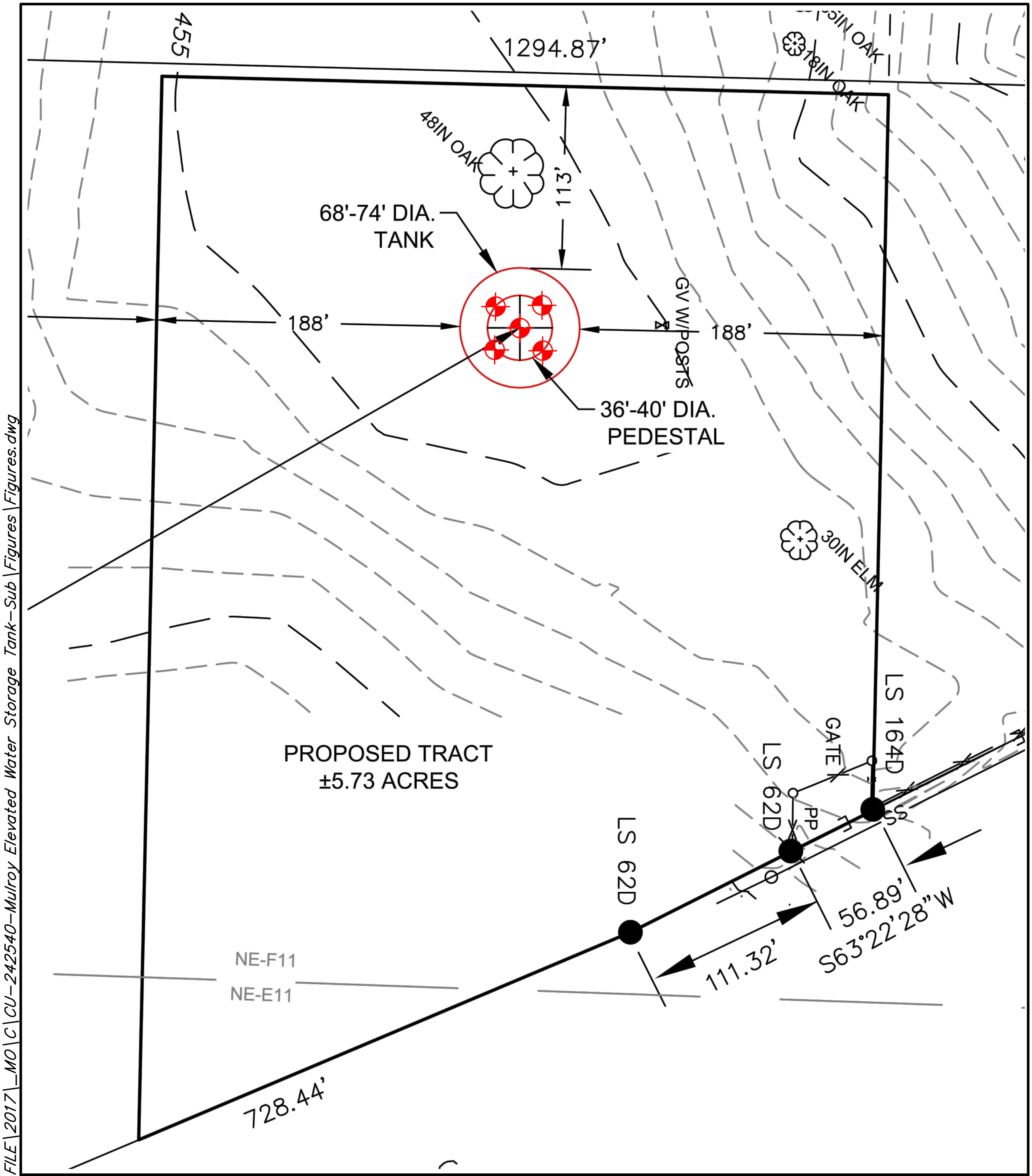
- An experienced Engineering Technician of PPI should monitor and test all fill placed within the building and pavement areas to determine whether the type of material, moisture content, and degree of compaction are within recommended limits;

- An experienced Technician or Engineer of PPI should observe and test all footing excavations. Where unsuitable bearing conditions are observed, remedial procedures can be established in the field to avoid construction delays; and
- The condition of the subgrade should be evaluated immediately prior to construction of the building floor slabs to determine whether the moisture content and relative density of the subgrade soils are as recommended.

12.0 REPORT LIMITATIONS

This report has been prepared in accordance with generally accepted practices of other consultants undertaking similar studies at the same time and in the same geographical area. Palmerton & Parrish, Inc. observed that degree of care and skill generally exercised by other consultants under similar circumstances and conditions. Palmerton & Parrish's findings and conclusions must be considered not as scientific certainties, but as opinions based on our professional judgment concerning the significance of the data gathered during the course of this investigation. Other than this, no warranty is implied or intended.

FIGURE



LEGEND

🔴 Boring Location

SCALE
1" = 80'

Project: Booster II - Elevated Water Storage Tank
Client: City Utilities of Springfield

Boring Location Plan 1

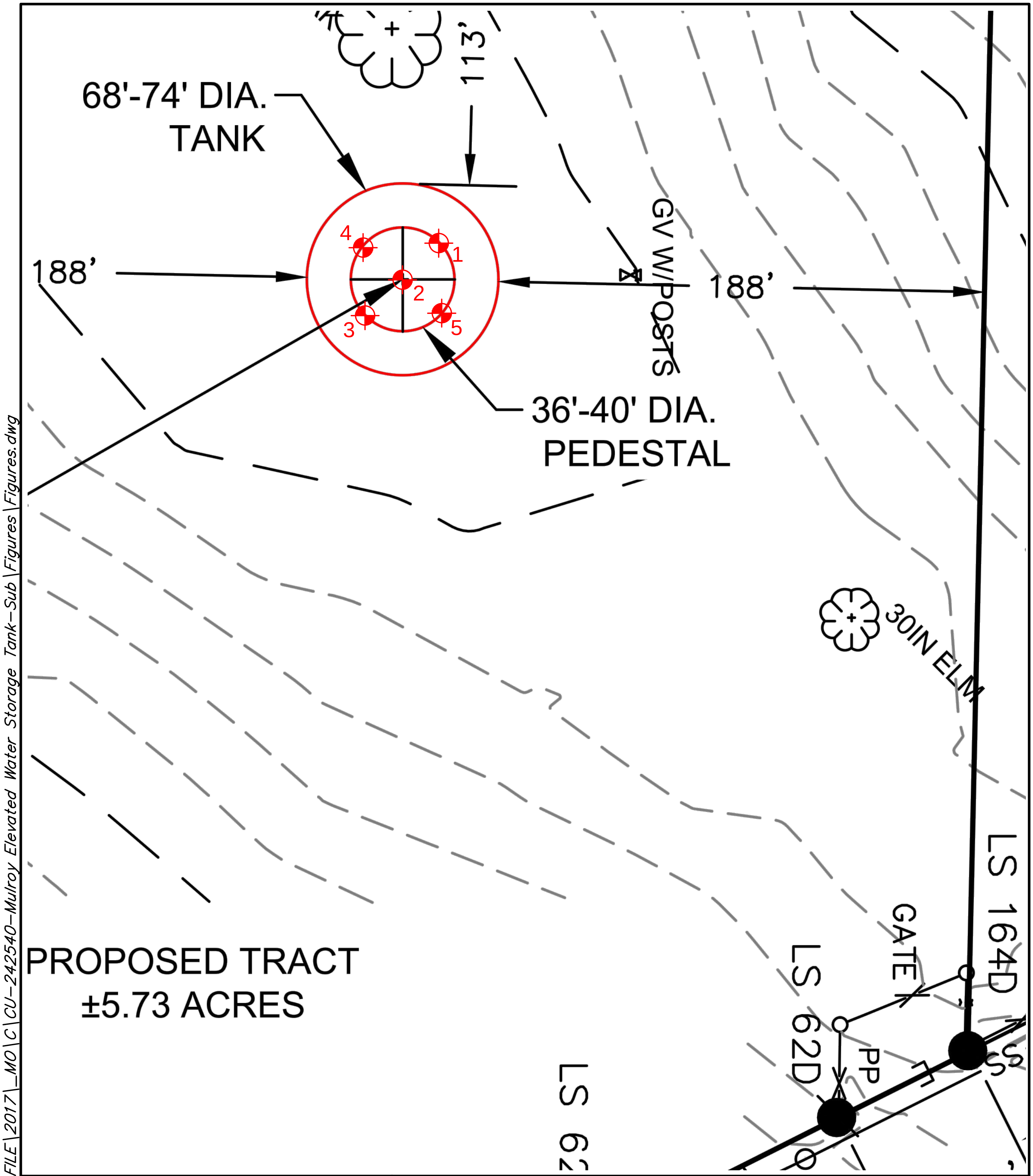
DATE: April 26, 2017

Project Number: 242540

PALMERTON & PARRISH, INC.
GEOTECHNICAL AND MATERIALS ENGINEERS/
MATERIALS TESTING LABORATORIES / ENVIRONMENTAL SERVICES

FIGURE 1

S:_MASTER PROJECT FILE\2017_MO\C\CU-242540-Mulroy Elevated Water Storage Tank-Sub\Figures\Figures.dwg



LEGEND

🔴 Boring Location

SCALE
1" = 50'

Project: Booster II - Elevated Water Storage Tank
Client: City Utilities of Springfield

Boring Location Plan 2

DATE: April 26, 2017

Project Number: 242540

PALMERTON & PARRISH, INC.
GEOTECHNICAL AND MATERIALS ENGINEERS/
MATERIALS TESTING LABORATORIES / ENVIRONMENTAL SERVICES

FIGURE 2

S:_MASTER PROJECT FILE\2017_MO\C\CU-242540-Mulroy Elevated Water Storage Tank-Sub\Figures\Figures.dwg

APPENDIX I

BORING LOGS & KEY TO SYMBOLS

4168 W. Kearney St.
Springfield, MO 65803
Telephone: (417) 864-6000
Fax: (417) 864-6004

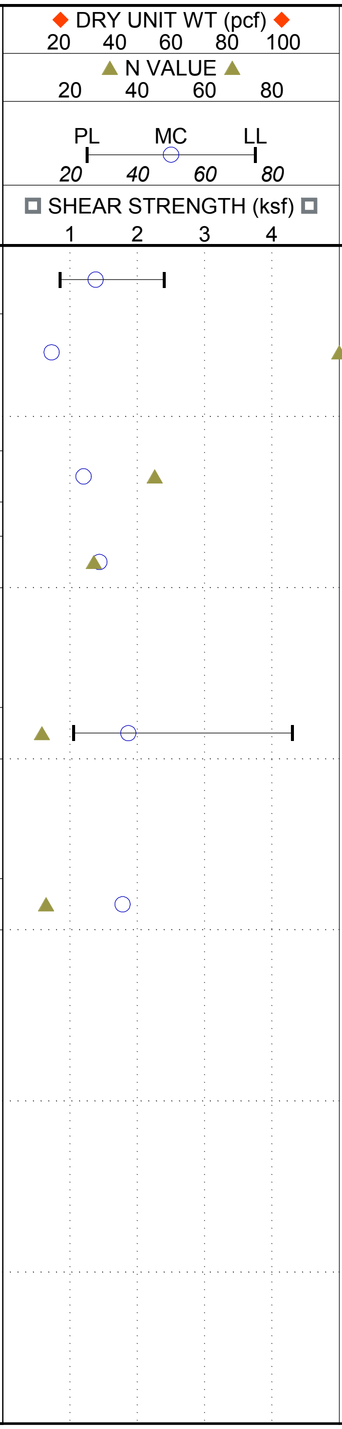
GEOTECHNICAL BORING LOG

BORING NUMBER 1

PAGE 1 OF 1

CLIENT City Utilities of Springfield
PROJECT NAME Booster II - Elevated Water Storage Tank
PROJECT NO. 242540
PROJECT LOCATION Springfield, MO
DATE STARTED 4/5/17
COMPLETED 4/6/17
SURFACE ELEVATION
BENCHMARK EL.
DRILLER EP
DRILL RIG CME-55
GROUND WATER LEVELS
HAMMER TYPE Auto
AT TIME OF DRILLING None
LOGGED BY JB
CHECKED BY BP
AT END OF DRILLING
NOTES

DEPTH (ft)	DRILLING METHOD	STRATA SYMBOL	MATERIAL DESCRIPTION Unified Soil Classification System	SAMPLE TYPE NUMBER	RECOVERY % (RQD %)	CORRECTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	ELEVATION (ft)
0	HSA - 4.25" I.D.		TOPSOIL, Grass Covered (6") 0.5 ft	ST 1	92		1	
			LEAN TO FAT CLAY, Trace Chert, Medium Stiff, Moist (CL-CH) 2.0 ft					
			CLAYEY GRAVEL, Brown/Tan, Dense, Moist (GC)	SPT 2		65/3"		
5			6.5 ft					
			CLAYEY GRAVEL, Red, Dense, Moist (GC) 7.5 ft	SPT 3		28-22-23 (45)	1.75	
			FAT CLAY, Trace Chert, Red/Tan, Very Stiff, Moist (CH)	SPT 4		14-17-10 (27)	2.5	
10			14.0 ft					
15			FAT CLAY, Scattered Chert, Red/Tan, Stiff, Moist (CH)	SPT 5		4-5-6 (11)	2	
				SPT 6		4-6-6 (12)	1.5	
20			23.0 ft					
	CORE BARREL - 2.0" I.D.		LIMESTONE, Weathered 24.0 ft					
25			-Clay Seam from 24 to 29.4' 29.4 ft					
30			LIMESTONE, Gray, Coarse Crystalline, Moderately Hard, Slightly Weathered, Medium to Thick Bedded, Fossiliferous, Scattered Chert Layers and Nodules 34.4 ft	NQ 7	100 (62)			



Bottom of borehole at 34.4 feet.

PPI
4168 W. Kearney St.
Springfield, MO 65803
Telephone: (417) 864-6000
Fax: (417) 864-6004

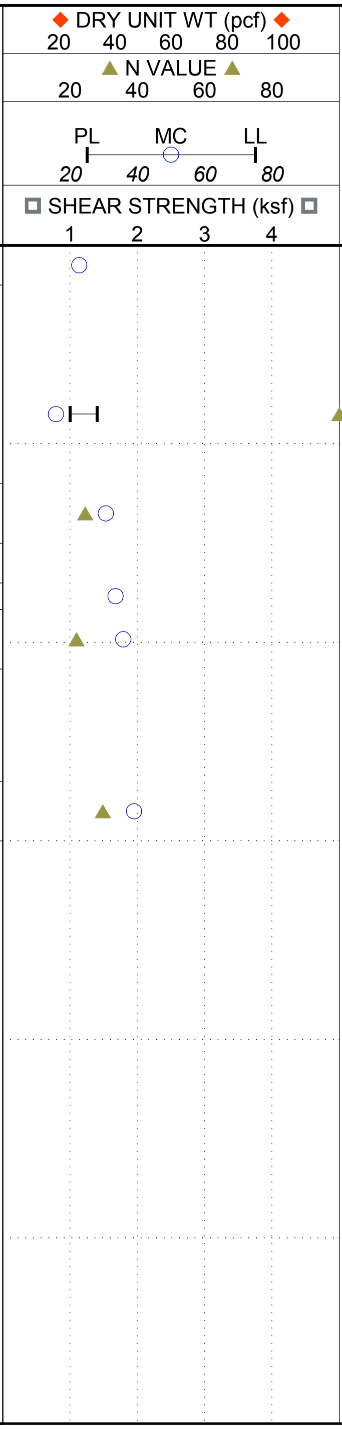
GEOTECHNICAL BORING LOG

BORING NUMBER 2

PAGE 1 OF 1

CLIENT City Utilities of Springfield
PROJECT NAME Booster II - Elevated Water Storage Tank
PROJECT NO. 242540
PROJECT LOCATION Springfield, MO
DATE STARTED 4/5/17
COMPLETED 4/5/17
SURFACE ELEVATION
BENCHMARK EL.
DRILLER EP
DRILL RIG CME-55
GROUND WATER LEVELS
AT TIME OF DRILLING None
HAMMER TYPE Auto
LOGGED BY JB
CHECKED BY BP
AT END OF DRILLING
NOTES

DEPTH (ft)	DRILLING METHOD	MATERIAL DESCRIPTION Unified Soil Classification System	SAMPLE TYPE NUMBER	RECOVERY % (RQD %)	CORRECTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	ELEVATION (ft)
0	HSA - 4.25" I.D.	TOPSOIL, Grass Covered (6") 0.5 ft	ST 1	92		1.5	
		LEAN CLAY, Trace Chert, Medium Stiff, Moist (CL) 0.8 ft					
		SANDY LEAN CLAY, w/ Gravel, Brown/Tan, Very Stiff, Moist (CL) 5.5 ft	SPT 2		40-53-52 (105)		
5		FAT CLAY, Scattered Chert, Red/Tan, Stiff, Moist (CH)	SPT 3		10-10-14 (24)	2.75	
			ST 4	100		1	
10			SPT 5		12-10-12 (22)	2	
		14.7 ft	SPT 6		5-4-26 (30)	2	
15		LIMESTONE, Weathered 15.8 ft					
	CORE BARREL - 2.0" I.D.	-Clay Seam from 15.8 to 24.7'					
20		24.7 ft					
25		LIMESTONE, Gray, Coarse Crystalline, Moderately Hard, Slightly Weathered, Medium to Thick Bedded, Fossiliferous, Scattered Chert Layers and Nodules -Vuggy from 24.7 to 25.4' 29.7 ft	NQ 7	100 (80)			



Bottom of borehole at 29.7 feet.

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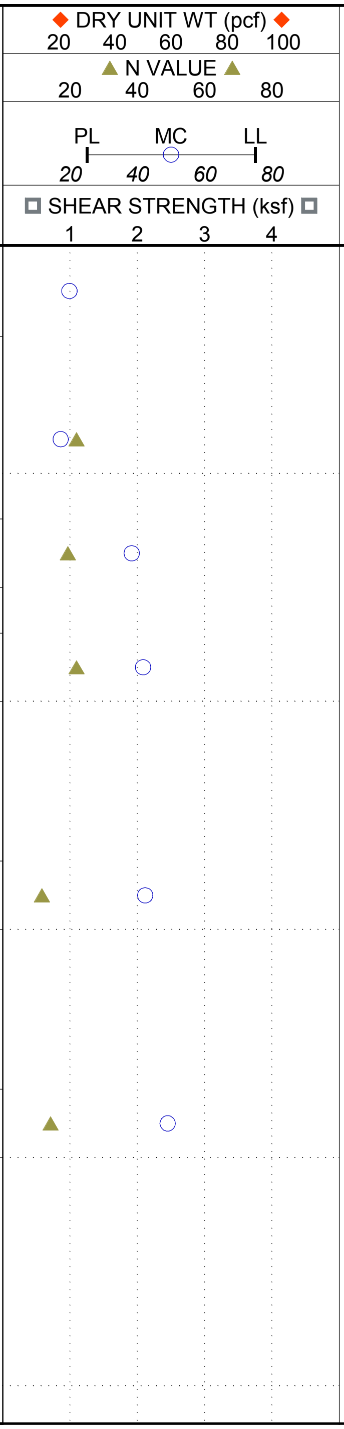
GEOTECHNICAL BORING LOG

BORING NUMBER 3

PAGE 1 OF 1

CLIENT City Utilities of Springfield
PROJECT NAME Booster II - Elevated Water Storage Tank
PROJECT NO. 242540
PROJECT LOCATION Springfield, MO
DATE STARTED 4/6/17
COMPLETED 4/6/17
SURFACE ELEVATION
BENCHMARK EL.
DRILLER EP
DRILL RIG CME-55
GROUND WATER LEVELS
HAMMER TYPE Auto
AT TIME OF DRILLING None
LOGGED BY JB
CHECKED BY BP
AT END OF DRILLING
NOTES

DEPTH (ft)	DRILLING METHOD	MATERIAL DESCRIPTION Unified Soil Classification System	SAMPLE TYPE NUMBER	RECOVERY % (RQD %)	CORRECTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	ELEVATION (ft)
0	HSA - 4.25" I.D.	TOPSOIL, Grass Covered (6") 0.5 ft					
		LEAN CLAY, Trace Chert, Stiff, Moist (CL) 2.5 ft	ST 1	50		1.5	
		LEAN CLAY, w/ Chert, Brown/Tan, Very Stiff, Moist (CL) 5.5 ft	SPT 2		14-13-9 (22)		
5		FAT CLAY, Scattered Chert, Red/Tan, Stiff, Moist (CH)	SPT 3		14-10-9 (19)	0.75	
10			SPT 4		6-9-13 (22)	2.75	
15		16.0 ft	SPT 5		6-5-6 (11)	2	
		FAT CLAY, Trace Chert, Red/Tan, Stiff, Moist (CH) 19.0 ft					
20		LIMESTONE, Gray, Coarse Crystalline, Moderately Hard, Weathered, Medium to Thick Bedded, Fossiliferous, Scattered Chert Layers and Nodules 20.8 ft	SPT 6		9-8-6 (14)	1.5	
25	CORE BARREL - 2.0" I.D.	LIMESTONE, Gray, Coarse Crystalline, Moderately Hard, Slightly Weathered, Medium to Thick Bedded, Fossiliferous, Scattered Chert Layers and Nodules 25.8 ft	NQ 7	98 (93)			



Bottom of borehole at 25.8 feet.

BORING LOG - PPI - PPI STD TEMPLATE.GDT - 4/28/17 13:17 - S:_MASTER PROJECT FILE\2017_MO\C\CU-242540-MULROY ELEVATED WATER STORAGE TANK-SUB\BORING LOGS\BORING LOGS.GPJ

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Springfield, MO 65803
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Fax: (417) 864-6004

GEOTECHNICAL BORING LOG

BORING NUMBER 4

PAGE 1 OF 1

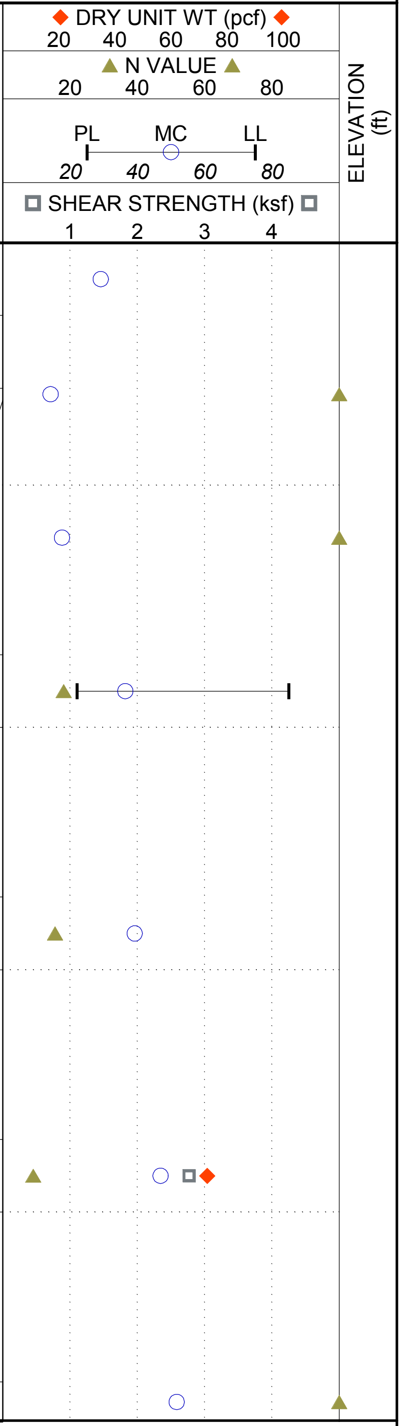
CLIENT City Utilities of Springfield
PROJECT NAME Booster II - Elevated Water Storage Tank
PROJECT NO. 242540
PROJECT LOCATION Springfield, MO
DATE STARTED 4/4/17
COMPLETED 4/4/17
SURFACE ELEVATION
BENCHMARK EL.
DRILLER EP
DRILL RIG CME-55
GROUND WATER LEVELS
HAMMER TYPE Auto
AT TIME OF DRILLING None
LOGGED BY JB
CHECKED BY BP
AT END OF DRILLING
NOTES

DEPTH (ft): 0, 5, 10, 15, 20

DRILLING METHOD: HSA - 4.25" I.D.

MATERIAL DESCRIPTION Unified Soil Classification System	SAMPLE TYPE NUMBER	RECOVERY % (RQD %)	CORRECTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)
TOPSOIL, Grass Covered (6") 0.5 ft	ST 1	89		1.25
LEAN CLAY, Trace Chert, Brown, Stiff, Moist (CL) 1.0 ft				
LEAN CLAY, Trace Chert, Tan/Grey/Brown, Medium Stiff, Moist (CL) 2.0 ft				
LEAN CLAY, w/ Chert, Brown/Tan, Stiff, Moist (CL) 4.0 ft	SPT 2		65/3"	1.5
CLAYEY GRAVEL, Red, Very Dense, Moist (GC) 6.5 ft	SPT 3		25-59-65/2"	
FAT CLAY, Scattered Chert, Red/Tan/Grey, Stiff, Moist (CH)	SPT 4		14-9-9 (18)	2.75
	SPT 5		9-9-6 (15)	1.5
17.0 ft				
FAT CLAY, Scattered Chert, Red/Tan, Stiff, Moist (CH)	SPT 6		4-4-5 (9)	2.5
24.0 ft				
LIMESTONE, Gray, Coarse Crystalline, Moderately Hard, Weathered, Medium to Thick Bedded, Fossiliferous, Scattered Chert Layers and Nodules 24.3 ft	SPT 7		9-65/4"	1.5

Bottom of borehole at 24.3 feet.



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Fax: (417) 864-6004

GEOTECHNICAL BORING LOG

BORING NUMBER 5

PAGE 1 OF 1

CLIENT City Utilities of Springfield
PROJECT NAME Booster II - Elevated Water Storage Tank
PROJECT NO. 242540
PROJECT LOCATION Springfield, MO
DATE STARTED 4/4/17
COMPLETED 4/4/17
SURFACE ELEVATION
BENCHMARK EL.
DRILLER EP
DRILL RIG CME-55
GROUND WATER LEVELS
HAMMER TYPE Auto
AT TIME OF DRILLING None
LOGGED BY JB
CHECKED BY BP
AT END OF DRILLING
NOTES

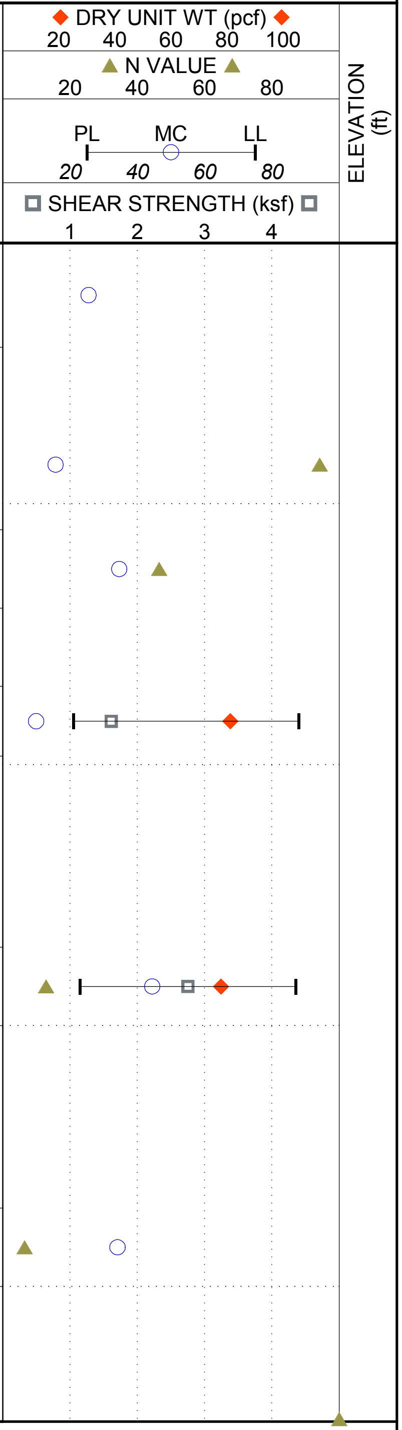
DEPTH (ft): 0, 5, 10, 15, 20

DRILLING METHOD: HSA - 4.25" I.D.

MATERIAL DESCRIPTION Unified Soil Classification System	
TOPSOIL, Grass Covered (6")	0.5 ft
LEAN CLAY, Trace Chert, Brown, Medium Stiff, Moist (CL)	1.0 ft
LEAN CLAY, Trace Chert, Tan/Grey/Brown, Medium Stiff, Moist (CL)	2.0 ft
CLAYEY GRAVEL, w/ Sand, Brown/Tan, Very Dense, Moist (GC)	6.0 ft
FAT CLAY, Trace Chert, Red, Stiff, Moist (CH)	10.0 ft
FAT CLAY, Scattered Chert, Red, Stiff, Moist (CH)	17.0 ft
FAT CLAY, Trace Chert, Red/Tan, Medium Stiff, Moist (CH)	22.3 ft
LIMESTONE, Gray, Coarse Crystalline, Moderately Hard, Weathered, Medium to Thick Bedded, Fossiliferous, Scattered Chert Layers and Nodules	22.6 ft

Bottom of borehole at 22.6 feet.

SAMPLE TYPE NUMBER	RECOVERY % (RQD %)	CORRECTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)
ST 1	75		1
SPT 2		36-45-49 (94)	
SPT 3		23-28-18 (46)	2.25
ST 4	100		4.25
SPT 5		4-6-6 (12)	2.75
SPT 6		4-3-4 (7)	1.25
SPT 7		65/1"	



4168 W. Kearney St.
Springfield, Missouri 65803
Telephone: (417) 864-6000
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KEY TO SYMBOLS

CLIENT City Utilities of Springfield

PROJECT NAME Booster II - Elevated Water Storage Tank

PROJECT NO. 242540

PROJECT LOCATION Springfield, MO

LITHOLOGIC SYMBOLS
(Unified Soil Classification System)

▨ CH: USCS High Plasticity Clay

▨ CL: USCS Low Plasticity Clay

▨ CL-CH: USCS Low to High Plasticity Clay

▩ GC: USCS Clayey Gravel

▦ LIMESTONE

⬚ TOPSOIL: Topsoil

SAMPLER SYMBOLS

◫ NQ

⧗ Standard Penetration Test

■ Shelby Tube

WELL CONSTRUCTION SYMBOLS

ABBREVIATIONS

LL - LIQUID LIMIT (%)
PI - PLASTIC INDEX (%)
W - MOISTURE CONTENT (%)
DD - DRY DENSITY (PCF)
NP - NON PLASTIC
-200 - PERCENT PASSING NO. 200 SIEVE
PP - POCKET PENETROMETER (TSF)

TV - TORVANE
PID - PHOTOIONIZATION DETECTOR
UC - UNCONFINED COMPRESSION
ppm - PARTS PER MILLION

▽ Water Level at Time Drilling, or as Shown

▼ Water Level at End of Drilling, or as Shown

▽ Water Level After 24 Hours, or as Shown

KEY TO SYMBOLS - PPI STD TEMPLATE.GDT - 4/25/17 14:44 - S:_MASTER PROJECT FILE\2017_MO\C\CU-242540-MULROY ELEVATED WATER STORAGE TANK-SUB\BORING LOGS\BORING LOGS.GPJ

APPENDIX II

GENERAL NOTES

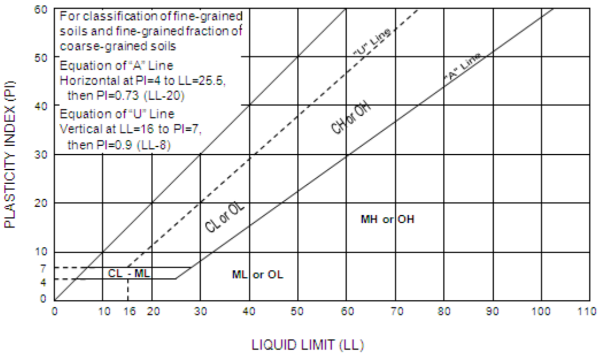
PALMERTON & PARRISH, INC.

GENERAL NOTES

SOIL PROPERTIES & DESCRIPTIONS

COHESIVE SOILS

Consistency	Unconfined Compressive Strength (Qu)	Pocket Penetrometer Strength	N-Value
	(psf)	(tsf)	(blows/ft)
Very Soft	<500	<0.25	0-1
Soft	500-1000	0.25-0.50	2-4
Medium Stiff	1001-2000	0.50-1.00	5-8
Stiff	2001-4000	1.00-2.00	9-15
Very Stiff	4001-8000	2.00-4.00	16-30
Hard	>8000	>4.00	31-60
Very Hard			>60



Group Symbol	Group Name
CL –	Lean Clay
ML –	Silt
OL –	Organic Clay or Silt
CH –	Fat Clay
MH –	Elastic Silt
OH –	Organic Clay or Silt
PT –	Peat
CL-CH –	Lean to Fat Clay

Plasticity		Moisture	
Description	Liquid Limit (LL)	Descriptive Term	Guide
Lean	<45%	Dry	No indication of water
Lean to Fat	45-49%	Moist	Indication of water
Fat	≥50%	Wet	Visible water

Fine Grained Soil Subclassification	Percent (by weight) of Total Sample
Terms: SILT, LEAN CLAY, FAT CLAY, ELASTIC SILT	PRIMARY CONSTITUENT
Sandy,gravelly, abundant cobbles, abundant boulders	>30-50]
with sand, with gravel, with cobbles, with boulders	>15-30] – secondary coarse grained constituents
scattered sand, scattered gravel, scattered cobbles, scattered boulders	5-15]
a trace sand, a trace gravel, a few cobbles, a few boulders	<5]
The relationship of clay and silt constituents is based on plasticity and normally determined by performing index tests. Refined classifications are based on Atterberg Limits tests and the Plasticity Chart.	

NON-COHESIVE (GRANULAR) SOILS

RELATIVE DENSITY	N-VALUE
Very Loose	0-4
Loose	5-10
Medium Dense	11-24
Dense	25-50
Very Dense	≥51

MOISTURE CONDITION	
Descriptive Term	Guide
Dry	No indication of water
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table.

**GRAIN SIZE IDENTIFICATION		
Name	Size Limits	Familiar Example
Boulder	12 in. or more	Larger than basketball
Cobbles	3 in. to 12 in.	Grapefruit
Coarse Gravel	¾-in. to 3 in.	Orange or lemon
Fine Gravel	No. 4 sieve to ¾-in.	Grape or pea
Coarse Sand	No. 10 sieve to No. 4 sieve	Rock salt
Medium Sand	No. 40 sieve to No. 10 sieve	Sugar, table salt
Fine Sand*	No. 200 sieve to No. 40 sieve	Powdered sugar
Fines	Less than No. 200 sieve	
*Particles finer than fine sand cannot be discerned with the naked eye at a distance of 8 in.		

Coarse Grained Soil Subclassification	Percent (by weight) of Total Sample
Terms: GRAVEL, SAND, COBBLES, BOULDERS	PRIMARY CONSTITUENT
Sandy,gravelly, abundant cobbles, abundant boulders	>30-50]
with gravel, with sand, with cobbles, with boulders	>15-30] – secondary coarse grained constituents
scattered gravel, scattered sand, scattered cobbles, scattered boulders	5-15]
a trace gravel, a trace sand, a few cobbles, a few boulders	<5]
Silty (MH & ML)*, clayey (CL & CH)*	<15]
(with silt, with clay)*	5-15] – secondary fine grained constituents
(trace silt, trace clay)*	<5]
*Index tests and/or plasticity tests are performed to determine whether the term "silt" or "clay" is used.	

*Modified after Ref. ASTM D2487-93 & D2488-93
**Modified after Ref. Oregon DOT 1987 & FHWA 1997
***Modified after Ref. AASHTO 1988, DM 7.1 1982, and Oregon DOT 1987

GENERAL NOTES

BEDROCK PROPERTIES & DESCRIPTIONS

ROCK QUALITY DESIGNATION (RQD)

Description of Rock Quality	*RQD (%)
Very Poor	< 25
Poor	25-50
Fair	50-75
Good	75-90
Excellent	90-100

*RQD is defined as the total length of sound core pieces 4 in. or greater in length, expressed as a percentage of the total length cored. RQD provides an indication of the integrity of the rock mass and relative extent of seams and bedding planes.

SCALE OF RELATIVE ROCK HARDNESS

Term	Field Identification	Approx. Unconfined Compressive Strength (tsf)
Extremely Soft	Can be indented by thumbnail	2.6-10
Very Soft	Can be peeled by pocket knife	10-50
Soft	Can be peeled with difficulty by pocket knife	50-260
Medium Hard	Can be grooved 2 mm deep by firm pressure of knife	260-520
Moderately Hard	Requires one hammer blow to fracture	520-1040
Hard	Can be scratched with knife or pick only with difficulty	1040-2610
Very Hard	Cannot be scratched by knife or sharp pick	>2610

DEGREE OF WEATHERING

Slightly Weathered	Rock generally fresh, joints stained and discoloration extends into rock up to 25mm (1 in), open joints may contain clay, core rings under hammer impact.
Weathered	Rock mass is decomposed 50% or less, significant portions of rock show discoloration and weathering effects, cores cannot be broken by hand or scraped by knife.
Highly Weathered	Rock mass is more than 50% decomposed, complete discoloration of rock fabric, core may be extremely broken and gives clunk sound when struck by hammer, may be shaved with a knife.

GRAIN SIZE (TYPICALLY FOR SEDIMENTARY ROCKS)

Description	Diameter (mm)	Field Identification
Very Coarse Grained	>4.76	
Coarse Grained	2.0-4.76	Individual grains can easily be distinguished by eye.
Medium Grained	0.42-2.0	Individual grains can be distinguished by eye.
Fine Grained	0.074-0.42	Individual grains can be distinguished by eye with difficulty.
Very Fine Grained	<0.074	Individual grains cannot be distinguished by unaided eye.

VOIDS

Pit	Voids barely seen with naked eye to 6mm (¼-in)
Vug	Voids 6 to 50mm (¼ to 2 in) in diameter
Cavity	50 to 6000mm (2 to 24 in) in diameter
Cave	>600mm

BEDDING THICKNESS

Very Thick Bedded	> 3’ thick
Thick Bedded	1’ to 3’ thick
Medium Bedded	4” to 1’ thick
Thin Bedded	1¼” to 4” thick
Very Thin Bedded	½” to 1¼” thick
Thickly Laminated	⅛” to ½” thick
Thinly Laminated	⅛” or less (paper thin)

DRILLING NOTES

Drilling and Sampling Symbols

NQ – Rock Core (2-in. diameter)
HQ – Rock Core (3 in. diameter)
HSA – Hollow Stem Auger
CFA – Continuous Flight (Solid Stem) Auger
SS – Split Spoon Sampler
ST – Shelby Tube
WB – Wash Bore or Mud Rotary
TP – Test-Pit
HA – Hand Auger

Soil Sample Types

Shelby Tube Samples: Relatively undisturbed soil samples were obtained from the borings using thin wall (Shelby) tube samplers pushed hydraulically into the soil in advance of drilling. This sampling, which is considered to be undisturbed, was performed in accordance with the requirements of ASTM D 1587. This type of sample is considered best for the testing of "in-situ" soil properties such as natural density and strength characteristics. The use of this sampling method is basically restricted to soil containing little to no chert fragments and to softer shale deposits.

Split Spoon Samples: The Standard Penetration Test is conducted in conjunction with the split-barrel sampling procedure. The “N” value corresponds to the number of blows required to drive the last 1 foot of an 18-in. long, 2-in. O.D. split-barrel sampler with a 140 lb. hammer falling a distance of 30 in. The Standard Penetration Test is carried out according to ASTM D-1586.

Water Level Measurements

Water levels indicated on the boring logs are levels measured in the borings at the times indicated. In permeable materials, the indicated levels may reflect the location of groundwater. In low permeability soils, shallow groundwater may indicate a perched condition. Caution is merited when interpreting short-term water level readings from open bore holes. Accurate water levels are best determined from piezometers.

Automatic Hammer

Palmerton and Parrish’s CME’s are equipped with automatic hammers. The conventional method used to obtain disturbed soil samples used a safety hammer operated by company personnel with a cat head and rope. However, use of an automatic hammer allows a greater mechanical efficiency to be achieved in the field while performing a Standard Penetration resistance test based upon automatic hammer efficiencies calibrated using dynamic testing techniques.

*Modified after Ref. ASTM D2487-93 & D2488-93
**Modified after Ref. Oregon DOT 1987 & FHWA 1997
***Modified after Ref. AASHTO 1988, DM 7.1 1982, and Oregon DOT 1987

APPENDIX III

GRAIN SIZE ANALYSIS

4168 W. Kearney St.
Springfield, Missouri 65803
Telephone: (417) 864-6000
Fax: (417) 864-6004

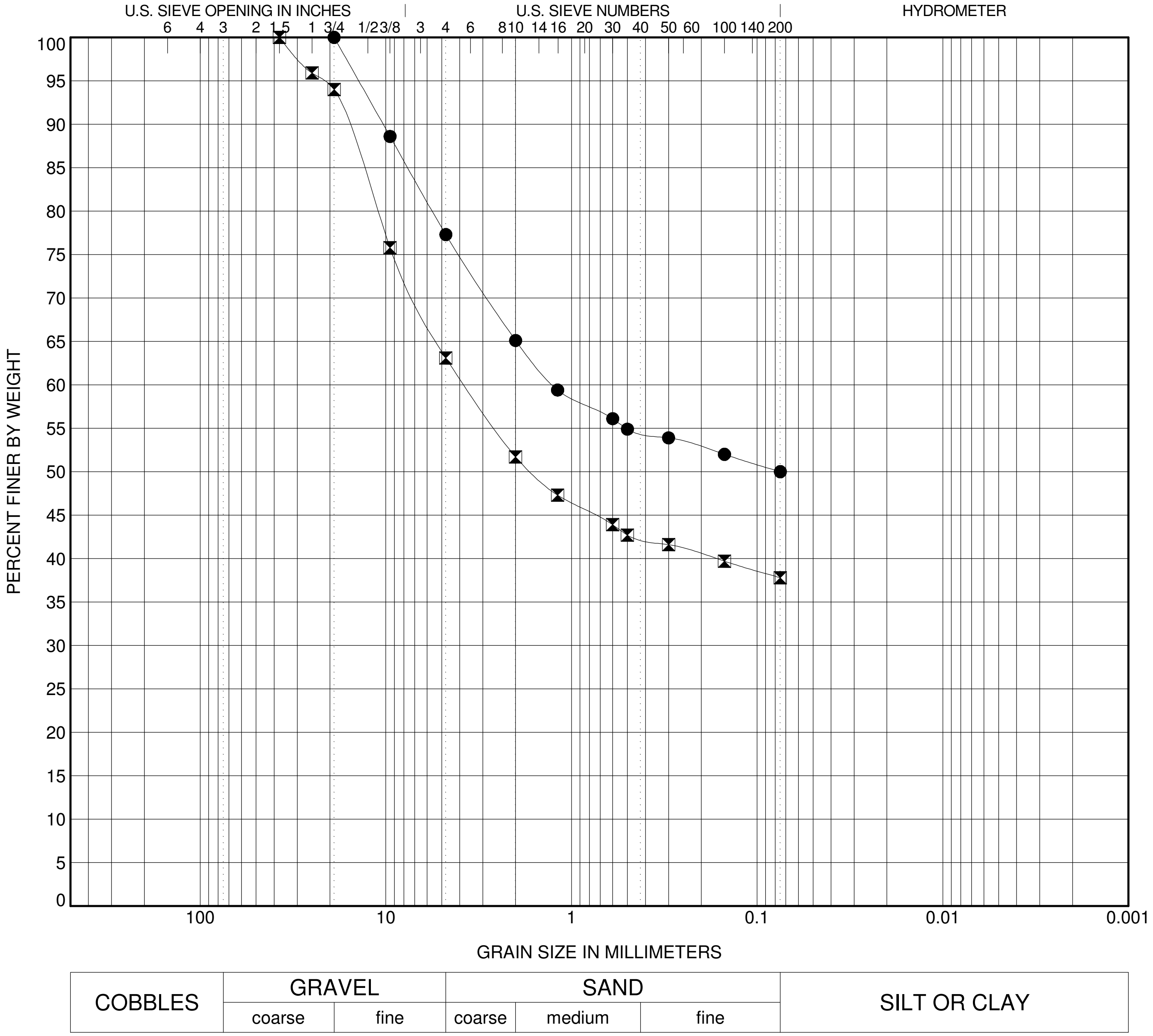
GRAIN SIZE DISTRIBUTION

CLIENT City Utilities of Springfield

PROJECT NAME Booster II - Elevated Water Storage Tank

PROJECT NO. 242540

PROJECT LOCATION Springfield, MO



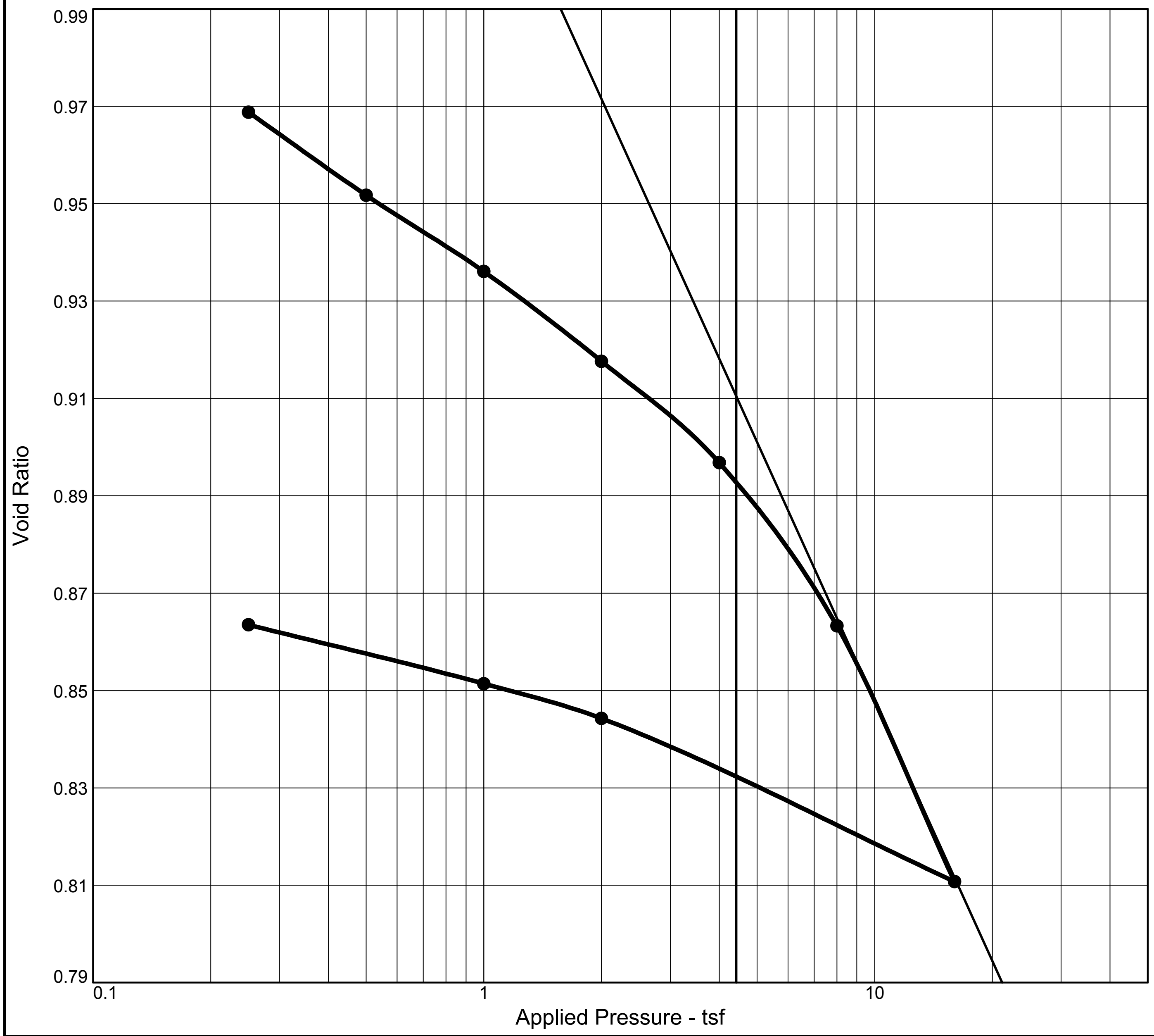
	BOREHOLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**2**	**3.5**	**SANDY LEAN CLAY with GRAVEL(CL)**	**28**	**20**	**8**		
⧗	**5**	**3.5**	**CLAYEY GRAVEL with SAND(GC)**					

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**2**	**3.5**	**19**	**1.255**			**22.7**	**27.3**	**50.0**	
⧗	**5**	**3.5**	**37.5**	**3.754**			**36.9**	**25.3**	**37.8**	

APPENDIX IV

CONSOLIDATION TEST RESULTS

CONSOLIDATION TEST REPORT



MATERIAL DESCRIPTION	USCS	AASHTO

LL	PI	Sp. Gr.	Overburden (tsf)	Dry Dens. (pcf)		Moisture		Saturation		Void Ratio		P_c (tsf)	C_c
				Init.	Final	Init.	Final	Init.	Final	Init.	Final		
88	21	2.65		83.4		35.7 %	32.8	96.3 %	100.0	0.983	0.863	6.0	0.18

Preparation Process:

Condition of Test:

D2435 Method	C_r	Swell Press. (tsf)	%
	0.03		

Project No. 242540 **Client:** City Utilities of Springfield

Project: Mulroy Elevated Water Staorage Tank

Location: B-5 **Depth:** 8.5"-9.83"

Palmerton & Parrish, Inc.

Remarks:

Checked By:

Title:

Figure

APPENDIX V

ROCK CORE PHOTOGRAPHS

City Utilities of Springfield
Mulroy Elevated Storage Tank
Springfield, MO

Boring # 1

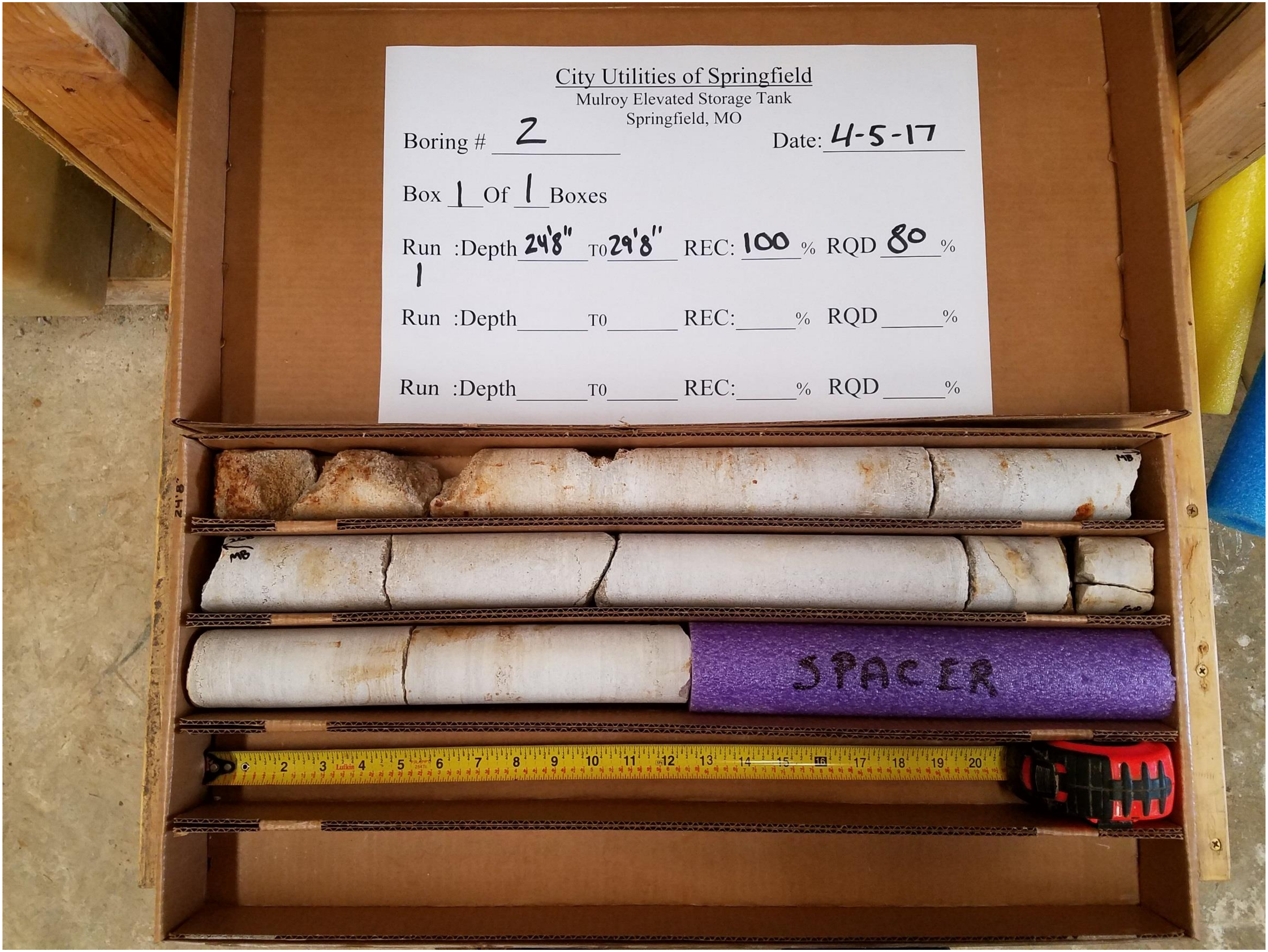
Date: 4·6·17

Box 1 Of 1 Boxes

Run 1 :Depth 29'5" T0 34'5" REC: 100 % RQD 62 %

Run :Depth ______ T0 ______ REC: _____% RQD _____%

Run :Depth ______ T0 ______ REC: _____% RQD _____%



City Utilities of Springfield

Mulroy Elevated Storage Tank
Springfield, MO

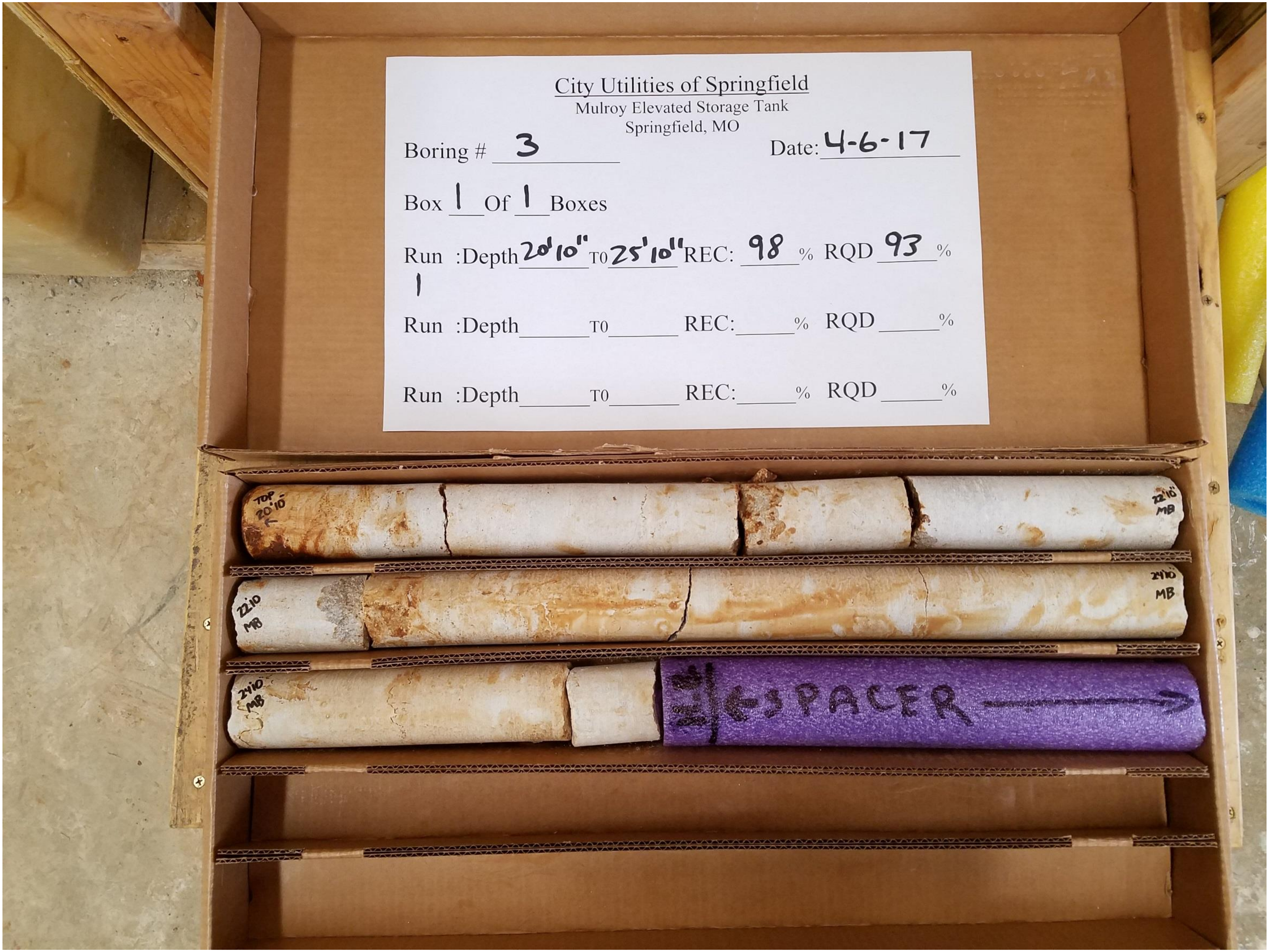
Boring # 2 Date: 4-5-17

Box 1 Of 1 Boxes

Run 1 :Depth 24'8" T0 29'8" REC: 100 % RQD 80 %

Run :Depth ______ T0 ______ REC: _____ % RQD _____ %

Run :Depth ______ T0 ______ REC: _____ % RQD _____ %



City Utilities of Springfield

Mulroy Elevated Storage Tank
Springfield, MO

Boring # 3 Date: 4-6-17

Box 1 Of 1 Boxes

Run 1 :Depth 20'10" T0 25'10" REC: 98 % RQD 93 %

Run :Depth ______ T0 ______ REC: ______ % RQD ______ %

Run :Depth ______ T0 ______ REC: ______ % RQD ______ %

APPENDIX VII

ITEMS TO INCLUDE IN DRILLED PIER SPECIFICATIONS

Items to Include in the Drilled Pier Specifications

The following items include both design considerations and items that may be included in the project specifications for the drilling contractor's use.

1. The piers may be designed using a contact pressure of 20 ksf on non-proof-tested piers or 60 ksf on acceptable bedrock if proof-tested. No additional allowance should be given to skin friction in the soil overburden.
2. The piers should extend into fresh sound bedrock a minimum of 1 ft. The final bottom should be a flat level plane without steps.
3. A 2-inch diameter probe hole should be drilled in the bottom of each pier to a depth of at least 1.5 times the pier diameter, but no less than 5 ft. A scratch test should reveal that the seams and voids encountered meet the following criteria:
    a. No open seams or voids in the top 3 ft.
    b. No individual seam or void greater than 1/4 in. in the next 3 ft.
    c. Total accumulation of open seams or voids shall not exceed 1/2 inch.
4. A minimum of two (2) exploratory probe holes should be required for pier shafts with a 4 ft. or greater diameter.
5. Soft wet soil is common above the top of rock in the site area. These conditions should be expected. The drilling contractor should provide casing capable of being screwed or drilled into the bedrock to seal out the wet, soft soils.
6. The completion of the foundation system may require the penetration of several feet of bedrock in various piers. The drilling contractor should expect to perform both rock excavation and water removal by pumping.
7. An effort should be made to restrict the number of shaft sizes. The minimum shaft diameter in which a man can drill the probe hole and check the rock quality is about 30 in.
8. The bottom of the pier should be cleaned of all loose soil and rock fragments at the time of concrete placement. No more than 2 or 3 inches of clean water should be present in the bottom when concrete is introduced into the shaft. Casing extraction should proceed slowly during the concrete placement so that at least a 3-ft. head of concrete is always present above the bottom of casing during extraction. In some cases, more than 3 ft. of head may be required.
9. Method of concrete placement and vibration should be selected by the structural engineer consistent with the placement requirements on the other portions of the structure. The required strength and mix design characteristics should also be specified by the design team.
10. Clays overlying the bedrock can be jointed. While this jointing pattern does not materially affect the soils supporting strength, it does affect its "stand-up" time in pier excavations. Lateral stability of the soils surrounding the pier shaft may be low.
11. To assure plumbness of pier shafts, plumbness should be checked using a string and plumb bob. Shafts should be out of plumb no more than 2 percent of shaft length.

APPENDIX VIII

IMPORTANT INFORMATION REGARDING YOUR GEOTECHNICAL REPORT

Important Information about Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a.modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

ASFE
THE BEST PEOPLE ON EARTH

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org