FINAL
GEOTECHNICAL INVESTIGATION
WILLOWBROOK WASTEWATER TREATMENT PLANT IMPROVEMENTS
WBS NO. R-000265-0104-3
HOUSTON, TEXAS

SUBMITTED TO
WESTON SOLUTIONS, INC.
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HOUSTON, TEXAS 77056

BY
HVJ ASSOCIATES, INC.
HOUSTON, TEXAS

JUNE 25, 2014

REPORT NO. HG1312760
KEY MAP NO. 370K

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June 25, 2014

Mr. Champak Sadhu, PE, FASCE
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5599 San Felipe, Suite 700
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Re: Geotechnical Investigation
Willowbrook Wastewater Treatment Plant Improvements
WBS No. R-000265-0104-3
Houston, Texas
Owner: City of Houston
HVJ Report No. HG1312760

Dear Mr. Sadhu:

Submitted herein is the final report of our geotechnical investigation for the above referenced project. The study was conducted in general accordance with our proposal number HG1312760 dated May 21, 2013 (Revised on June 20, 2013) and is subject to the limitations presented in this report.

We appreciate the opportunity of working with you on this project. Please read the entire report and notify us if there are questions concerning this report or if we may be of further assistance.

Sincerely,

HVJ ASSOCIATES, INC.
Texas Firm Registration No. F-000646

STATE OF TEXAS
NISHANT DAYAL
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LICENSED
PROFESSIONAL ENGINEER

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The seal appearing on this document was authorized by Nishant Dayal, PE 109464 on June 25, 2014. Alteration of a sealed document without proper notification to the responsible engineer is an offense under the Texas Engineering Practice Act.

The following lists the pages which complete this report:

- Main Text – 24 pages
- Plates – 7 pages
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1 EXECUTIVE SUMMARY

HVJ Associates, Inc. was retained by Weston Solutions, Inc. to provide geotechnical services for the proposed improvements at Willowbrook Wastewater Treatment Plant located at 7101 W. Greens Road in Houston, Texas. The project involves the rehabilitation of an existing secondary clarifier, construction of new secondary clarifier, pier supported odor control pad, RAS pump station, scum pumps supported by on-grade pad, pavement and installation of gravity and force main pipes. The clarifier section drawings presented in Appendix F shows that the bottom of clarifier perimeter is at 8.5 feet and the center is at 20.5 feet below existing grade. We understand that the odor control platform will be no more than 4 feet above the existing ground surface.

The purpose of this geotechnical investigation is to conduct a subsurface exploration and perform laboratory testing to provide foundation design recommendations for the secondary clarifier, odor control platform, RAS pump station, scum pumps and also for the installation of gravity and force main pipes. This investigation was performed in general accordance with the City of Houston Department of Public Works and Engineering Infrastructure Design Manual dated July 2012. A site vicinity map showing the approximate project location is presented on Plate 1 of the report.

Subsurface conditions were investigated by drilling one (1) soil boring to a depth of 15 feet at the location of proposed odor control pad and drilling three (3) soil borings to a depth of 50 feet at the location of the proposed 100 feet diameter secondary clarifier. Based on the subsurface conditions revealed by the soil boring, the findings and recommendations of this report are summarized below:

1. Secondary clarifier: Based on the cross section of the clarifier provided to us (See Appendix F), we understand that the clarifier will be founded at a depth of 8.5 to 20.5 feet below existing grade. The location corresponds to borings B-2, B-3, and B-4. Based on the information revealed by the test borings, the soils at the foundation level consist of stiff to very stiff cohesive soils. An allowable bearing capacity of 4,000 psf can be used to design the mat foundation founded at a depth of 8.5 to 20.5 feet below existing grade. A generalized stratigraphy is provided below:

Boring No.	Depth, Feet		Material
	From	To	
B-2,3,4	0	8	Medium Dense Silt With Sand (ML) and/or Very Loose Poorly Graded Sand (SP)
B-2,3,4	8	22	Stiff to Very Stiff Clay (CH/CL)
B-3	22	26	Medium Dense Sandy Silt (ML)
B-2,4	22	28	Medium Dense Sandy Silt (ML) and Medium Dense Silt With Sand (ML)
B-2,3,4	28	40	Very Stiff Clay (CL/CH/CL-ML)
B-2	40	Below	Dense to Very Dense Silty Sand (SM)
B-3,4	40	Below	Stiff to Very Stiff Clay (CH/CL) and Dense to Very Dense Clayey Sand (SC)

2. Odor Control Structure: To support a total load of 30 kips on pier coming from the platform supporting Odor Control Structure, we recommend placing the bottom of underreamed pier

with 3 feet base diameter at 8 feet depth below the existing grade. The bell-to-shaft diameter ratio should be 2:1 forming 30° angle with the vertical.

3. For the pavement considerations, sandy silt was observed approximately at the top 6 feet at the four borings. The pavement design recommendations are provided below:

Types of loads	**Rigid Pavement Section**	**Flexible Pavement Section**
Heavy Loads (Heavy Truck Traffic)	7" Concrete 6" Cement Stabilized Subgrade	3" Asphaltic Concrete 9" Limestone Base 6" Cement Stabilized Subgrade
Medium Loads (Medium Truck and Heavy Use Driveways)	6" Concrete 6" Cement Stabilized Subgrade	2.5" Asphaltic Concrete 8" Limestone Base 6" Cement Stabilized Subgrade
Light Loads (Automobile Parking Areas)	5" Concrete 6" Cement Stabilized Subgrade	2" Asphaltic Concrete 6" Limestone Base 6" Cement Stabilized Subgrade

4. For the utilities consideration, stiff to very stiff clay was observed at the depth of approximately 15 to 18 feet, and silt at the top 5 feet at borings B-2, B-3, and B-4. Utility design criteria and construction considerations are provided in Sections 9 and 10 of this report.

5. Groundwater was not encountered at boring B-1 during the drilling operation. Groundwater could not be measured during drilling at B-2 and B-4 since wet rotary drilling started at 18 feet and 24 feet depth, respectively. Groundwater was observed at a depth of 15 feet during the drilling operations at boring B-3. It should be noted that groundwater levels determined during drilling may not accurately reflect the true groundwater conditions, and therefore should only be considered as approximate. Excavation for the clarifier and underground utilities may encounter groundwater. A well point system may be utilized to lower the groundwater table to facilitate construction processes.

6. A literature review of surface faults was made from published reports. The primary objective of this review was to evaluate available information from published reports and open file reports. Based on our review, we did not find documented faults within 2 miles radius from the site location.

7. Corrosion Tests: Based on the sulfate lab results of the current condition of the soil (29.5 mg/kg), the water soluble sulfate in the soil is less than the criteria for corrosive environment for concrete as indicated in Section 11.2 of this report.

Please, note that this executive summary does not fully relate our findings and recommendations. Those findings and opinions are only presented through our full report.

2 INTRODUCTION

2.1 Project Description

HVJ Associates, Inc. was retained by Weston Solutions, Inc. to provide geotechnical services for the proposed improvements at Willowbrook Wastewater Treatment Plant located at 7101 W. Greens Road in Houston, Texas. The project involves the rehabilitation of an existing secondary clarifier, construction of new secondary clarifier, pier supported odor control pad, RAS pump station, scum pumps supported by on-grade pad, pavement and installation of gravity and force main pipes. The clarifier section drawings presented in Appendix F shows that the bottom of clarifier perimeter is at 8.5 feet and the center is at 20.5 feet below existing grade. We understand that the odor control platform will be no more than 4 feet above the existing ground surface.

The purpose of this geotechnical investigation is to conduct a subsurface exploration and perform laboratory testing to provide foundation design recommendations for the secondary clarifier, odor control platform, RAS pump station, scum pumps and also for the installation of gravity and force main pipes. This investigation was performed in general accordance with the City of Houston Department of Public Works and Engineering Infrastructure Design Manual dated July 2012.

2.2 Geotechnical Investigation Program

The primary objectives of this study were to gather information on subsurface conditions at the site and to provide geotechnical recommendations for design and construction of proposed structures. The objectives were accomplished by:

- Drilling one (1) soil boring to a depth of 15 feet at the location of proposed equipment pad and drilling three (3) soil borings to a depth of 50 feet at the location of the proposed 100 feet diameter secondary clarifier and to obtain samples for laboratory testing;
- Performing laboratory tests to determine physical and engineering characteristics of the soils; and
- Performing engineering analyses to develop design guidelines and recommendations for the proposed structures.

Subsequent sections of this report contain descriptions of the field exploration, laboratory-testing program, general subsurface conditions, design recommendations, and construction considerations.

3 FIELD INVESTIGATION

3.1 Geotechnical Borings

The field exploration program undertaken at the project site was performed on July 18 and 19, 2013. Subsurface conditions were investigated by drilling one (1) soil boring to a depth of 15 feet at the location of proposed equipment pad and drilling three (3) soil borings to a depth of 50 feet at the location of the proposed secondary clarifier. Borings B-2 and B-4 were drilled additional 5 feet due to the presence of cohesionless soil at the termination depth.

3.2 Survey Data

The coordinates and elevations of borings are provided to us by Weston Solutions, Inc. and are summarized in Table 3-1.

Boring	Northing, Feet	Easting, Feet	Ground Surface Elevation, Feet	Boring Depth, Feet
B-1	13,912,144.59	3,066,428.46	115.88	15
B-2	13,911,389.63	3,066,757.42	114.71	55
B-3	13,911,349.35	3,066,725.55	114.97	50
B-4	13,911,432.71	3,066,789.37	114.71	55

3.3 Sampling Methods

Soil samples were obtained continuously to the termination depth of the boring. Cohesive soil samples were obtained with a three-inch thin-walled (Shelby) tube sampler in general accordance with ASTM D-1587 standard. Each sample was removed from the sampler in the field, carefully examined and then classified. The shear strength of the cohesive soils was estimated by a hand penetrometer in the field. Suitable portions of each sample were sealed and packaged for transportation to our laboratory.

Detailed descriptions of the soils encountered in the borings are given on the boring log presented in Appendix A. A key to terms and symbols used on boring log is also presented in Appendix A.

4 LABORATORY TESTING

Selected soil samples were tested in the laboratory to determine applicable physical and engineering properties. All tests except pocket penetrometer were performed according to the relevant ASTM Standards. These tests consisted of moisture content measurements, Percent Passing No. 200 Sieve, Atterberg Limits, unconsolidated undrained compression, unit weight, swell and consolidation tests.

The Atterberg limits and percent passing number 200 sieve tests were utilized to verify field classification by the ASTM version of the Unified Soils Classification System, and the unconsolidated undrained tests were performed to obtain the undrained shear strength of the soil, consolidation and swell tests were performed to analyze swelling and settlement potential of the soil. The type and number of tests performed for this investigation are summarized below:

Type of Test	Number of Tests
Moisture Content (ASTM D2216)	77
Atterberg Limits (ASTM D4318)	25
Percent Passing No. 200 Sieve (ASTM D1140 & ASTM 2487)	39
Unconsolidated Undrained Compression (UU) (ASTM D 2850)	16
Unit Wet Weight (ASTM D 7263)	16
Swell Test (ASTM D 4546)	2
Consolidation Test (ASTM D 2435)	1
Sulfate (EPA 300.0)	1
Chloride (EPA 300.0)	1
Sulfide (SW-846 METHOD 9034)	1

The laboratory test results are presented on the boring logs in Appendix A. A summary of laboratory test results are provided in Appendix B. Swell test and consolidation test results are provided in Appendices C and D, respectively.

4.1 Corrosion Test Results

The results of sulfate and chloride analysis based on test method E300, Sulfide analysis based on test method SW-8469034, and pH based on test method SW-846 9045D performed on in-situ moist sample are presented below.

Boring No.	Depth (feet)	Total Sulfate (mg/Kg)	Chloride (mg/Kg)	Sulfide (mg/Kg)	pH
B-2	4'-6'	29.5	18.5	BRL	8.33
B-3	6'-8'	12.5	4.66	BRL	8.01
B-4	10-12'	4.99	6.54	BRL	7.31

An evaluation of the corrosive effect of the soil on the ductile iron pipe is presented in Section 11. A summary of the Sulfate, Sulfide, Chloride, and pH test results are presented in Appendix E.

5 SITE CHARACTERIZATION

5.1 General Geology

There are two major surface geological formations that exist in the Houston area: the Beaumont formation and the Lissie formation. The Beaumont formation is a relatively younger formation generally found to the southeast of the Lissie formation. The Beaumont formation dips southeastward and extends beneath beach sand and waters of the Gulf of Mexico as far as the continental shelf. The project site is located in an area where the upper Lissie formation is typically encountered.

The upper Lissie formation is sometimes denoted as the Montgomery formation. The upper Lissie formation is heterogeneous, containing interbedded layers of clay, sand and silt. It was deposited in mid-Pleistocene times in shallow coastal river channels and flood plains.

The clay present in the formation has been preconsolidated by a process of desiccation. Numerous wetting and drying cycles have produced a network of randomly oriented and closely spaced joints, which are sometimes slickensided, that is, have a shiny appearance when exposed. The joint pattern strongly influences the engineering behavior of the soil.

The sand layers vary in compactness from loose to very dense, and in thickness from a fraction of an inch to many feet due to an irregular depositional environment. Sands are generally subrounded to subangular and vary from coarse to very fine, are poorly graded, and often contain significant amounts of silt-sized particles in the sand matrix. The coastal plain in this region has a complex tectonic geology, several major features of which are: Gulf Coastal geosyncline, salt domes, major sea level fluctuations during the glacial stages, subsidence and faulting activities. Most of these faulting activities have ceased for millions of years, but some are still active.

5.2 Geologic Faulting

The tectonic history of the Texas Gulf Coast includes a relatively stable depositional cycle since the Cretaceous Period (about 65 million years). During this period the area has been subjected to deposition of clays, silts, and sands resulting in over 30 thousand feet of sedimentary rocks.

Underlying this clastic sequence are salt formations, which have migrated upwards to produce the typical salt dome features associated with the Texas Gulf Coast. In conjunction with salt movement, dewatering and compaction of some of the deeper sediments in the basin have resulted in the development of growth faults.

A literature review of surface faults was made from published reports. The primary objective of this review was to evaluate available information from published reports and open file reports. Based on our review, we did not find documented faults within 2 miles radius from the site location. We believe that faulting should not impact the project site; however, it should be noted that unmapped faults that could impact the project site might exist within the project area. A detailed fault study was not within the scope of this study.

5.3 Soil Stratigraphy

Our interpretation of soil and groundwater conditions at the project site is based on information obtained at the boring locations only. This information has been used as the basis for our conclusions and recommendations. Significant variations at areas not explored by the project boring may require reevaluation of our findings and conclusions. Soil stratigraphy encountered at boring at different depths is detailed below.

<u>Odor control structure (Boring B-1):</u> At boring B-1, silt with sand was observed at the top 6 feet with layer of sandy lean clay in between 6 to 12 feet followed by fat clay to the termination depth of 15 feet at this boring.

<u>Secondary clarifier borings (Borings B-2,3,4):</u> The generalized subsurface conditions encountered in Borings B-2, B-3 and B-4 is shown in the table below.

Boring No.	Depth, Feet		Material
	From	To	
B-2,3,4	0	8	Medium Dense Silt With Sand (ML) and/or Very Loose Poorly Graded Sand (SP)
B-2,3,4	8	22	Stiff to Very Stiff Clay (CH/CL)
B-3	22	26	Medium Dense Sandy Silt (ML)
B-2,4	22	28	Medium Dense Sandy Silt (ML) and Medium Dense Silt With Sand (ML)
B-2,3,4	28	40	Very Stiff Clay (CL/CH/CL-ML)
B-2	40	Below	Dense to Very Dense Silty Sand (SM)
B-3,4	40	Below	Stiff to Very Stiff Clay (CH/CL) and Dense to Very Dense Clayey Sand (SC)

Details of the subsurface stratigraphy encountered in the borings are shown on the boring logs presented in Appendix A.

5.4 Groundwater Conditions

Groundwater was not encountered at boring B-1 during the drilling operation. Groundwater could not be measured during drilling at B-2 and B-4 since wet rotary drilling started at 18 feet and 24 feet depth, respectively. No groundwater was observed at B-2 and B-4 until switched to wet rotary.

Groundwater was observed at a depth of 15 feet during the drilling operations at boring B-3. It should be noted that groundwater levels determined during drilling may not accurately reflect the true groundwater conditions, and therefore should only be considered as approximate. Groundwater levels measured in open standpipe piezometers are, on the other hand, more accurate; however, water level fluctuates seasonally and in response to rainfall. Other factors that might impact piezometric groundwater levels include leakage from existing sewers and/or sanitary sewers.

6 SECONDARY CLARIFIER DESIGN RECOMMENDATIONS

6.1 General

Mat foundations or deep foundations such as driven piles or drilled piers could be used to support the clarifier. Due to the relatively low applied pressure, and strong soils, deep foundations are not cost effective for this project. Recommendations for mat foundations are provided below. We understand that the bottom of the clarifier will be located at a depth approximately between 8.5 and 20.5 feet below existing grade.

6.2 Foundation Recommendations

Foundations for the proposed structures must satisfy two basic design criteria. First, the bearing pressure transmitted by the foundation should not exceed the allowable bearing capacity computed with an adequate factor of safety. Second, foundation movement due to soil volume change must be within desirable limits.

The soil borings drilled at the site revealed the presence of a clay layer at the foundation depth. As indicated to us by Weston Solutions, the clarifier will have a sloped bottom starting at a depth of approximately 8.5 feet to a depth of approximately 20.5 feet (Appendix F). Based on the properties of this cohesive soil deposit, the allowable net bearing pressure for a mat type founded at a depth of 8.5 to 20.5 feet below grade should be limited to 4,000 psf, which includes factor of safety of three. This bearing capacity recommendation assumes that the base of excavation is adequately dewatered if ground water is encountered and the bearing surface is relatively dry and undisturbed. The applied net bearing pressure may be determined by the following criteria.

1. Summing the load applied to the foundation, the weight of the foundation, and the weight of any soil backfill placed directly above the foundation;
2. Subtracting the weight of soil excavated from above the foundation depth; and
3. Dividing the total by the base area of the foundation.

The mat foundation may be designed as a semi-flexible component. In this case, a subgrade reaction modulus (k) may be required for the structural design analysis. Based on the encountered subgrade material, a reaction modulus value of 50 pounds per cubic inch (Table 4-1, Technical Manual 5-809-12/AFM 88-3, Chapter 15, 1987, US Army) is recommended.

6.3 Uplift

Buoyant uplift pressures will act on the base of the unit located below the water table. Buoyant uplift pressure is a function of the depth to groundwater. The largest uplift pressure will occur when groundwater is at the ground surface and the clarifier is empty. We recommend that the secondary clarifier be designed to resist buoyant uplift based on the dead weight of the structure. For normal operations, minimum factor safety of 1.5 is recommended against floatation.

If dead weight alone is inadequate to resist uplift forces, a toe may be constructed into the soil at the base of the structure. Construction of a toe is most appropriate when open-cut excavation methods are used. The toe may consist of a bottom slab that extends into adjacent backfill. The weight of the material above the extension can then be relied upon to resist the uplift forces. The total unit weight of soil for a compacted backfill will be about 120 pcf. Backfill should be compacted as recommended in section 6.8.

6.4 Basal Heave

A major problem, particularly for deep excavation in clay, is expansion and/or lateral flow into the excavation base so that the base elevation rises and values of 1 to 2 inches are very common while values up to 8 inches are reported in literature (Bowles, 1996, page 542, 5th Edition). Bottom heave/rebound in excavations lying above compressible in clay soils is influenced by shear strength, loading history of the clay and undrained elastic unloading strains in these strata. There may be additional long-term heave due to wetting of soil following reduction in pore water pressure following removal of overburden in excavation. We expect the heave after excavation will be less than 1 inch. The Stability Number, N, can be utilized to indicate the performance of an excavation in clay.

$$N = \gamma H/c$$

Where:

γ	=	average effective unit weight, pcf
H	=	Height, feet
C	=	undrained shear strength, psf

Ground movement becomes significant with N values above 3 or 4 and base failure is likely with values greater than 5.

Therefore, at $H = 18$ feet, $\gamma = 125$ pcf, and $c = 2{,}500$ psf (average),
$N = 0.9$

Hence, no significant ground movement is anticipated.

6.5 Settlement

No significant settlement (greater than 1-inch) is expected to occur after the placement of the clarifier due to recompression. There will be no net increase in the pressure at the bottom of the unit due to the excavation of soils and replacement by the structure. Since the foundation will be placed between 8.5 and 20.5 feet below the existing grade and the load on the subsurface would be less than that previously experienced, settlement of foundation shouldn't be a concern. The differential settlement of the proposed structure is also expected to be less than 1-inch.

Any lowering of groundwater table may result in settlement of clay because of increase in effective stress if this increase combined with the dead and live load of the proposed structure exceeds the stress removed because of excavation.

6.6 Lateral Earth Pressures

The pressures which a soil can be expected to exert on the exterior walls of an underground structure depend on the type of soil and the construction technique. If the construction method encompasses backfilling along the exterior walls of the underground structure, then the lateral earth pressures become a function of the type of backfill and its method of placement. If construction is performed by incorporating temporary excavation sheeting as part of the permanent structure, then the lateral earth pressures would vary with the type of the existing soil. For a backfill type of soil, the following considerations should be taken into account when estimating lateral earth pressure acting on the walls of a permanent underground structure.

1. Over-compaction of backfill and utilization of highly plastic expansive clay backfill are practices, which generally produce the highest wall pressures. In these cases, horizontal earth pressures exceeding the vertical pressure can be expected. Backfill selection and method of placement are critical.
2. Bank sand, cement stabilized sand and select cohesive soil are some types of fill material that can be considered for this project. The backfill (cohesive or cohesionless) should be in accordance with City of Houston Standard Specification 02316. Cement stabilized sand should be in accordance with City of Houston Standard Specification 02321.

Excavated soils from this site are expected to be cohesive soils, and cohesionless sand and silt. The backfill to be used should be tested prior to use as explained in the following section.

The backfill around the underground structures will impose active to at rest earth pressures against the embedded walls. Based on the type of material expected, typical soil parameters (Table 3-1, Technical Manual 5-818-1/AFM U-3, Chapter 7, 1983, US Army, see Appendix D) were used conservatively to compute the coefficients of earth pressure to be used at this site are presented in the following table.

The lateral earth pressures can be evaluated generally as a linear distribution of lateral effective overburden pressure on the wall due to nonhomogeneous conditions (Fang, 1991). For example, lateral earth pressure, p, can be computed as shown below:

$$P \text{ (psf)} = k(\gamma' z + q_s)$$

Where:

γ'	=	effective unit weight, pcf
z	=	depth, feet
q_s	=	surcharge, psf
k	=	coefficient of lateral earth pressure (active, at-rest, or passive)

While a circular wall relatively limits the deflection, we recommend the use of at-rest earth pressures for the design of the walls. We also recommend a proper drainage system behind the walls to discourage build-up of hydrostatic pressure. Based on the boring logs, the water table should be assumed at a depth of 15 feet from the ground surface.

Fill Type	Total Unit Weight (γ) pcf	Friction Angle (ϕ)°	Coefficient of Active Earth Pressure (K_a)	Coefficient of Earth Pressure at Rest(K_0)
Select Fill (PI ≤ 20)	130	28	0.361	0.531
Bank sand, gravel, or cement stabilized sand	135	34	0.283	0.441
Cohesive soil (PI ≥ 20)	125	19	0.509	0.674

An allowance for lateral loads due to surcharge must be considered. We recommend a minimum surcharge load of 250 psf be included in the design.

6.7 Secondary Clarifier Construction Open-Cut Method

The excavation should satisfy two requirements. First, the soils above final grade must be removed without disturbing the soil below excavation grade, which will support the structure.
Second, the sides and base of the excavation must be stable to prevent damage to adjacent facilities as a result of either lateral or vertical movements of the soil. For vertical-sided excavations, the principal problem related to satisfactory excavation procedures is the design and installation of sheeting and bracing to form vertical sides of the excavation. In addition, a satisfactory excavation procedure must include an adequate construction dewatering system if ground water is encountered during construction to lower and maintain the water level at least a few feet below the lowest excavation grade. The existing clarifier located nearby is at 40 feet away from the proposed new clarifier and its bottom is at the same level as the proposed bottom of the new clarifier and thus its foundation and stability is not expected to be effected by the construction of the proposed clarifier. However, we recommend monitoring the existing clarifier during construction of the proposed clarifier as a precaution measure.

The excavation may be shored, laid back to a stable slope or some other equivalent means should be used to provide a stable side slope. The contractor may design a shored excavation as an alternate method to sloped excavations. Soils at this site have an OSHA Soil Classification Type A and C.

6.8 Structural Fill and General Earthwork Requirements

Provisions for backfill around the underground structure should be in accordance with City of Houston Standard Specification Section 02316 or equivalent. Select fill if required shall be used to one-foot below paved areas. Select fill should consist of sandy clay with a liquid limit less than 40 and a Plasticity Index between 8 and 20. Fill material should be placed in loose lifts not exceeding eight inches in thickness and should be compacted to 95 percent of Standard Proctor maximum dry density as determined by ASTM D698.

7 RAS PUMP STATION AND SCUM PUMPS PAD

7.1 General

The project also involves the construction of a RAS pump station (Appendix G) and an on-grade pad supporting scum pumps (Appendix F, next to the secondary clarifier near B-3). Based on the RAS pump station drawings provided to us, we understand that the existing grade will be raised by 2 feet (El. 117.71) using select fill and the slab will be poured on fill material with perimeter grade beams extending 2 feet deep below the existing grade. The RAS pump station will cover an area of 20 feet long and 12 feet wide. The pad supporting scum pumps (near clarifier) will be formed on a select fill of finished grade El. 115.94 feet and will be 5 feet long and 3.5 feet wide.

We do not expect the soil to shrink/swell because of the presence of silt and sandy lean clays underneath the slabs. A Potential Vertical Rise (PVR) of 0.20 inches was estimated by the TEX 124-E method for the upper soils at the site which is acceptable. The PVR represents the potential ability of a soil material at a specific density, moisture and loading condition to swell. It indicates the potential movement of the soils that may be triggered if the soils are wetted up from a relatively dry condition.

7.2 Foundation Recommendations

As planned, these structures can be supported on a grade-supported slab (mat) system. Using a factor of safety of 3, an allowable bearing capacity of 1,900 psf can be used for a RAS pump station. For scum pumps pad of 5 feet long and 3.5 feet wide placed on compacted structural fill, an allowable bearing capacity of 1,500 psf can be used.

7.2.1 Construction Recommendations

RAS pump station pad will be placed 2 feet above the existing soil. The exposed surface should be checked and tested to identify any soft or weak areas, and debris or undesirable materials which should be removed and replaced with select fill. This should include leveling the exposed surface soils and compacting the top 3 inches. The minimum compaction should be 95% of the maximum dry density in accordance with ASTM D 698. Cohesionless soils, if encountered while excavation should be compacted with vibratory or sheep footed rollers to achieve proper compaction.

As mentioned earlier, the select fill should consist of sandy clay with a liquid limit less than 40 and a Plasticity Index between 8 and 20. Fill material should be placed in loose lifts not exceeding eight inches in thickness and should be compacted to 95 percent of Standard Proctor maximum dry density as determined by ASTM D698. Water should not be allowed to accumulate within the excavations. Should water accumulate, then any wet or softened soils should be removed or reworked if appropriate, and subsequently re-compacted. We recommend that 1-2% grade sloping away from the structure be provided so as to discourage water logging in the foundation areas. For RAS pump station, the perimeter grade beams should extend at least 2 feet below the existing grade.

8 ODOR CONTROL STRUCTURE FOUNDATION RECOMMENDATIONS

8.1 General

We understand that the project will involve the construction of pier supported platform to support Odor Control Structure (Appendix G). Based on the information provided to us by Weston Solutions, Inc., we understand that the platform will be no more than 4 feet above ground. Boring B-1 was drilled within the footprint of this structure to provide foundation recommendations.

8.2 Drilled Piers

Based on the Odor Control Structure drawings provided to us (Appendix G), we understand that the drilled and underreamed piers will be used to support the elevated platform. The structural load is estimated to be 30 kips on each pier as indicated to us by Weston Solutions. To support 30 kips pier load, we recommend placing the bottom of underreamed pier with 3.0 feet base diameter at 8 feet depth below the existing grade. This should keep the top of the bell out of the granular soil. The bell-to-shaft diameter ratio can be 2:1 forming 30° angle with the vertical.

<u>Drilled Footing Design and Construction Considerations</u>. The drilled piers should be wide enough for cleaning and inspection purposes. Each shaft should be provided with sufficient vertical steel reinforcement extending from the top to within six inches of the bottom of the piers to resist tension stresses created by lateral forces.

A minimum clearance of one diameter of the larger footing should be provided between the drilled shafts to develop the recommended bearing pressures and to control settlements. If a clearance of one diameter cannot be maintained in every case, the above bearing capacities should be reduced by 20 percent for a clearance between one-half and one diameters. Drilled shafts closer than a clearance of one half of the bell diameter are not recommended.

Based on our groundwater observations, excavations drilled to a depth of 8 feet may not encounter water but the contractor should be prepared to dewater the footing excavation if groundwater is encountered. To prevent the silt from caving during construction, temporary steel casings or drilling mud should be used during footing installation.

It is recommended that each foundation excavation be inspected by the Project Engineer, Architect, or Owner's Representative prior to placing concrete. The excavation should be checked to verify that a) the excavation has been constructed to the specified dimensions at the correct depth and into appropriate stratum as recommended in this report, b) the footings are concentric with columns, and c) the loose cuttings and any soft-compressible materials have been removed from the bottom of the excavation. Placement of concrete should be accomplished as soon as possible to reduce changes in the state of the stress and caving of the foundation soils. No piers should be poured without the prior approval of the Project Engineer, Architect, or Owner's Representative.

9 UTILITIES DESIGN CRITERIA AND RECOMMENDATIONS

9.1 General

Based on the information provided to us by Weston Solutions, Inc., we understand that there will be a clarifier influent line at approximately 18 feet below ground surface at the center of the clarifier, and approximately 15 feet below ground surface at the periphery of the clarifier, and other piping will be 3 to 5 feet below the ground surface. Our analyses and recommendations for the installation of utilities using open cut excavation method are presented below.

9.2 Geotechnical Parameters

Geotechnical design parameters are presented in the following table. Design parameters given in the table are based on field and laboratory test data obtained at boring locations only and at the approximate invert depth. It must be noted that because of the nature of the soil stratigraphy at this site, parameters at locations away from the borings may vary substantially from values reported in the table.

Boring No.	Approximate Invert Depth (ft)	Soil Description at Invert Depth	Total Unit Weight (pcf)	Undrained Shear Strength (psf) or Friction Angle (deg)	Allowable Bearing Pressure (psf)	E'n, Long Term (psi)
B-2	18	Stiff Sandy Lean Clay	133	2,460	4,100	1,000
B-3	15	Very Stiff Lean Clay	136	2,580	4,300	1,000
B-4	15	Very Stiff Fat Clay	128	2,740	4,567	1,000
B-3	5	Medium Dense Silt	120	30 °	1,600	1,000
B-4	5	Medium Dense Silt	120	30 °	1,600	1,000

The values shown in the above table represent our interpretation of the soil properties based on the available laboratory and field test data. Use of the soil properties shown above may or may not be appropriate for a particular analysis, since choice of design parameters often depends on whether total or effective stress analysis is used, rate of loading, duration of loading, geometry of loaded area, and other factors. The total unit weight values shown above represent our interpretation of soil unit weight at natural moisture content. The undrained shear strength and allowable bearing pressure values represent our interpretation of the shear strength in clay soils based primarily on the results of unconsolidated undrained compression tests and hand penetrometer tests. The allowable bearing pressures include a factor of safety of three.

9.3 Pipe Design

The loads imposed on underground pipes depend principally upon the method of installation, the weight of overburden soils, roadway traffic load, and loads due to existing surface structures. For design of rigid pipes installed using open-cut excavation methods, loads due to overburden and traffic can be determined from Plate 5.

The traffic load applied to the pipe can be calculated using 85% of wheel load with an impact factor of 1.5 for one foot of soil cover, 50% of wheel load with an impact factor of 1.35 for 2 feet of cover, and 30% of wheel load with an impact factor of 1.15 for 3 feet of cover. This results in a total design traffic load on the pipe or box culvert of about 1.28, 0.68 and 0.35 times the wheel load for 1, 2 and 3 feet of cover, respectively. For pipes with four or more feet of cover, the traffic loads may be taken as a surcharge equivalent to 250 psf.

The design of flexible pipes requires the modulus of soil reaction of the native soil (E_n') in the trench wall as input. The E_n' values are based on empirical relationships to the soil consistency as defined by unconfined compression tests for cohesive soils. E_n' values for the native soils are presented in the table above. The E_n' values for short-term conditions in cohesive soils may be assumed to be 1.5 times the long-term values. These values are based on the soil data obtained at the boring locations only and may be used for the noted invert depth zone.

9.4 Open Cut Bedding and Backfill

<u>Pipe Bedding.</u> Pipe bedding should be performed according to City of Houston Standard Specification Section 02317, part 3.08 with item (J) for the waste water lines. Pipes installed using open-cut trenches should be placed using City of Houston Drawing No. 02317-04.

The excavations should be performed with equipment capable of providing a relatively clean bearing area. Stable soils are essential to provide a strong base during construction. In addition, stable soils enhance trench bottom stability, support for bedding compaction, and minimize possible pipe settlement. Whenever soft foundation soils are encountered during trench excavation, in accordance with section 02317, item 3.07.C, we recommend over excavating 3 feet below the base of the foundation and replacing with on-site soils or approved bedding material compacted to at least 95% of maximum dry density determined by ASTM D698 in loose lifts not exceeding 8 inches.

Trench Backfill. Trench backfill for utilities should be in accordance with Section 02317, Excavation and Backfill for Utilities, and in particular item 3.09 of the City of Houston Standard Specifications, January 2011. Pipe backfill should be in accordance with Drawing No. 02317-04.

Pipe embedment should be in accordance with Section 02317 and the material used should be in accordance with City of Houston Standard Specification Sections 02320.

Trench zone backfill is that portion of trench backfill that extends vertically from the top of pipe embedment up to pavement subgrade or up to final grade when not beneath pavement. Trench zone backfill for utilities may consist of bank run sand, select fill, or random backfill material as specified in City of Houston Standard Specification Section 02320.

9.5 Thrust Force Design Recommendations

Piping System Thrust Restraint. Unbalanced thrust forces will be developed in force main lines due to changes in direction, cross-sectional areas, or when the pipe is terminated. These forces may cause joints to disengage if not adequately restrained. There will be a slight loss of head due to turbulence in bends in the pipes. This loss will cause a pressure change across the bend, but it is usually small enough to be neglected.

The thrust force may require more reaction than is available just from the pipe bearing against the backfill. In order to prevent intolerable movement and overstressing of the pipe, suitable buttressing should be provided. In general, thrust blocks, concrete encasement, restrained joints and tie rods are common methods of providing reaction for the thrust restraint design. The thrust restraint design provisions described in this section are based on the American Water Works Association (AWWA) Manual M9 (2008) Concrete Pressure Pipe.

Various types of thrust restraint systems are used depending on the type of pipes and installation conditions. The thrust force at the bends should be evaluated based on the procedures described in Chapter 9 of AWWA manual M9.

Frictional Resistance. The unbalanced force produced by grade and alignment changes can also be resisted by friction on the pipe. The length of pipe will be formed by tying or welding joints together for the distance required to develop adequate capacity or by encasing the pipe in concrete. The resisting frictional force, F_R is computed, for most cases, as

$$F_R = f\,(2W_e+W_w+W_p)$$

Where:

f = Coefficient of friction between pipe and soil

W_e = Weight of soil over pipe in lb/ft

W_w = Weight of contained water in lb/ft
W_p = Weight of pipe in lb/ft

The friction value depends on the material in contact with the pipe and the soil used in the backfill around the pipe. For pipe surrounded by compacted sand or crushed stone, the friction between the pipe and soil may be based on a friction angle of 30 degrees. The allowable coefficient of friction, f, of 0.28, 0.23 and 0.18 can be used for concrete, steel and PVC pipes, respectively. This value includes a factor of safety of 2.0. The weight of soil above the pipe will depend on the soil unit weight and the pipe depth. For compacted soils used for backfill, a total unit weight of 125 pcf can be used.

In low cover situations, where depth of cover is less than 50% of pipe diameter, we should be contacted to evaluate the impact of shallow cover on thrust resistance.

Tied joints are used to transmit thrust across joints. These ties may be welded or harnessed joints. Joints may be welded in the field in order to transmit the thrust involved. Information concerning types of harnessed joints available and size and pressure limitations can be obtained from the pipe manufacturers.

10 UTILITY CONSTRUCTION CONSIDERATIONS

10.1 General

This section is intended to address issues that might arise during construction. Our recommendations are intended for use as guidelines in dealing with particular soil conditions. The topics addressed in this section include trench excavation stability, groundwater control, and open-cut construction considerations.

The recommendations contained herein are not intended to dictate construction methods or sequences. Instead they are provided solely to assist designers in identifying potential construction problems related to excavation, based upon findings derived from sampling. Depending upon the final design chosen for the project, the recommendations may also be useful to personnel who observe construction activity.

Prospective contractors for the project must evaluate potential construction problems on the basis of their review of the contract documents, their own knowledge of and experience in the local area, and on the basis of similar projects in other localities, taking into account their own proposed methods and procedures.

10.2 Open Cut Excavation Considerations

Excavations should satisfy two requirements. First, the soils above final grade must be removed without disturbing the soil below, which will support constructed facilities. Second, the sides of the excavation must be stable to prevent damage to adjacent streets and facilities as a result of either vertical or lateral movements of the soil. In addition, a satisfactory excavation procedure must include an adequate construction dewatering system to lower and maintain the water level at least a few feet below the lowest excavation grade.

<u>Excavation Stability</u>. Excavations shall be shored, laid back to a stable slope or some other equivalent means may be used to provide safety for workers and adjacent structures. Earth

pressures for braced excavations are presented on Plate 4. Assessment of the need for excavation sloping, use of trench boxes, or other measures required to provide a stable excavation, and the use of appropriate construction practices and/or equipment is the contractor's responsibility. The following comments are intended to represent common solutions to stability problems encountered in similar soil conditions in the Houston area, and may not be construed as excavation system design recommendations. The excavation operations shall be performed in accordance with 29 CFR Part 1926 subpart P, as amended, including rules published in the Federal Register, Vol. 54, No. 209, dated October 31, 1989, as a minimum. In addition, the provisions of legislation enacted by the Texas Legislature and City of Houston should be satisfied.

Boring No	**OSHA Soil Type**			
	Depth of Trench (feet)			
	0-5	**5 – 10**	**10-15**	**15-20**
B-1	C	C	C	-
B-2	C	C	C	C
B-3	C	C	C	C
B-4	C	C	C	C

In general, it is our opinion that the pressure distribution (for braced walls) should be used for design of sheeting or trench boxes. To reduce the potential for ground movement adjacent to the top of the excavation, the bracing should be preloaded in stages as the excavation is deepened. The detailed earth pressure diagram is presented on Plate 4.

The planned construction will be performed along alignments near existing utility installations (either crossing or paralleling the new alignments). The contractors should be aware of potential excavation stability problems while working in the vicinity of old trenches and the excavation system should be designed to accommodate this weak material (trench backfill).

The vertical walls of excavations should be located a safe distance from existing utilities in order to prevent movement in the soil mass behind the excavation that may adversely affect the utilities. We recommend that the horizontal distance should be 4 feet for excavation depths of up to 10 feet.

10.3 Select Fill and General Earthwork Recommendations

The select fill required to raise the grade or backfill should consist of sandy clay with a liquid limit less than 40 and a plasticity index between 8 and 20. Fill material that is used should be placed in loose lifts not exceeding eight inches and should be compacted to 95 percent of standard Proctor maximum dry density as determined by ASTM D698.

10.4 Groundwater Control

The contractor should plan for some appropriate dewatering system as explained in the paragraphs below.

Excavations may encounter groundwater seepage to varying degrees depending upon the groundwater conditions at the time of construction. Assessment of the need for groundwater control and installation of appropriate dewatering equipment is the contractor's responsibility.

The following comments are intended to represent common solutions to groundwater control problems encountered in similar soil conditions in the Houston area, and may not be construed as dewatering system design recommendations.

If water table is encountered at the time of construction, conventional pump and sump arrangement, deep wells or eductors may be utilized to lower the groundwater level to at least three feet below the excavation level. Well points are generally not effective below about 15 feet beneath the top of the well point, and deeper dewatering requires deep wells or eductors. In any case, the groundwater control system used must provide a relatively dry, stable base for construction.
Based on our field investigation, groundwater was encountered at 15 feet during the drilling operation. Groundwater may be present at the invert depth of the clarifier during construction. Control of groundwater if encountered, should be accomplished in a manner that will preserve the strength of the foundation soils, will not cause instability of the excavation, and will not result in damage to existing structures. Where necessary to this purpose, the water will be lowered in advance of excavation by the above methods.

Open pumping should not be permitted if it results in boils, loss of fines, softening of the subgrade, or excavation instability. Wells and well points should be installed with suitable screen and filter so that pumping of fines does not occur. Discharge should be arranged to facilitate sampling by owner's representative or engineer.

10.5 Spoil Disposal
Spoil from construction will be generated from trench excavations. Soils that will be excavated from this project area will consist of cohesive as well as cohesionless soils. Economically, possible uses of the cohesive spoil material may be limited to land reclamation, site grading, and final cover in sanitary landfill operations. These soils may not be suitable for use in engineered fill.

11 CORROSIVITY

11.1 General
Based on the sulfate lab results of the current condition of the soil (29.5 mg/kg), the water soluble sulfate in the soil is less than the criteria for corrosive environment for concrete as indicated in section 11.2 of this report. There are various systems and code requirements for assessing the potential impacts on structures, foundations, and buried utilities in contact with soil and groundwater depending on the material. Some common standards/codes are listed below:

- Reinforced Concrete – American Concrete Institute, ACI 318, Building Code Requirements for Structural Concrete, Chapter 4, Durability
- Ductile Iron Pipe – American National Standards Institute/American Water Works Association, C105/A21.5, Polyethylene Encasement for Ductile Iron Pipe.

Summaries of the relevant provisions from these codes and standards are presented below along with recommendations for this project.

11.2 Reinforced Concrete
Reinforced concrete is susceptible to damage due to corrosion of reinforcement and sulfate attack of the concrete. Per the ACI code reinforcement in concrete cast against and permanently exposed to earth needs a minimum of 3 inches cover. Precast concrete exposed to earth or water needs a

minimum cover of ¾ inches except for No. 14 bars, No. 18 bars, and prestressing tendons larger the 1.5 inches diameter.

In corrosive environments the ACI code provides requirements for maximum water-cement ratio, minimum concrete compressive strength, limitations on various concrete mix components (i.e. flyash, slag, silica fume, etc.), and/or requirements for certain types of cement. There are special requirements for the concrete mix design in the following conditions:

1. Low permeability concrete
2. Freeze/thaw exposure in a moist condition
3. Exposure to deicing chemicals, salt, salt water, brackish water, or spray from these sources
4. Water soluble sulfates in soils > 0.1% by weight or sulfate in water ≥ 150 ppm

Based on the test results and our understanding of the project requirements the only condition listed that may apply is low permeability concrete. ACI requires a maximum water cement ratio of 0.5 and a minimum 28 day compressive strength of 4,000 psi for low permeability concrete.

11.3 Ductile Iron Pipe

American Water Works Association (AWWA) has developed a point based system for determining if ductile iron pipe needs to be protected from aggressive soil subsurface conditions. Several measures of corrosivity are evaluated with points assigned for different values. The total of the points is used to determine whether special measures are needed to protect the pipe. The system considers the following properties shown with the range of little corrosivity potential:

1. Resistivity – greater than 2,500 ohm-cm
2. pH – between 4.0 and 8.5
3. Redox Potential - > +100 mV
4. Sulfides – Negative

For the full system refer to the standard. Where ductile iron pipe is being considered for the project special measures or alternate materials may be needed where the test results are outside the limits shown above and an analysis based on the full system should be used. Note that City of Houston Standard Specifications for Ductile Iron Pipe (Section 2501) require either cathodic protection or polyethylene encasement, therefore, a separate determination of corrosivity is not necessary.

11.4 Severe Environments

Severe environments for corrosion can exist due to other causes such as exposure to salt water, extreme low resistivity (< 500 ohm-cm), or presence of stray electrical currents. Stray electrical currents can occur in the vicinity of electric railways, industrial equipment, and ground beds for cathodic protection. It is possible that soils and groundwater impacted by chemicals such as petroleum hydrocarbons, chlorinated solvents, or other substances may present corrosion potentials that are not accounted for in the standards and codes summarized in this section. Where such conditions exist a corrosion engineer should be consulted regarding the need for measures to protect proposed subsurface structures and pipes.

12 ACCESS ROAD PAVEMENT RECOMMENDATIONS

12.1 General

We understand that the project may involve construction of access road or additional parking. We have used the information obtained from boring B-1 to provide the pavement recommendations in this section.

12.2 Pavement Sections

Due to the presence of silty material in the upper six feet, we recommend stabilizing the top 6 inches with 6 percent cement. Cement stabilization should extend at least 2 feet beyond the edge of the pavement. Based on results of the field and laboratory investigation program and City of Houston's minimum standards, the following rigid and flexible pavement sections may be used for this project.

Types of loads	Rigid Pavement Section	Flexible Pavement Section
Heavy Loads (Heavy Truck Traffic)	7" Concrete 6" Cement Stabilized Subgrade	3" Asphaltic Concrete 9" Limestone Base 6" Cement Stabilized Subgrade
Medium Loads (Medium Truck and Heavy Use Driveways)	6" Concrete 6" Cement Stabilized Subgrade	2.5" Asphaltic Concrete 8" Limestone Base 6" Cement Stabilized Subgrade
Light Loads (Automobile Parking Areas)	5" Concrete 6" Cement Stabilized Subgrade	2" Asphaltic Concrete 6" Limestone Base 6" Cement Stabilized Subgrade

We recommend using sections corresponding to heavy loads for the design of access road. The design sections should be reviewed if specific traffic loading information is available. We further recommend that an appropriate drainage system should be provided to drain the surface water as quickly as possible. Providing appropriate drainage system will reduce development of future pavement distress due to softened subgrade.

12.3 Preparation of Subgrade

Subgrade preparation for the proposed pavement section should consist of clearing, stripping, proof-rolling and cement stabilization. We recommend the following procedures for subgrade preparation:

1. Clear the proposed pavement areas. Grubbing operations should be performed to remove root systems of vegetation and loose gravel within the limits of the proposed construction.
2. Surfaces exposed after clearing and grubbing should be proof-rolled with heavy equipment, to identify any underlying zones or pockets of soft soils and to remove such weak materials.
3. In areas where soft, compressible or very loose soils are encountered, additional stripping may be required. Stripping should extend a minimum of two feet beyond the edge of the proposed pavement. If backfill is required, the fill material should be prepared as described in Section 9.3.

Mix about 6 percent cement with the upper six inches of the subgrade soils or backfill soils and compact it to 95 percent of maximum unit dry weight as determined by ASTM D558(11). Construction of cement treated subgrade should conform to City of Houston or equivalent Specifications. This is a preliminary estimate; actual amount of cement shall be determined for subgrade soils by conducting laboratory cement series tests on the exposed subgrade material during construction.

13 MONITORING

13.1 Excavation Safety

As required under OSHA regulations, the contractor should provide a "competent person" to inspect trench excavations daily before the start of work, as needed during the shift, and after every rainstorm or other hazard increasing occurrence. When the competent person finds evidence of a hazardous condition, exposed workers should be removed from the hazardous area until the necessary precautions have been taken to ensure their safety. A competent person means one who is capable of identifying existing and predictable hazards in the surroundings or working conditions which are unsanitary, hazardous or dangerous to workers, and who has authorization to take prompt corrective measures to eliminate them.

13.2 Construction Materials Testing

We recommend that backfill be monitored by an accredited testing laboratory to verify that construction is performed in conformance with project specifications. HVJ Associates routinely provides these services and would be pleased to do so for this project.

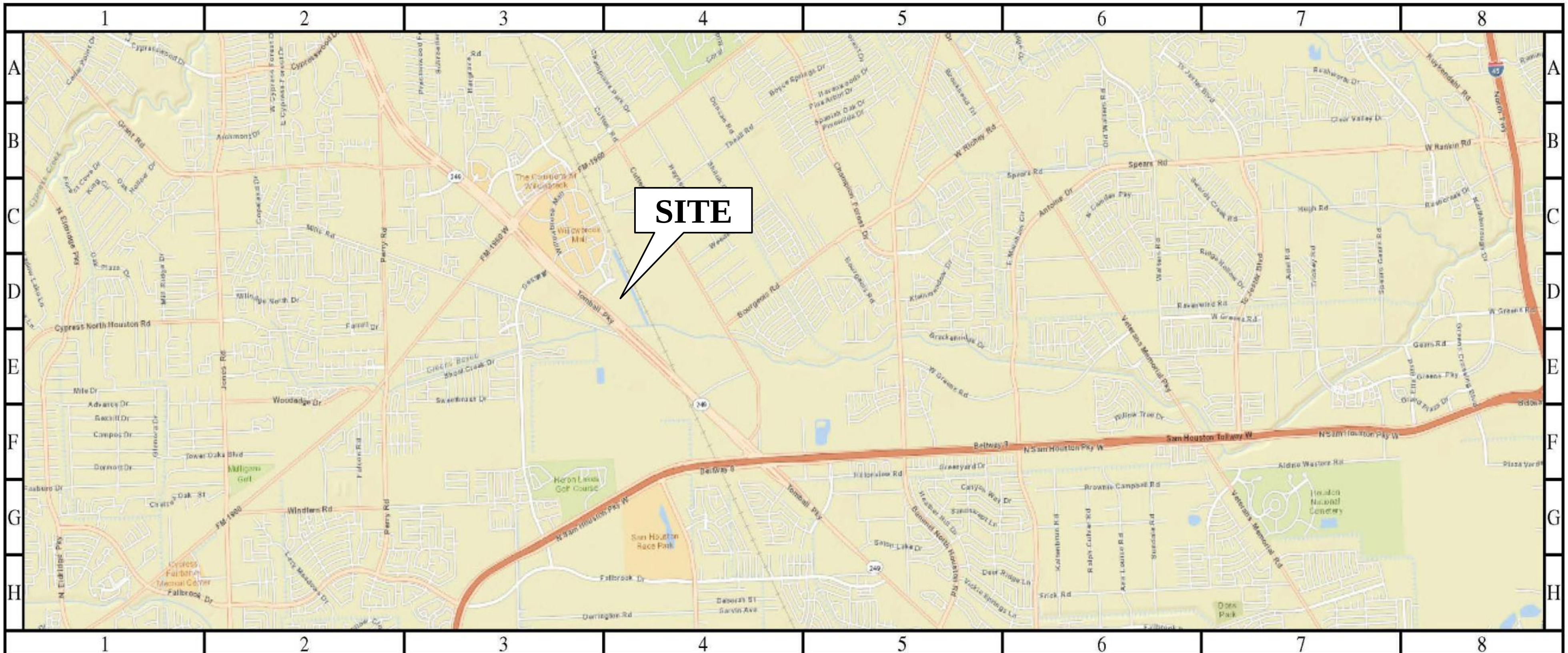
14 DESIGN REVIEW

HVJ Associates should be retained to review the final design plans and specifications for this project. During all excavation, grading and construction phases of this project, HVJ should provide the materials testing verification and observation services so our geotechnical recommendations may be interpreted and implemented correctly.

15 LIMITATIONS

This investigation was performed for the exclusive use of Weston Solutions Inc. and the City of Houston for the proposed wastewater treatment plant improvements at Willowbrook WWTP in Houston, Texas. HVJ Associates, Inc. has endeavored to comply with generally accepted geotechnical engineering practice common in the local area. HVJ Associates, Inc. makes no warranty, express or implied. The analyses and recommendations contained in this report are based on data obtained from subsurface exploration, laboratory testing, the project information provided to us and our experience with similar soils and site conditions. The methods used indicate subsurface conditions only at the specific locations where samples were obtained, only at the time they were obtained, and only to the depths penetrated. Samples cannot be relied on to accurately reflect the strata variations that usually exist between sampling locations. Should any subsurface conditions other than those described in our boring logs be encountered, HVJ Associates, Inc. should be immediately notified so that further investigation and supplemental recommendations can be provided.

PLATES



CITY OF HOUSTON

Department of Public Works and Engineering

Geographic Information & Management System (GIMS)

N W E S

1 inch = 4440 feet

DISCLAIMER: THIS MAP REPRESENTS THE BEST INFORMATION AVAILABLE TO THE CITY.
THE CITY DOES NOT WARRANT ITS ACCURACY OR COMPLETENESS.
FIELD VERIFICATIONS SHOULD BE DONE AS NECESSARY.

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DATE: 8/12/2013	APPROVED BY: ZA	PREPARED BY: SS

SITE VICINITY MAP
WILLOWBROOK WWTP IMPROVEMENTS
WBS No. R-000265-0104-3

PROJECT NO.: HG1312760	DRAWING NO.: PLATE 1

LEGEND:

APPROXIMATE BORING LOCATIONS

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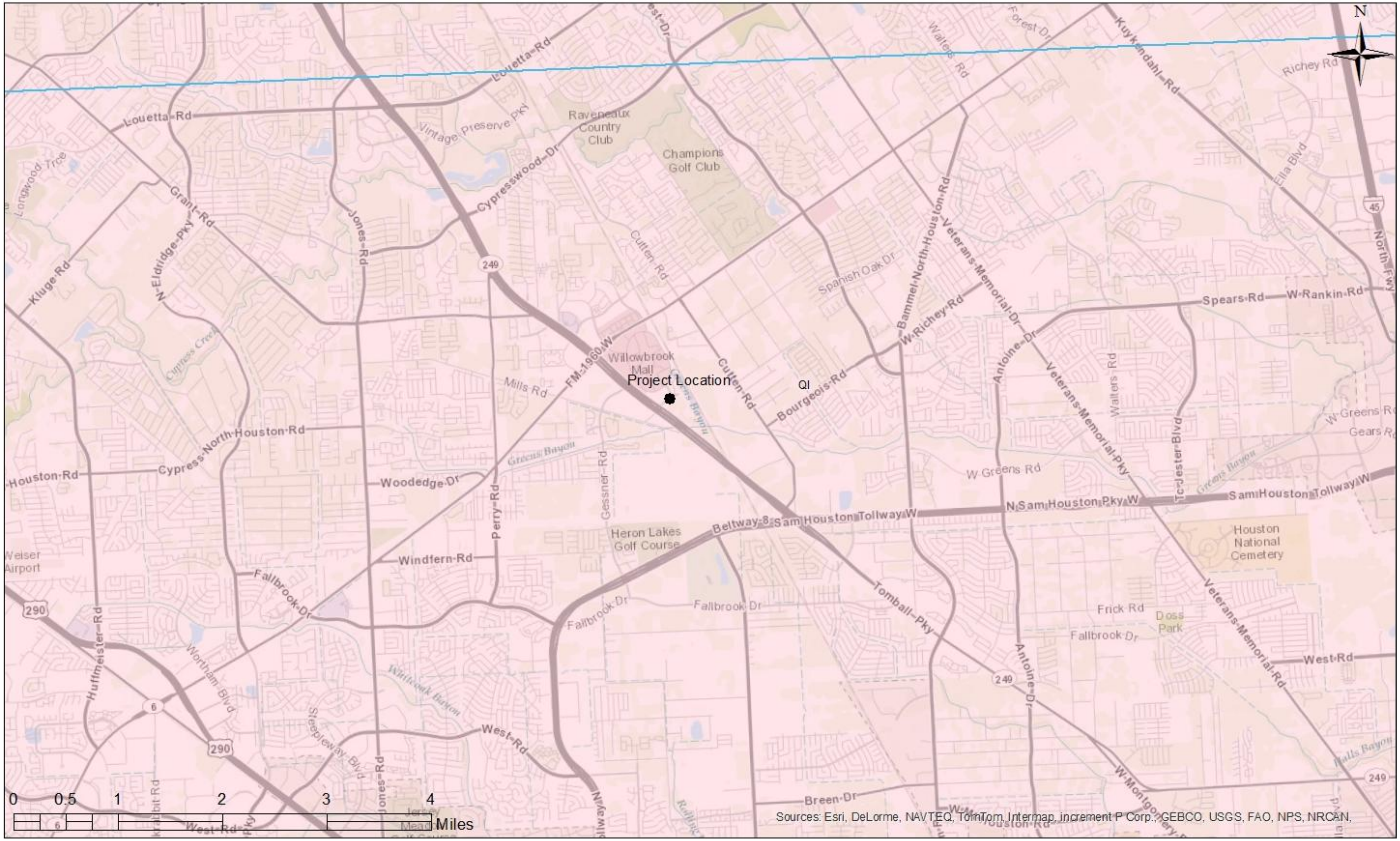
PLAN OF BORINGS
WILLOWBROOK WWTP IMPROVEMENTS
WBS No. R-000265-0104-3

PROJECT NO.: HG1312760	DRAWING NO.: PLATE 2



Ql

Lissie Formation - Upper part, clay, slit, sand, and very minor siliceous gravel of granule and small pebble size gravel more abundant northwestward, locally calcareous, concretions of calcium carbonate, iron oxide, and iron-manganese oxides common in zone of weathering; fluviatile; surface fairly flat and featureless except for numerous rounded shallow depressions and pimple mounds, bower part, clay, silt, sand, and minor amount of gravel; gravel slightly coarser than in upper part, noncalcareous, iron oxide concretions mare abundant than in upper part; fluviatile; very gently rolling;

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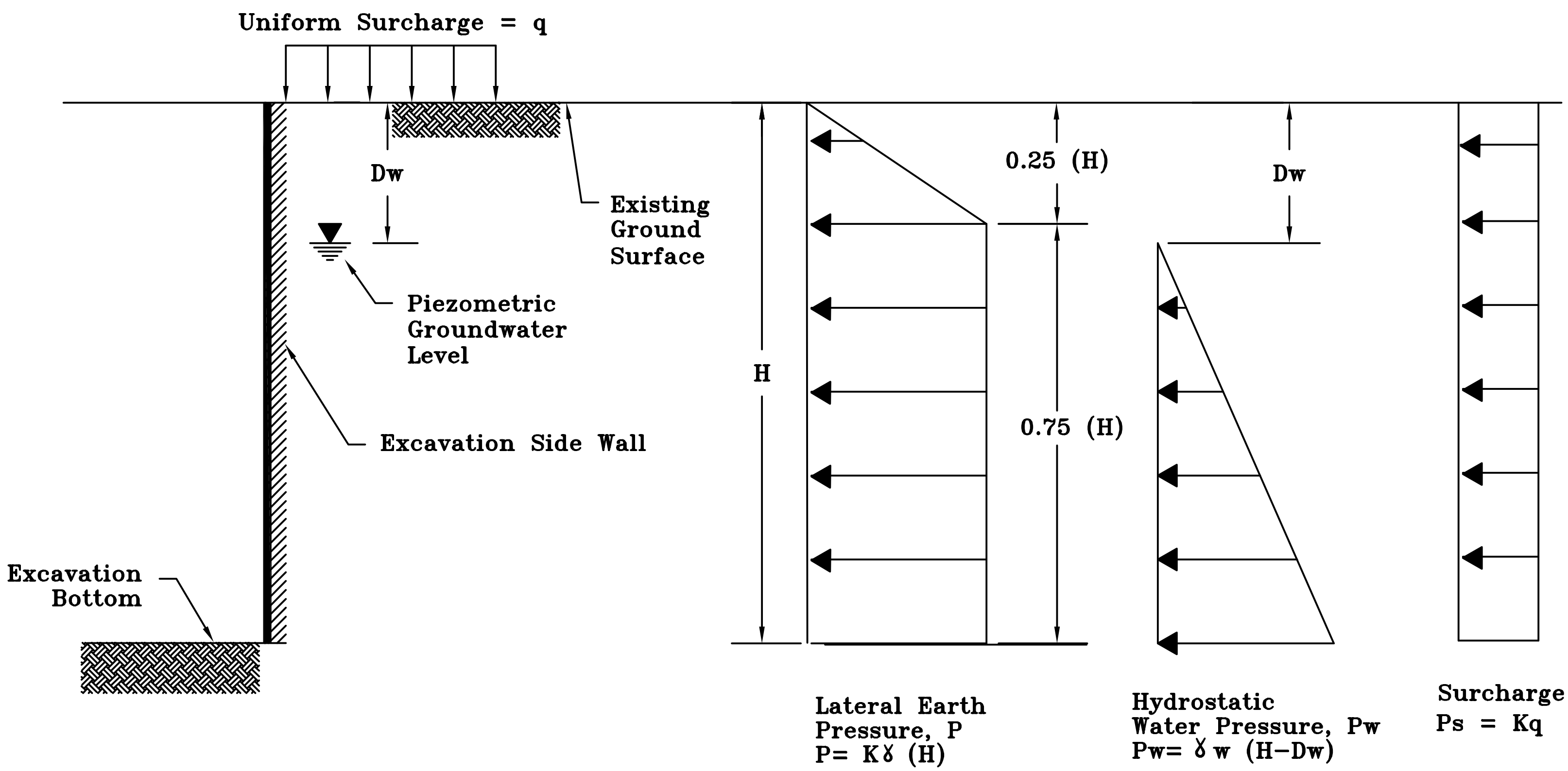
GEOLOGIC MAP
WILLBROOK WASTEWATER TREATMENT PLANT

PROJECT NO.: HG1312760	DRAWING NO.: PLATE 3



H, (ft) = Depth to Excavation Bottom

Ps, (psf) = Surcharge loading adjacent to Excavation wall

Dw, (ft) = Depth to groundwater below Existing grade

= Zero for temporary excavation

K = Lateral Earth Pressure coefficient

= Ka "active" for short−term conditions (use 0.50)

= Ko "at rest" for long−term conditions (use 1.0)

γ, (pcf) = Total unit weight above water table or submerged unit weight below groundwater level

γw, (pcf) = Unit weight of water = 62.4 pcf

Note: The pressure diagram shown is not appropriate for design of cantilever walls.

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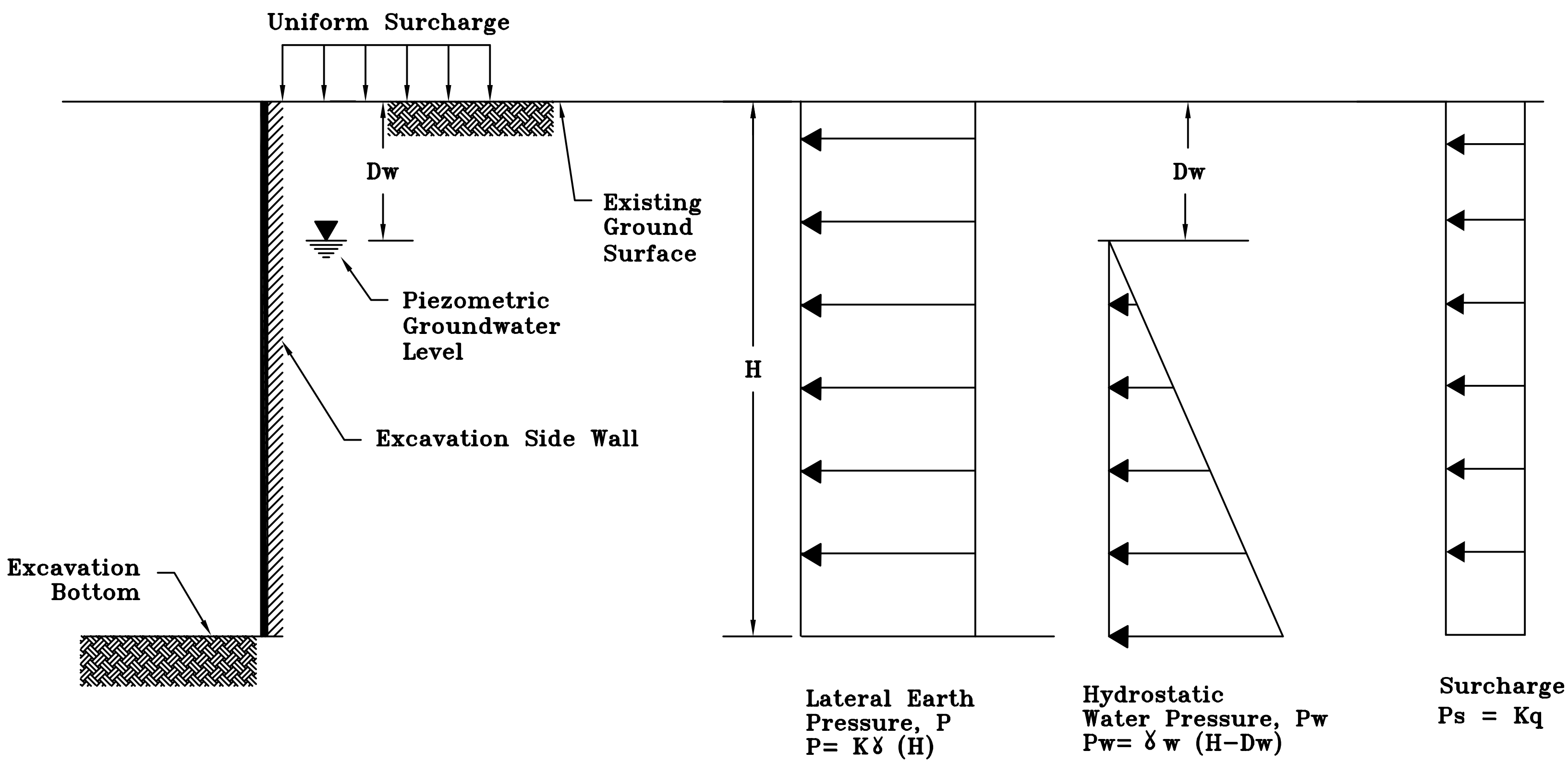
BRACED EXCAVATION
LATERAL EARTH PRESSURE DIAGRAM (CLAY)

PROJECT NO.:	DRAWING NO.:
HG 1312760	PLATE 4A



H, (ft) = Depth to Excavation Bottom

Ps, (psf) = Surcharge loading adjacent to Excavation wall

Dw, (ft) = Depth to groundwater below Existing grade

= Zero for temporary excavation

K = Lateral Earth Pressure coefficient

= Ka "active" for short-term conditions (use 0.35)

= Ko "at rest" for long-term conditions (use 0.50)

γ, (pcf) = Total unit weight above water table or submerged unit weight below groundwater level

γ_w, (pcf) = Unit weight of water = 62.4 pcf

Note: The pressure diagram shown is not appropriate for design of cantilever walls.

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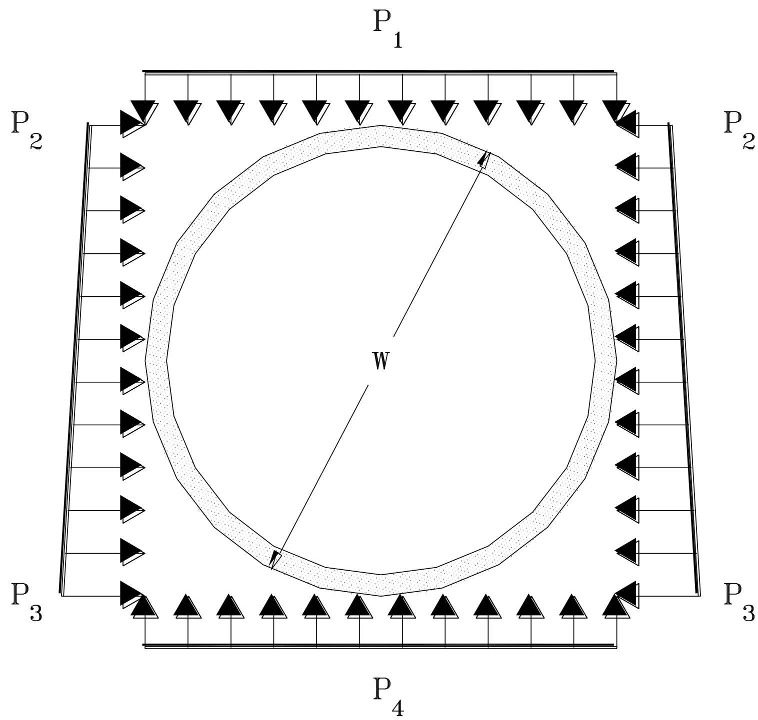
BRACED EXCAVATION
LATERAL EARTH PRESSURE DIAGRAM(SAND/SILT)

PROJECT NO.:	DRAWING NO.:
HG 1312760	PLATE 4B



For

$D_w \leq H$

$P_1 = \gamma D_w + (H - D_w)(\gamma - \gamma_w) + P_s + (H - D_w)\gamma_w$

$P_2 = [\gamma D_w + (H - D_w)(\gamma - \gamma_w) + P_s]K_o + (H - D_w)\gamma_w$

$P_3 = [\gamma D_w + (H + W - D_w)(\gamma - \gamma_w) + P_s]K_o + (H + W - D_w)\gamma_w$

$P_4 = \gamma D_w + (H + W - D_w)(\gamma - \gamma_w) + P_s + (H + W - D_w)\gamma_w$

For

$H < D_w < H + W$

$P_1 = H\gamma + P_s$

$P_2 = (\gamma H + P_s)K_o$

$P_3 = [\gamma D_w + (H + W - D_w)(\gamma - \gamma_w) + P_s]K_o + (H + W - D_w)\gamma_w$

$P_4 = \gamma D_w + (H + W - D_w)(\gamma - \gamma_w) + P_s + (H + W - D_w)\gamma_w$

For

$D_w \geq (H + W)$

$P_1 = H\gamma + P_s$

$P_2 = (\gamma H + P_s)K_o$

$P_3 = [(H + W)\gamma + P_s]K_o$

$P_4 = (H + W)\gamma + P_s$

Where

P_1, P_2, P_3 = Pressure imposed on pipe, psf
D_w = Depth of groundwater, feet
H = Depth of top of pipe from ground surface, feet
W = Diameter of pipe, feet
γ = Total Unit weight of soil, pcf
γ_w = Unit weight of water, pcf
P_s = Surcharge load, psf
K_o = Coefficient of earth pressure, (1.0 for clays and 0.5 for sands)

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DATE: 8/14/2013	APPROVED BY: ZA	PREPARED BY: SS

RIGID PIPE AND TUNNEL LINER LOADS
WBS No. R-000265-0104-3

PROJECT NO.: HG1312760	DRAWING NO.: PLATE 5

APPENDIX A

BORING LOGS AND KEY TO TERMS & SYMBOLS

LOG OF BORING

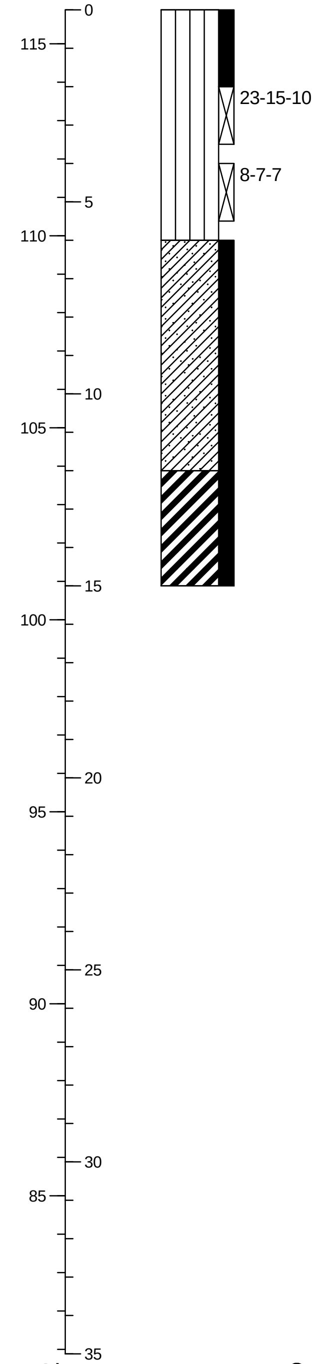
Project: Willowbrook WWTP Improvements | Project No.: HG1312760 | WBS No.: R-000265-0104-3

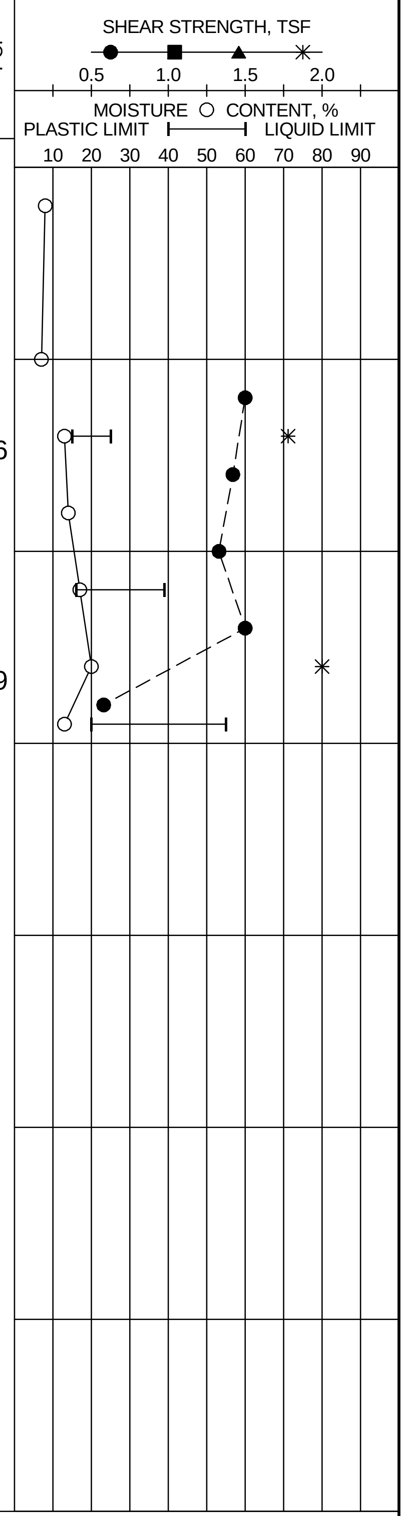
Boring No.: B-1 | Date: 7/19/2013 | Elevation: 115.88 feet

Groundwater during drilling: --- | Northing: 13,912,144.6 | Station: --

Groundwater after 24 hrs: --- | Easting: 3,066,428.5 | Offset: --

ELEV. DEPTH, FEET	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	SOIL/ROCK CLASSIFICATION	% PASSING NO. 200 SIEVE	DRY DENSITY PCF
0		Medium dense brown SILT WITH SAND (ML)	72.3	
	23-15-10		73.8	
	8-7-7		74.8	
6		Very stiff brown and gray SANDY LEAN CLAY (CL)		116
			82.6	
12		Stiff to very stiff reddish brown FAT CLAY (CH)		109





Shear Types: ● = Hand Penet. ■ = Torvane ▲ = Unconf. Comp. ✳ = UU Triaxial

See Plate 2 for boring location.

PLATE A-1

HVJ ASSOCIATES

LOG OF BORING

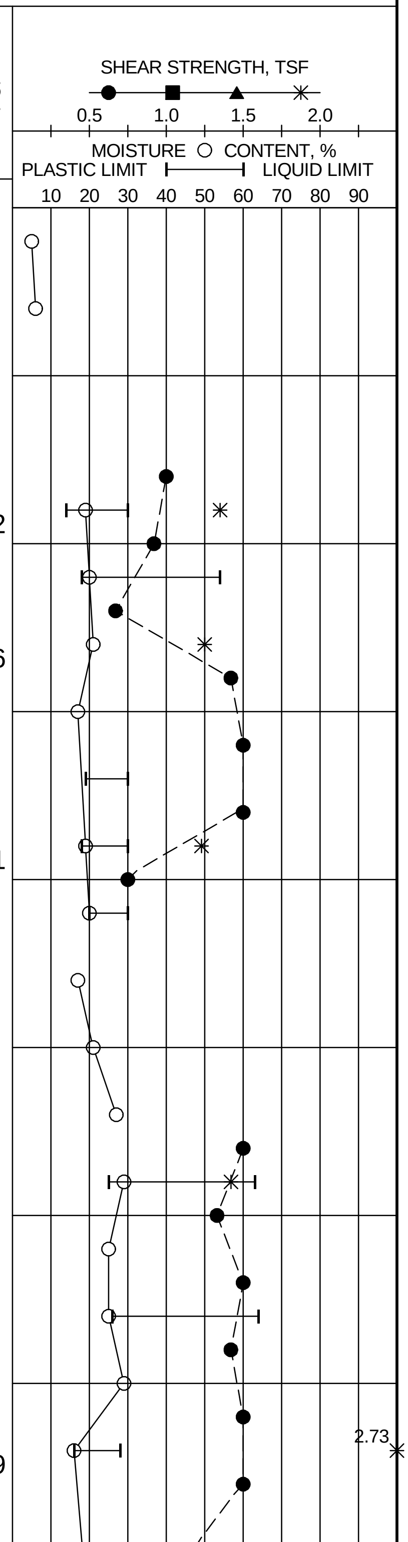
Project: Willowbrook WWTP Improvements
Boring No.: B-2
Groundwater during drilling: ---
Groundwater after 24 hrs: ---

Project No.: HG1312760
Date: 7/18/2013
Northing: 13,911,389.6
Easting: 3,066,757.4

WBS No.: R-000265-0104-3
Elevation: 114.71 feet
Station: --
Offset: --

ELEV. DEPTH, FEET	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	SOIL/ROCK CLASSIFICATION	% PASSING NO. 200 SIEVE	DRY DENSITY PCF
[0–8]	7-7-8 6-8-11 7-8-8	Medium dense brown SILT WITH SAND (ML)	69.7 74.7	
[8–10]		Very stiff reddish brown and gray SANDY LEAN CLAY (CL)		112
[10–16]		Stiff to very stiff reddish brown and gray FAT CLAY (CH)		106
[16–22]		Stiff to very stiff reddish brown and gray SANDY LEAN CLAY (CL) -w/ silt layer 20'-22'	98.7	111
[22–28]	7-8-12 11-15-17	Medium dense to dense brown and gray SILT WITH SAND (ML)	71.3 75.6 85.8	
[28–36]		Very stiff reddish brown FAT CLAY (CH)		97
[36–40]		Very stiff to hard reddish brown and gray SANDY LEAN CLAY (CL)	94.4	119

Elevation scale: 110, 105, 100, 95, 90, 85, 80, 75
Depth scale: 0, 5, 10, 15, 20, 25, 30, 35, 40



Shear Types: ● = Hand Penet. ■ = Torvane ▲ = Unconf. Comp. ✳ = UU Triaxial

See Plate 2 for boring location.

PLATE A-2a

HVJ ASSOCIATES

LOG OF BORING

Project: Willowbrook WWTP Improvements
Project No.: HG1312760 WBS No.: R-000265-0104-3
Boring No.: B-2
Date: 7/18/2013
Elevation: 114.71 feet
Groundwater during drilling: ---
Northing: 13,911,389.6
Station: --
Groundwater after 24 hrs: ---
Easting: 3,066,757.4
Offset: --

ELEV. DEPTH, FEET	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	SOIL/ROCK CLASSIFICATION	% PASSING NO. 200 SIEVE	DRY DENSITY PCF
40		Very stiff to hard reddish brown and gray SANDY LEAN CLAY (CL)	49.5	
	24-23-25	Dense to very dense brown and gray SILTY SAND (SM)	36.5	
70 — 45	20-17-15			
	17-15-18		26.2	
	13-22-29		41.2	
65 — 50	25-26-29		48.8	
	17-28-36		47.9	
60 — 55	22-31-25		35.2	
55 — 60				
50 — 65				
45 — 70				
40 — 75				
35 — 80				

SHEAR STRENGTH, TSF: 0.5, 1.0, 1.5, 2.0

MOISTURE CONTENT, %: PLASTIC LIMIT — LIQUID LIMIT: 10, 20, 30, 40, 50, 60, 70, 80, 90

Shear Types: ● = Hand Penet. ■ = Torvane ▲ = Unconf. Comp. ✳ = UU Triaxial

See Plate 2 for boring location.

PLATE A-2b

HVJ ASSOCIATES

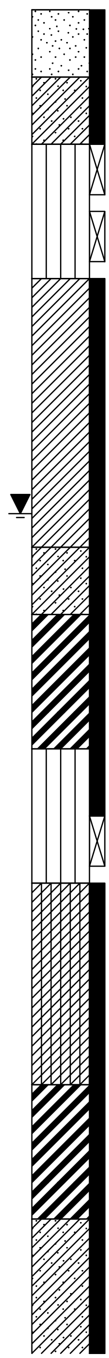
LOG OF BORING

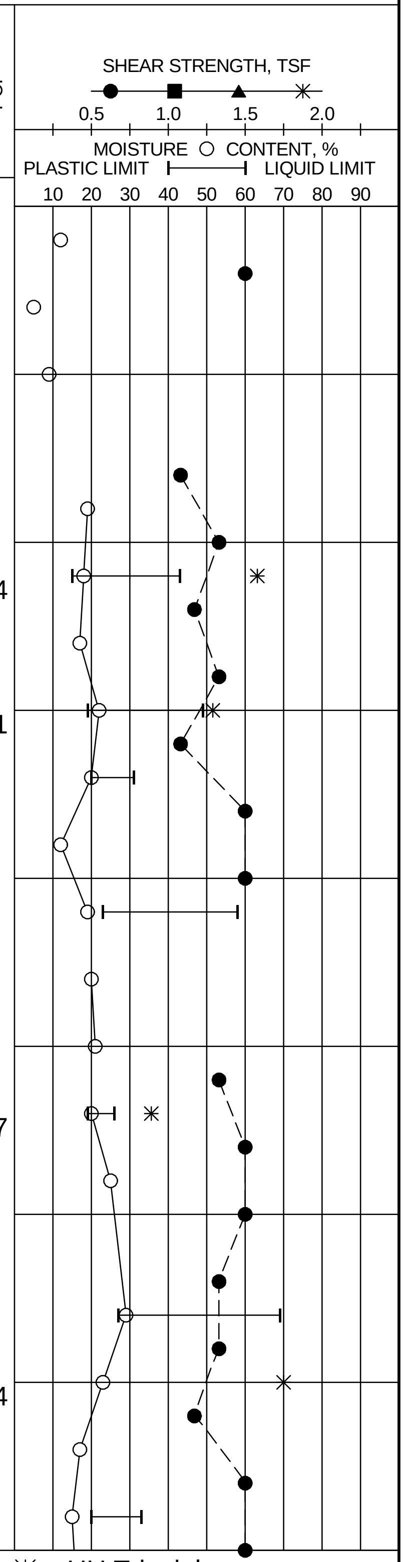
Project: Willowbrook WWTP Improvements
Boring No.: B-3
Groundwater during drilling: 15 feet
Groundwater after 24 hrs: ---

Project No.: HG1312760
Date: 7/18/2013
Northing: 13,911,349.4
Easting: 3,066,725.6

WBS No.: R-000265-0104-3
Elevation: 114.97 feet
Station: --
Offset: --

ELEV. DEPTH, FEET	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	SOIL/ROCK CLASSIFICATION	% PASSING NO. 200 SIEVE	DRY DENSITY PCF
0		Brown SAND (SP) -w/ roots		
2		Very stiff brown SANDY LEAN CLAY (CL)	66.6	
4	7-6-5	Medium dense brown SANDY SILT (ML)	69.3	
110 — 5	5-5-7			
8		Very stiff brown and gray LEAN CLAY (CL)		
105 — 10				114
		-w/ sand 12'-14'	76.1	
100 — 15				111
16		Very stiff brown and gray SANDY LEAN CLAY (CL)		
18		Very stiff reddish brown FAT CLAY WITH SAND (CH)	72.2	
95 — 20			83.8	
22		Medium dense reddish brown SANDY SILT (ML)	58.2	
90 — 25	9-12-17			
26		Stiff to very stiff reddish brown SILTY CLAY (CL-ML)		107
85 — 30			99.3	
32		Very stiff reddish brown FAT CLAY (CH)		
80 — 35				104
36		Very stiff to hard reddish brown SANDY LEAN CLAY (CL)	70.0	
75 — 40				





Shear Types: ● = Hand Penet. ■ = Torvane ▲ = Unconf. Comp. ✳ = UU Triaxial

See Plate 2 for boring location.

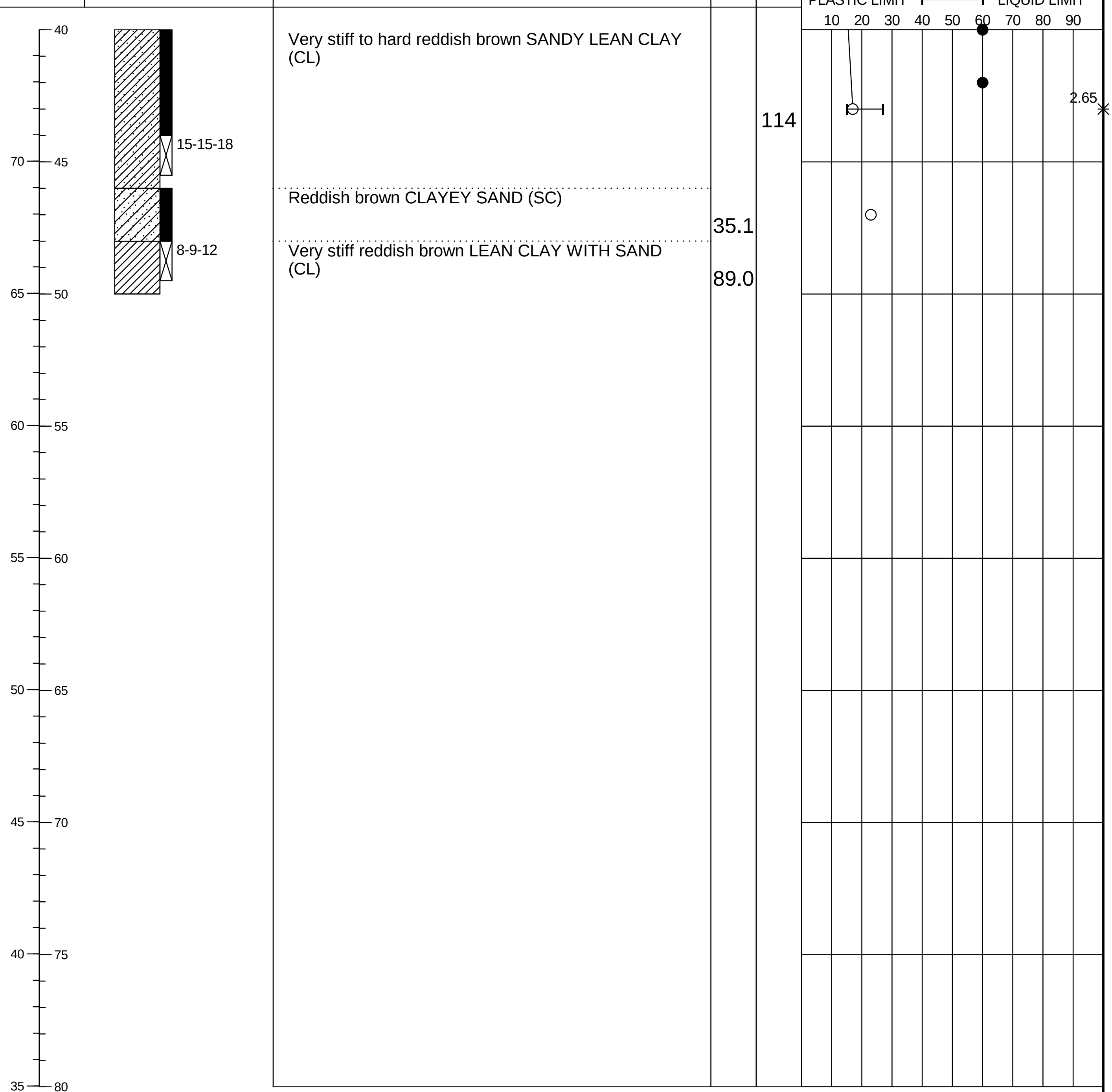
PLATE A-3a

HVJ ASSOCIATES

LOG OF BORING

Project: Willowbrook WWTP Improvements
Project No.: HG1312760 WBS No.: R-000265-0104-3
Boring No.: B-3
Date: 7/18/2013 Elevation: 114.97 feet
Groundwater during drilling: 15 feet
Northing: 13,911,349.4 Station: --
Groundwater after 24 hrs: ---
Easting: 3,066,725.6 Offset: --

ELEV. DEPTH, FEET	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	SOIL/ROCK CLASSIFICATION	% PASSING NO. 200 SIEVE	DRY DENSITY PCF	SHEAR STRENGTH, TSF / MOISTURE CONTENT, %
40 – 46	15-15-18	Very stiff to hard reddish brown SANDY LEAN CLAY (CL)		114	2.65
46 – 48		Reddish brown CLAYEY SAND (SC)	35.1		
48 – 50	8-9-12	Very stiff reddish brown LEAN CLAY WITH SAND (CL)	89.0		



Shear Types: ● = Hand Penet. ■ = Torvane ▲ = Unconf. Comp. ✳ = UU Triaxial

See Plate 2 for boring location.

PLATE A-3b

HVJ ASSOCIATES

LOG OF BORING

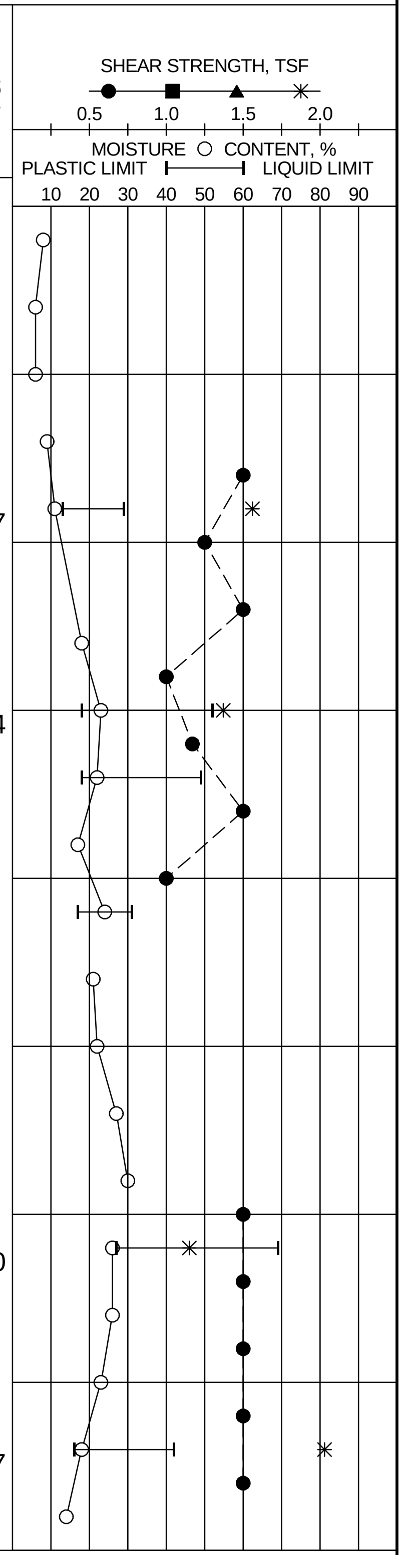
Project: Willowbrook WWTP Improvements
Boring No.: B-4
Groundwater during drilling: ---
Groundwater after 24 hrs: ---

Project No.: HG1312760
Date: 7/19/2013
Northing: 13,911,342.7
Easting: 3,066,789.4

WBS No.: R-000265-0104-3
Elevation: 114.71 feet
Station: --
Offset: --

ELEV. / DEPTH, FEET: 110 — 5; 105 — 10; 100 — 15; 95 — 20; 90 — 25; 85 — 30; 80 — 35; 75 — 40

SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	SOIL/ROCK CLASSIFICATION	% PASSING NO. 200 SIEVE	DRY DENSITY PCF
17-13-12 9-9-12	Medium dense brown SANDY SILT (ML)	68.4	
8-6-7	Stiff to very stiff reddish brown SANDY LEAN CLAY (CL)	68.7	117
	Very stiff reddish brown FAT CLAY (CH) -w/ sand 18'-20'	72.4	104
	Very stiff brown and gray SANDY LEAN CLAY (CL)		
5-12-17 10-10-15	Medium dense reddish brown SANDY SILT (ML)	65.8	
7-8-11 9-12-14	Medium dense reddish brown SILT WITH SAND (ML)	83.6	
	Very stiff brown and gray FAT CLAY (CH)		100
	Very stiff to hard reddish brown LEAN CLAY (CL)		107
	Very stiff reddish brown SANDY LEAN CLAY (CL)	65.7	



Shear Types: ● = Hand Penet. ■ = Torvane ▲ = Unconf. Comp. ✳ = UU Triaxial

See Plate 2 for boring location.

PLATE A-4a

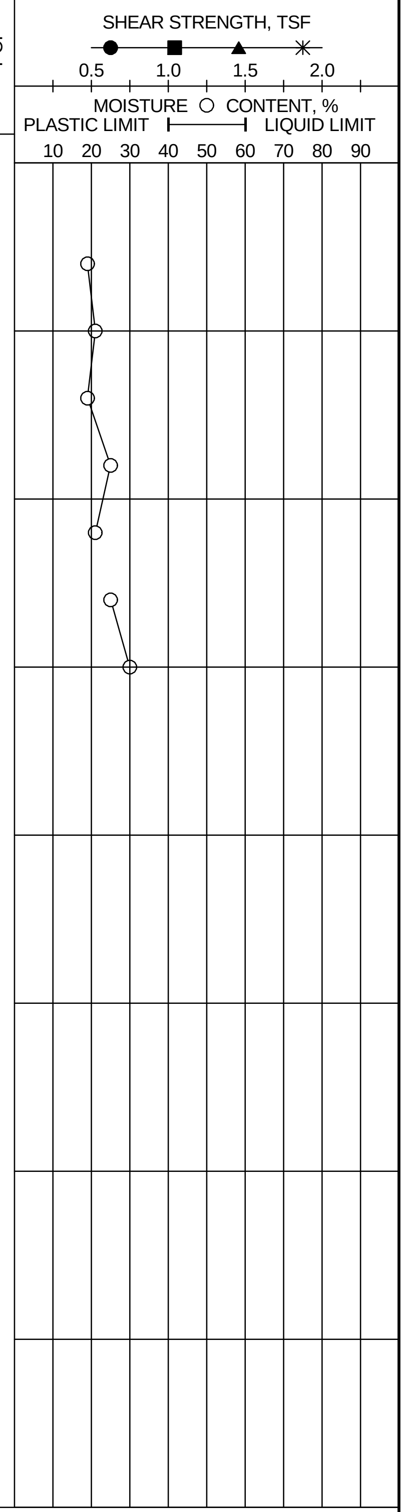
HVJ ASSOCIATES

LOG OF BORING

Project: Willowbrook WWTP Improvements | Project No.: HG1312760 | WBS No.: R-000265-0104-3
Boring No.: B-4 | Date: 7/19/2013 | Elevation: 114.71 feet
Groundwater during drilling: --- | Northing: 13,911,342.7 | Station: --
Groundwater after 24 hrs: --- | Easting: 3,066,789.4 | Offset: --

ELEV. DEPTH, FEET	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	SOIL/ROCK CLASSIFICATION	% PASSING NO. 200 SIEVE	DRY DENSITY PCF
40		Dense to very dense brown CLAYEY SAND (SC)	42.8	
	12-17-16			
70 — 45	13-14-17		27.6	
	15-18-19			
	16-18-20			
65 — 50	15-27-35		30.5	
			39.4	
	12-13-12	Very stiff reddish brown FAT CLAY (CH)		
60 — 55	9-12-14		97.6	

Elevation scale: 70, 65, 60, 55, 50, 45, 40, 35
Depth scale: 40, 45, 50, 55, 60, 65, 70, 75, 80



Shear Types: ● = Hand Penet. ■ = Torvane ▲ = Unconf. Comp. ✳ = UU Triaxial

See Plate 2 for boring location.

PLATE A-4b

HVJ ASSOCIATES

SOIL SYMBOLS

Soil Types

Clay | Silt | Sand | Gravel

Modifiers

Clayey | Silty | Sandy | Cemented

Construction Materials

Asphaltic Concrete | Stabilized Base | Fill or Debris | Portland Cement Concrete

SAMPLER TYPES

Thin Walled Shelby Tube

No Recovery

Split Barrel

Core

Liner Tube

Jar Sample

WATER LEVEL SYMBOLS

Groundwater level after drilling in open borehole or piezometer

Groundwater level determined during drilling operations

SOIL GRAIN SIZE

Classification	Particle Size	Particle Size or Sieve No. (U.S. Standard)
Clay	< 0.002 mm	< 0.002 mm
Silt	0.002 - 0.075 mm	0.002 mm - #200 sieve
Sand	0.075 - 4.75 mm	#200 sieve - #4 sieve
Gravel	4.75 - 75 mm	#4 sieve - 3 in.
Cobble	75 - 200 mm	3 in. - 8 in.
Boulder	> 200 mm	> 8 in.

DENSITY OF COHESIONLESS SOILS

Descriptive Term	Penetration Resistance "N" * Blows/Foot
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	> 50

CONSISTENCY OF COHESIVE SOILS

Consistency	Undrained Shear Strength (tsf)	Penetration Resistance "N" * Blows/Foot
Very Soft	0 - 0.125	0 - 2
Soft	0.125 - 0.25	2 - 4
Firm	0.25 - 0.5	4 - 8
Stiff	0.5 - 1.0	8 - 16
Very Stiff	1.0 - 2.0	16 - 32
Hard	> 2.0	> 32

PENETRATION RESISTANCE

3/6	Blows required to penetrate each of three consecutive 6-inch increments per ASTM D-1586 *
50/4"	If more than 50 blows are required, driving is discontinued and penetration at 50 blows is noted
0/18"	Sampler penetrated full depth under weight of drill rods and hammer

* The N value is taken as the blows required to penetrate the final 12 inches

TERMS DESCRIBING SOIL STRUCTURE

Slickensided	Fracture planes appear polished or glossy, sometimes striated
Fissured	Breaks along definite planes of fracture with little resistance to fracturing
Inclusion	Small pockets of different soils, such as small lenses of sand scattered through a mass of clay
Parting	Inclusion less than 1/4 inch thick extending through the sample
Seam	Inclusion 1/4 inch to 3 inches thick extending through the sample
Layer	Inclusion greater than 3 inches thick extending through the sample
Laminated	Soil sample composed of alternating partings of different soil type
Stratified	Soil sample composed of alternating seams or layers of different soil type
Intermixed	Soil sample composed of pockets of different soil type and laminated or stratified structure is not evident
Calcareous	Having appreciable quantities of calcium carbonate
Ferrous	Having appreciable quantities of iron
Nodule	A small mass of irregular shape

HVJ ASSOCIATES
6120 S. Dairy Ashford Road
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KEY TO TERMS AND SYMBOLS USED ON BORING LOGS

PROJECT NO.: HG 1312760

DRAWING NO.: PLATE A5

APPENDIX B

SUMMARY OF LABORATORY TEST RESULTS

Project: Willowbrook WWTP Improvements
Location: Houston, Texas
Number: HG1312760
WBS No.: R-000265-0104-3

Borehole	Depth, Feet	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	% Passing #200 Sieve	Moisture Content (%)	Total Unit Weight (pcf)	Shear Strength (UU) (tsf)	Shear Strength (Pocket Pen) (tsf)
B-1	1				72	8.3			
B-1	3				74				
B-1	5				75	6.5			
B-1	6								1.5
B-1	7	25	15	10		12.8	131.4	1.78	
B-1	8								1.42
B-1	9				83	14.1			
B-1	10								1.33
B-1	11	39	16	23		17.5			
B-1	12								1.5
B-1	13					19.6	130.4	2	
B-1	14								0.58
B-1	14.5	55	20	35		13.3			
B-2	1					4.7			
B-2	3				70	5.7			
B-2	7				75				
B-2	8								1
B-2	9	30	14	16		18.8	133	1.35	
B-2	10								0.92
B-2	11	54	18	36		19.8			
B-2	12								0.67
B-2	13					21	127.9	1.25	
B-2	14								1.42
B-2	15				99	16.8			
B-2	16								1.5
B-2	17	30	19	11					

Project: Willowbrook WWTP Improvements
Location: Houston, Texas
Number: HG1312760
WBS No.: R-000265-0104-3

Borehole	Depth, Feet	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	% Passing #200 Sieve	Moisture Content (%)	Total Unit Weight (pcf)	Shear Strength (UU) (tsf)	Shear Strength (Pocket Pen) (tsf)
B-2	18								1.5
B-2	19	30	18	12		19.4	132.5	1.23	
B-2	20								0.75
B-2	21	30	20	10	71	19.6			
B-2	23					17.4			
B-2	25				76	21			
B-2	27				86	27.2			
B-2	28								1.5
B-2	29	63	25	38		28.5	125.3	1.42	
B-2	30								1.33
B-2	31					25.4			
B-2	32								1.5
B-2	33	64	26	38	94	25.5			
B-2	34								1.42
B-2	35					28.9			
B-2	36								1.5
B-2	37	28	16	12		15.6	138.2	2.73	
B-2	38								1.5
B-2	40								1.17
B-2	41				50	19.1			
B-2	43				37	21.3			
B-2	45					20.6			
B-2	47				26	21.5			
B-2	49				41	22.8			
B-2	51				49	22.3			
B-2	53				48	19.6			

Project: Willowbrook WWTP Improvements
Location: Houston, Texas
Number: HG1312760
WBS No.: R-000265-0104-3

Borehole	Depth, Feet	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	% Passing #200 Sieve	Moisture Content (%)	Total Unit Weight (pcf)	Shear Strength (UU) (tsf)	Shear Strength (Pocket Pen) (tsf)
B-2	55				35	23.3			
B-3	1					11.8			
B-3	2								1.5
B-3	3				67	4.9			
B-3	5				69	9.3			
B-3	8								1.08
B-3	9					19			
B-3	10								1.33
B-3	11	43	15	28		18	134	1.58	
B-3	12								1.17
B-3	13				76	16.7			
B-3	14								1.33
B-3	15	49	19	30		22.3	136	1.29	
B-3	16								1.08
B-3	17	31	20	11		19.9			
B-3	18								1.5
B-3	19				72	11.8			
B-3	20								1.5
B-3	21	58	23	35	84	19			
B-3	23				58	20.2			
B-3	25					20.9			
B-3	26								1.33
B-3	27	26	19	7		20.1	128.2	0.89	
B-3	28								1.5
B-3	29					24.8			
B-3	30								1.5

Project: Willowbrook WWTP Improvements
Location: Houston, Texas
Number: HG1312760
WBS No.: R-000265-0104-3

Borehole	Depth, Feet	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	% Passing #200 Sieve	Moisture Content (%)	Total Unit Weight (pcf)	Shear Strength (UU) (tsf)	Shear Strength (Pocket Pen) (tsf)
B-3	31				99				
B-3	32								1.33
B-3	33	69	27	42		28.8			
B-3	34								1.33
B-3	35					22.8	127.8	1.75	
B-3	36								1.17
B-3	37				70	16.8			
B-3	38								1.5
B-3	39	33	20	13		14.7			
B-3	40								1.5
B-3	42								1.5
B-3	43	27	15	12		17.3	133.3	2.65	
B-3	47				35	22.5			
B-3	49				89				
B-4	1					8.2			
B-4	3				68	6.1			
B-4	5					5.6			
B-4	7				69	8.9			
B-4	8								1.5
B-4	9	29	13	16		11	129.4	1.56	
B-4	10								1.25
B-4	12								1.5
B-4	13					17.7			
B-4	14								1
B-4	15	52	18	34		23.1	127.9	1.37	
B-4	16								1.17

Project: Willowbrook WWTP Improvements
Location: Houston, Texas
Number: HG1312760
WBS No.: R-000265-0104-3

Borehole	Depth, Feet	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	% Passing #200 Sieve	Moisture Content (%)	Total Unit Weight (pcf)	Shear Strength (UU) (tsf)	Shear Strength (Pocket Pen) (tsf)
B-4	17	49	18	31		22			
B-4	18								1.5
B-4	19				72	17.4			
B-4	20								1
B-4	21	31	17	14		23.8			
B-4	23				66	21.1			
B-4	25					21.9			
B-4	27				84	26.7			
B-4	29					30			
B-4	30								1.5
B-4	31	69	27	42		25.9	125.7	1.15	
B-4	32								1.5
B-4	33					25.7			
B-4	34								1.5
B-4	35					22.5			
B-4	36								1.5
B-4	37	42	16	26		18.3	127.2	2.03	
B-4	38								1.5
B-4	39				66	13.8			
B-4	41				43				
B-4	43					19.4			
B-4	45				28	20.6			
B-4	47					18.6			
B-4	49				31	25			
B-4	51				39	21.2			
B-4	53					25.4			

Project: Willowbrook WWTP Improvements
Location: Houston, Texas
Number: HG1312760
WBS No.: R-000265-0104-3

Borehole	Depth, Feet	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	% Passing #200 Sieve	Moisture Content (%)	Total Unit Weight (pcf)	Shear Strength (UU) (tsf)	Shear Strength (Pocket Pen) (tsf)
B-4	55				98	29.7			
Total		**25**	**25**	**25**	**39**	**77**	**16**	**16**	**48**

APPENDIX C

SWELL TEST RESULTS

HVJ ASSOCIATES, INC.
SWELL TEST RESULTS

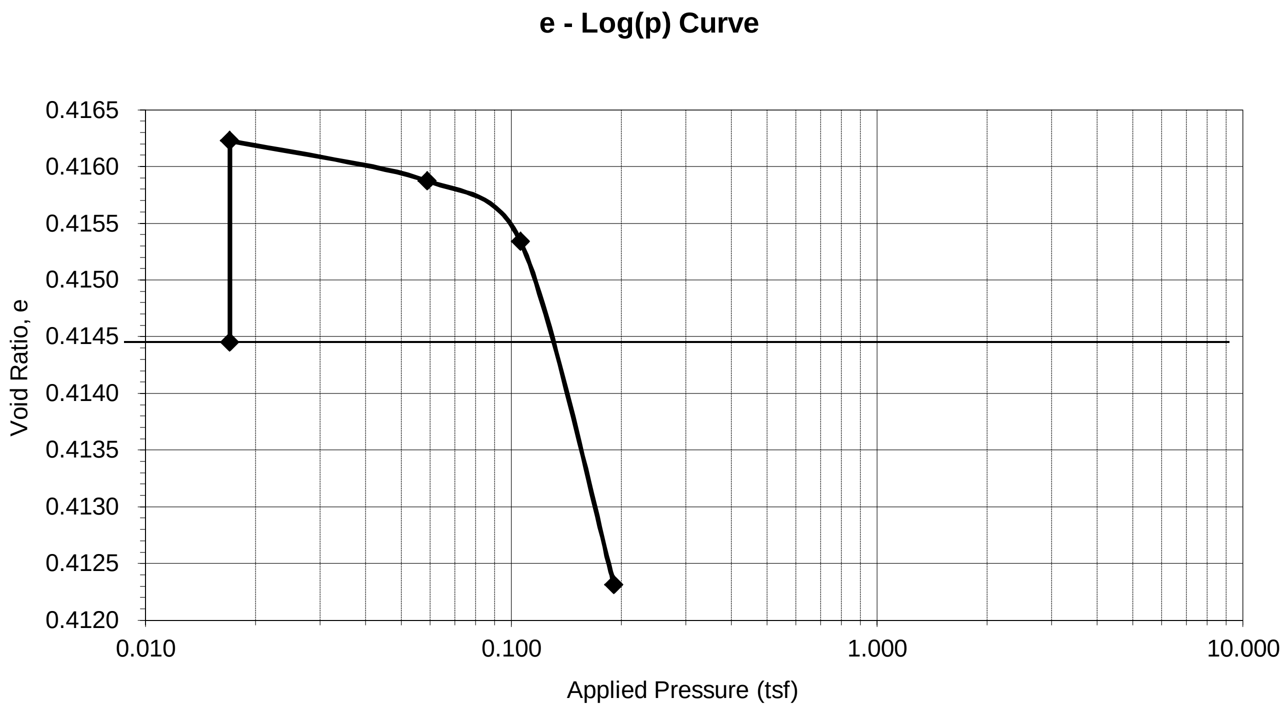
Project Name:	Willowbrook WWTP	Boring No.	1
Project No.	HG-13-12760	Sample No.	4
Date Tested:	7/26/2013 - 7/30/2013	Sample Depth	6-8
Technician:	KC	Date Calculated:	8/3/2013

Sample Data	**Initial**	**Final**	**Test Data**	**Initial**	**Final**
Sample Height (in)	0.795	0.794	Wet + Ring (g)	277.750	281.440
Diameter (in)	2.459	2.459	Dry + Ring (g)	263.410	263.410
Volume (cc)	61.869	61.776	Ring Wt. (g)	149.640	149.640
Height of Solids (in)	0.562	0.562	**Moisture Data (Trimmings)**		**LL**
Specific Gravity (assumed)	2.601	2.601	Wet + Tare (g)	164.400	
Moisture Content (%)	12.604	15.848	Dry + Tare (g)	149.360	**PI**
Wet Density (pcf)	129.209	133.132	Tare (g)	30.020	
Dry Density (pcf)	114.746	114.919	Moisture Content (%)	12.603	
Void Ratio	0.414	0.412	**Sample Description**	light brown sandy silt with clay and ferrous nodules	
Percent Saturation	79.1	100.0			

Applied Press. (tsf)	Calibr. Rdg. (in.)	Def.Rdg (in.)	Corr. Cum Reading (in.)	Strain (%)	Void Ratio Change	Void Ratio
0.017	0.0000	0.0000	0.0000	0.00	0.00	0.4144
0.017	0.0000	-0.0010	-0.0010	-0.13	0.00	0.4162
0.059	0.0001	-0.0007	-0.0008	-0.10	0.00	0.4159
0.106	0.0006	0.0001	-0.0005	-0.06	0.00	0.4153
0.191	0.0015	0.0027	0.0012	0.15	0.00	0.4123

HVJ ASSOCIATES, INC.
SWELL TEST RESULTS

Project Name:	Willowbrokk WWTP	Boring No.	1
Project No.	HG-13-12760	Sample No.	4
		Sample Depth	6-8



Swell Pressure (tsf) = 0.15
% Swell = 0.13

HVJ ASSOCIATES, INC.
SWELL TEST RESULTS

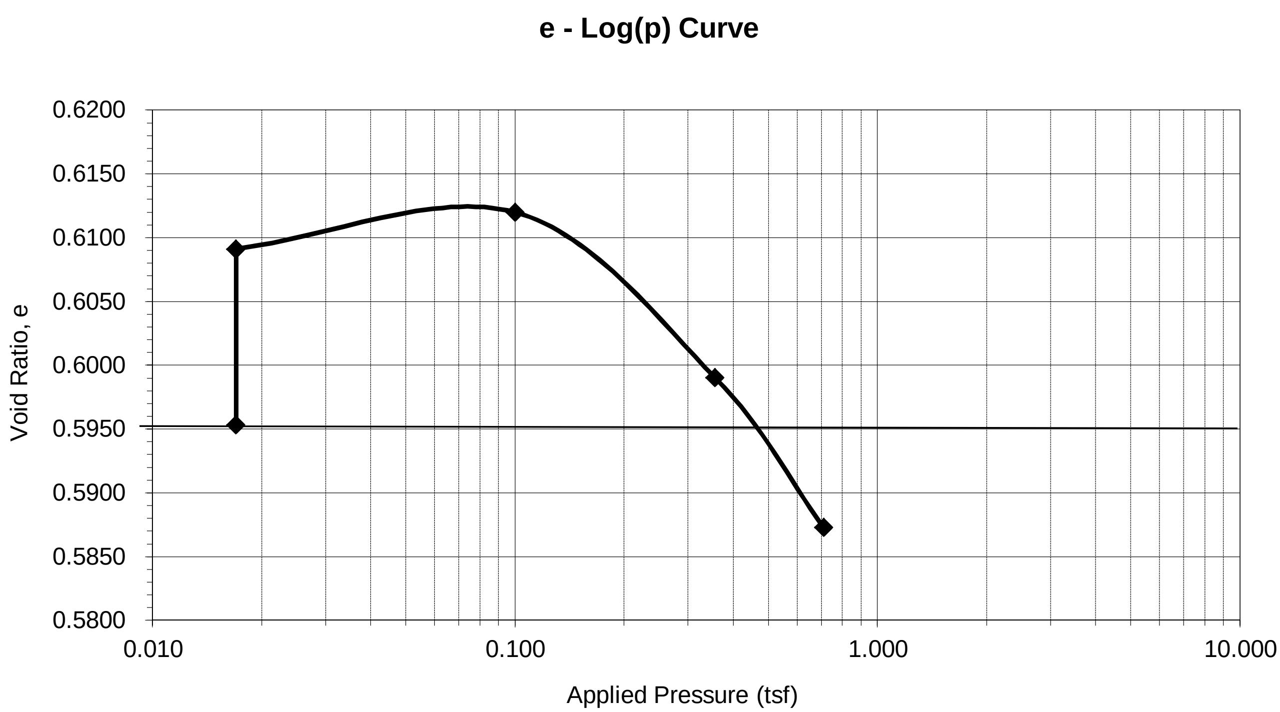
Project Name:	Willowbrook WWTP	Boring No.	2
Project No.	HG-13-12760	Sample No.	6
Date Tested:	7/31/2013 - 8/8/2013	Sample Depth	10-12'
Technician:	KC	Date Calculated:	8/9/2013

Sample Data	**Initial**	**Final**	**Test Data**	**Initial**	**Final**
Sample Height (in)	0.775	0.775	Wet + Ring (g)	277.150	279.610
Diameter (in)	2.500	2.500	Dry + Ring (g)	256.340	256.340
Volume (cc)	62.341	62.341	Ring Wt. (g)	149.620	149.620
Height of Solids (in)	0.486	0.486	**Moisture Data (Trimmings)**		**LL**
Specific Gravity	2.731	2.731	Wet + Tare (g)	128.210	
Moisture Content (%)	19.500	21.805	Dry + Tare (g)	112.320	**PI**
Wet Density (pcf)	127.651	130.113	Tare (g)	30.830	
Dry Density (pcf)	106.821	106.821	Moisture Content (%)	19.499	
Void Ratio	0.595	0.595	**Sample Description**	light grey, brownish yellow sandy clay with ferrous stains	
Percent Saturation	89.5	100.0			

Applied Press. (tsf)	**Calibr. Rdg. (in.)**	**Def.Rdg (in.)**	**Corr. Cum Reading (in.)**	**Strain (%)**	**Void Ratio Change**	**Void Ratio**
0.017	0.0002	0.0000	0.0000	0.00	0.00	0.5953
0.017	0.0002	-0.0065	-0.0067	-0.86	-0.01	0.6091
0.100	0.0024	-0.0057	-0.0081	-1.05	-0.02	0.6120
0.356	0.0064	0.0046	-0.0018	-0.23	0.00	0.5990
0.710	0.0088	0.0127	0.0039	0.50	0.01	0.5873

HVJ ASSOCIATES, INC.
SWELL TEST RESULTS

Project Name:	Willowbrook WWTP	Boring No.	2
Project No.	HG-13-12760	Sample No.	6
		Sample Depth	10-12'



Swell Pressure (tsf) = 0.45

% Swell = 1.64

APPENDIX D

CONSOLIDATION TEST RESULTS

HVJ ASSOCIATES

HVJ ASSOCIATES, INC.
CONSOLIDATION TEST RESULTS
ASTM D-2435

Project Name:	Willowbrook WWTP	**Boring No.**	3
Project No.	HG-13-12760	**Sample No.**	9
Date Tested:	8/2/2013 - 8/29/2013	**Sample Depth**	16-18
Technician:	KC	**Date Calculated:**	8/29/2013

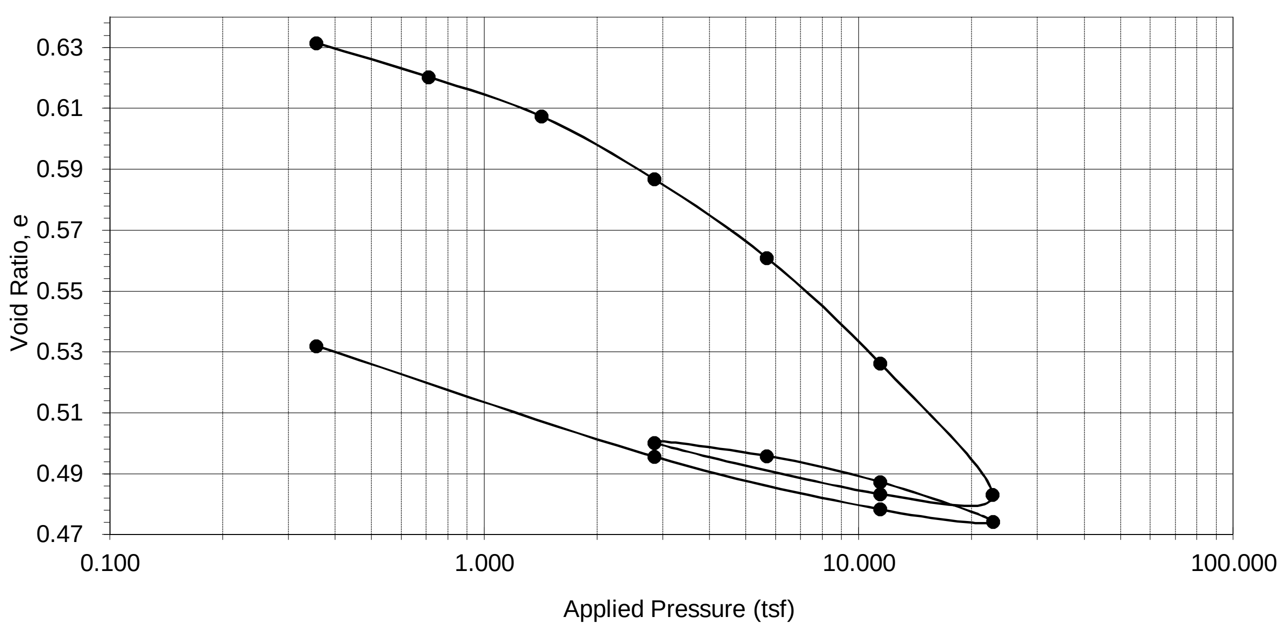
Sample Data	Initial	Final	Test Data	Initial	Final
Sample Height (in)	0.750	0.699	Wet + Ring (g)	184.510	182.140
Diameter (in)	2.500	2.500	Dry + Ring (g)	162.610	162.610
Volume (cc)	60.330	56.227	Ring Wt. (g)	60.580	60.580
Height of Solids (in)	0.456	0.456	Moisture Data (Trimmings)		LL
Specific Gravity	2.780	2.780	Wet + Tare (g)	143.660	31
Moisture Content (%)	21.464	19.141	Dry + Tare (g)	123.620	PI
Wet Density (pcf)	128.183	134.905	Tare (g)	30.260	11
Dry Density (pcf)	105.531	113.231	Moisture Content (%)	21.465	
Void Ratio	0.644	0.532	Sample Description	light grey, red silty clay	
Percent Saturation	92.7	100.0			

Applied Press. (tsf)	Calibr. Rdg. (in.)	Def. Rdg. (in.)	Corr. Cum Reading (in.)	Strain (%)	Void Ratio Change	Void Ratio	t_{50} (min.)	C_v (in^2/day)
0.356	0.0031	0.0087	0.0056	0.75	0.01	0.63	0.35	
0.710	0.0044	0.0151	0.0107	1.43	0.02	0.62	2.00	
1.419	0.0066	0.0232	0.0166	2.21	0.04	0.61	2.50	
2.839	0.0098	0.0358	0.0260	3.47	0.06	0.59	2.50	14.87
5.677	0.0134	0.0512	0.0378	5.04	0.08	0.56	2.50	14.39
11.387	0.0170	0.0706	0.0536	7.15	0.12	0.53	3.00	11.46
22.741	0.0208	0.0941	0.0733	9.77	0.16	0.48	4.00	8.12
11.387	0.0189	0.0921	0.0732	9.76	0.16	0.48	2.00	16.24
2.839	0.0154	0.0809	0.0655	8.73	0.14	0.50	4.50	7.38
5.677	0.0166	0.0841	0.0675	9.00	0.15	0.50	1.70	19.43
11.387	0.0185	0.0899	0.0714	9.52	0.16	0.49	1.20	27.21
22.839	0.0209	0.0983	0.0774	10.32	0.17	0.47	3.00	10.69
11.387	0.0191	0.0946	0.0755	10.07	0.17	0.48	1.00	32.26
2.839	0.0156	0.0832	0.0676	9.01	0.15	0.50	5.00	6.60
0.356	0.0111	0.0621	0.0510	6.80	0.11	0.53	19.00	1.82

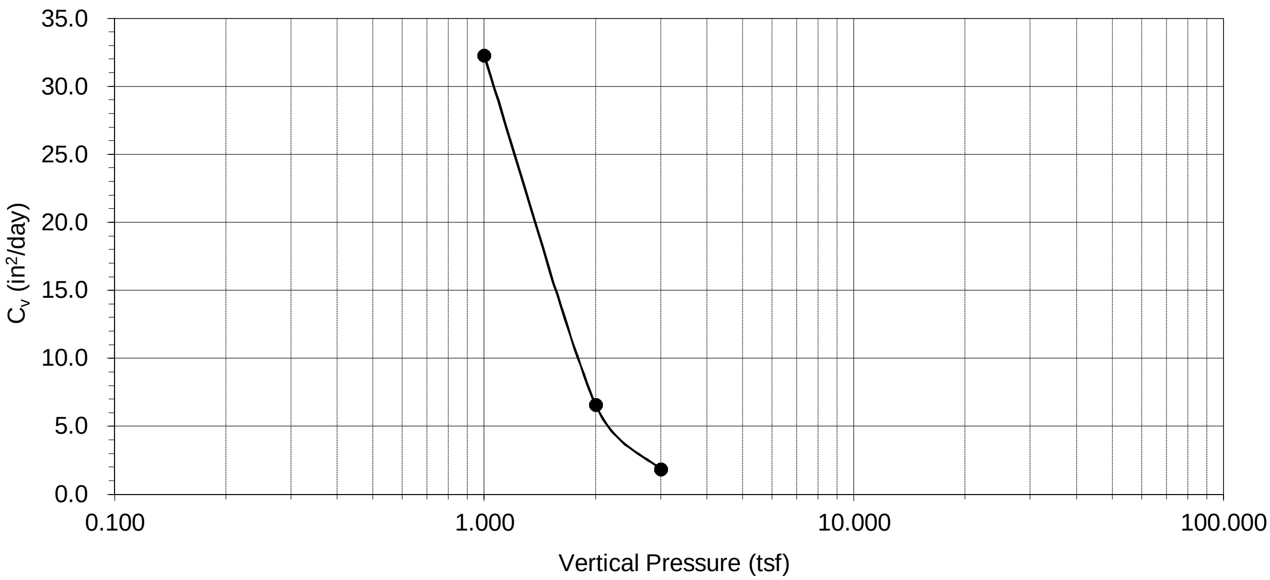
HVJ ASSOCIATES, INC.
CONSOLIDATION TEST RESULTS

Project Name:	Willowbrook WWTP	Boring No.	3
Project No.	HG-13-12760	Sample No.	9
		Sample Depth	16-18

e - Log(p) Curve



Cv - Log(p) Curve



APPENDIX E

SULFATE, SULFIDE, CHLORIDE AND PH TEST RESULTS

LABORATORY TEST RESULTS

Job ID : 13071138

Date 8/1/2013

Client Name:	HVJ Associates		Attn: Anil Raavi
Project Name:	HG-13-12760 / Willowbrook WWTP		

Client Sample ID:	B2, S-3, 4-6'	Job Sample ID:	13071138.01
Date Collected:	07/23/13	Sample Matrix	Soil
Time Collected:	11:45		
Other Information:			

Test Method	Parameter/Test Description	Result	Units	DF	Rpt Limit	Reg Limit	Q	Date Time	Analyst
EPA 300.0	Water Soluble Anions								
	Chloride	18.5	mg/Kg	1	1			07/30/13 14:15	XA
	Sulfate	29.5	mg/Kg	1	1			07/30/13 14:15	XA
SM 4500SO3-Bmod									
	Sulfite[1]	BRL	mg/Kg	1	20		H3	07/30/13 11:25	KS
SW-846 9045D	Corrosivity, pH								
	pH	8.33	s.u.	1			H3	07/26/13 10:35	SR
	Temperature when read, °C[1]	24.4	s.u.	1			H3	07/26/13 10:35	SR

HVJ ASSOCIATES
6120 S. Dairy Ashford Road
Houston, Texas 77072-1010
281.933.7388 Ph
281.933.7293 Fax

DATE: 8/16/2013	APPROVED BY: ZA	PREPARED BY: SS

SULFATE, SULFIDE, CHLORIDE AND PH TEST RESULTS
WBS No. R-000265-0104-3

PROJECT NO.: HG1312760	DRAWING NO.: PLATE E-1

LABORATORY TEST RESULTS

Job ID : 13071138

Date 8/1/2013

Client Name:	HVJ Associates	Attn: Anil Raavi	
Project Name:	HG-13-12760 / Willowbrook WWTP		

Client Sample ID:	B3, S-4, 6-8'	Job Sample ID:	13071138.02
Date Collected:	07/23/13	Sample Matrix	Soil
Time Collected:	11:45		
Other Information:			

Test Method	Parameter/Test Description	Result	Units	DF	Rpt Limit	Reg Limit	Q	Date Time	Analyst
EPA 300.0	Water Soluble Anions								
	Chloride	4.66	mg/Kg	1	1			07/25/13 12:59	XA
	Sulfate	12.5	mg/Kg	1	1			07/25/13 12:59	XA
SM 4500SO3-Bmod									
	Sulfite[1]	BRL	mg/Kg	1	20		H3	07/30/13 11:25	KS
SW-846 9045D	Corrosivity, pH								
	pH	8.01	s.u.	1			H3	07/26/13 10:35	SR
	Temperature when read, °C[1]	24.9	s.u.	1			H3	07/26/13 10:35	SR

HVJ ASSOCIATES
6120 S. Dairy Ashford Road
Houston, Texas 77072-1010
281.933.7388 Ph
281.933.7293 Fax

DATE: 8/16/2013	APPROVED BY: ZA	PREPARED BY: SS

SULFATE, SULFIDE, CHLORIDE AND PH TEST RESULTS
WBS No. R-000265-0104-3

PROJECT NO.: HG1312760	DRAWING NO.: PLATE E-2

LABORATORY TEST RESULTS

Job ID : 13071138

Date 8/1/2013

Client Name:	HVJ Associates		Attn: Anil Raavi
Project Name:	HG-13-12760 / Willowbrook WWTP		

Client Sample ID:	B4, S-6, 10-12'	Job Sample ID:	13071138.03
Date Collected:	07/23/13	Sample Matrix	Soil
Time Collected:	11:45		
Other Information:			

Test Method	Parameter/Test Description	Result	Units	DF	Rpt Limit	Reg Limit	Q	Date Time	Analyst
EPA 300.0	Water Soluble Anions								
	Chloride	6.54	mg/Kg	1	1			07/25/13 13:17	XA
	Sulfate	4.99	mg/Kg	1	1			07/25/13 13:17	XA
SM 4500SO3-Bmod									
	Sulfite[1]	BRL	mg/Kg	1	20		H3	07/30/13 11:25	KS
SW-846 9045D	Corrosivity, pH								
	pH	7.31	s.u.	1			H3	07/26/13 10:35	SR
	Temperature when read, °C[1]	25.1	s.u.	1			H3	07/26/13 10:35	SR

HVJ ASSOCIATES
6120 S. Dairy Ashford Road
Houston, Texas 77072-1010
281.933.7388 Ph
281.933.7293 Fax

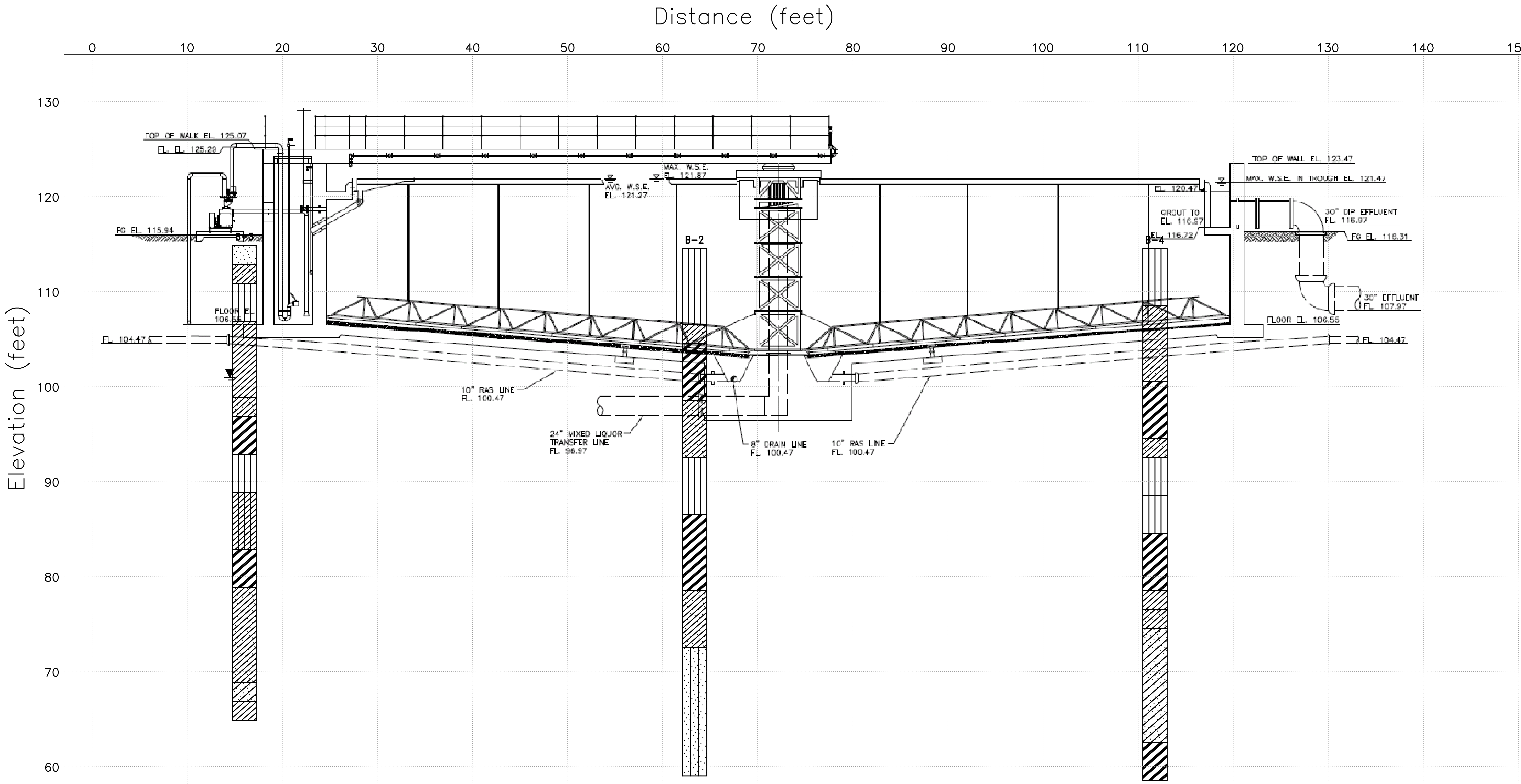
DATE: 8/16/2013	APPROVED BY: ZA	PREPARED BY: SS

SULFATE, SULFIDE, CHLORIDE AND PH TEST RESULTS
WBS No. R-000265-0104-3

PROJECT NO.: HG1312760	DRAWING NO.: PLATE E-3

APPENDIX F

SOIL PROFILE SHOWING PROPOSED CLARIFIER



LEGEND:

- Fat Clay (CH)
- Sandy Clay (CL)
- Silt (ML)
- Clayey Sand (SC)
- Silty Sand (SM)
- Groundwater during drilling.

HVJ ASSOCIATES
6120 S. Dairy Ashford Road
HOUSTON, TX 77072-1010
PH: 281.933.7388 FAX: 281.933.7293
TEXAS FIRM NO. F-000646

SCALE: 1" = 10'	
DATE: 6/23/2014	
DRAWN BY: ZL	PROJ. CHK: ND

SOIL PROFILE
WILLOWBROOK WWTP IMPROVEMENTS
WBS No. R-000265-0104-3

PROJECT NO.: HG1312760	DRAWING NO.: PLATE F-1

APPENDIX G

ODOR CONTROL AND RAS PUMP STATION
SECTIONS

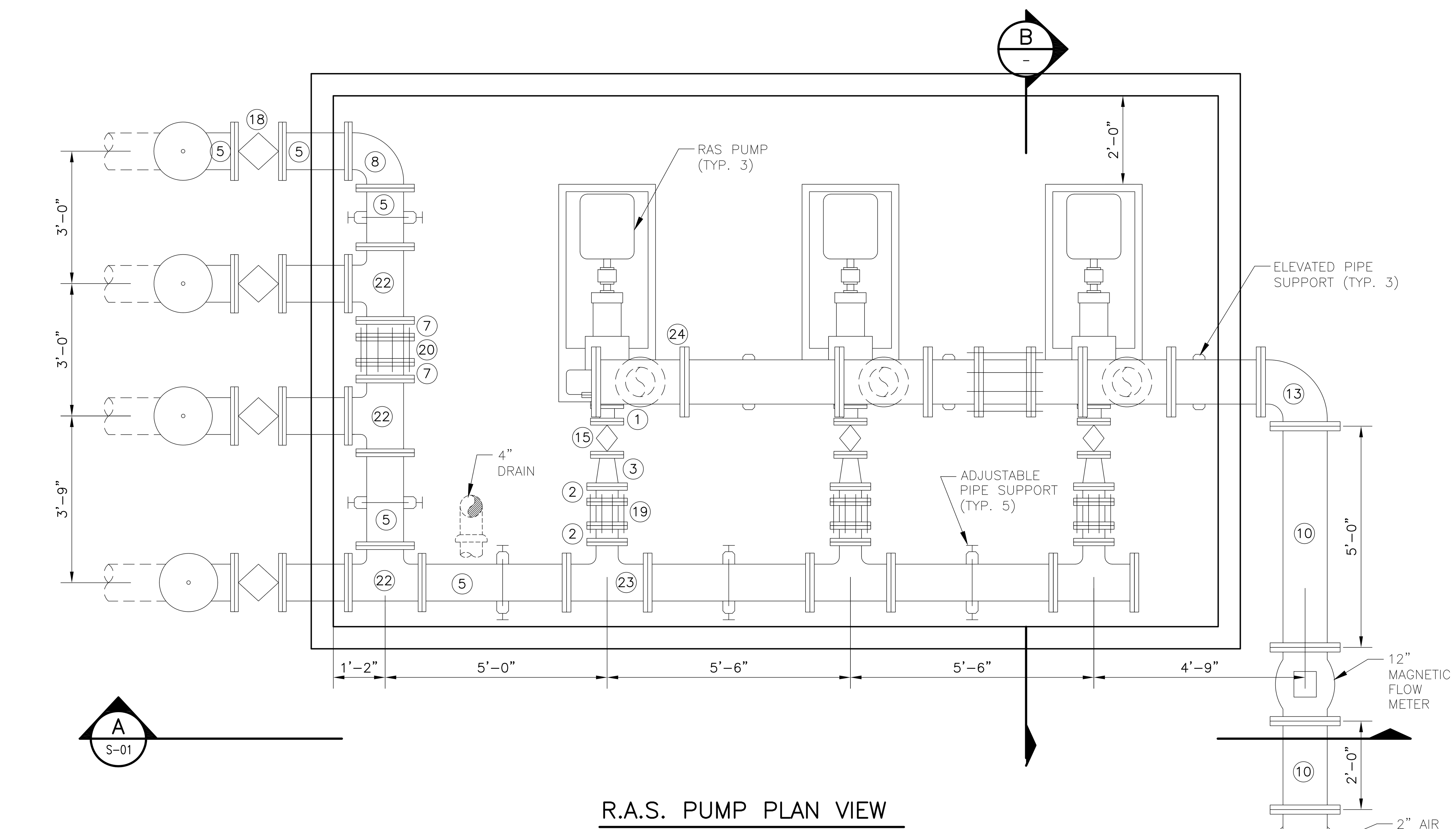
NOTES:

1. DIMENSIONS NOTED ARE RELATIVE TO THE PUMP SIZE AND MANUFACTURER SELECTED. CONTRACTOR SHALL CONFIRM.
2. PUMP MOTORS SHALL BE NON-OVERLOADING OVER ENTIRE PUMPING RANGE INCLUDING ABILITY TO PUMP INTO FORCE MAINS UNDER A FLOODED WET WELL CONDITION.
3. REFER TO ELECTRICAL DRAWINGS FOR PRESSURE TRANSDUCER AND LEVEL CONTROL INSTRUMENTATION DETAILS.
4. REFER TO STRUCTURAL DRAWINGS FOR PAD DETAILS.
5. 100-YEAR FLOOD ELEVATION IS 116.50 FEET. 500-YEAR FLOOD ELEVATION IS 118.00 FEET.

PIPING LIST

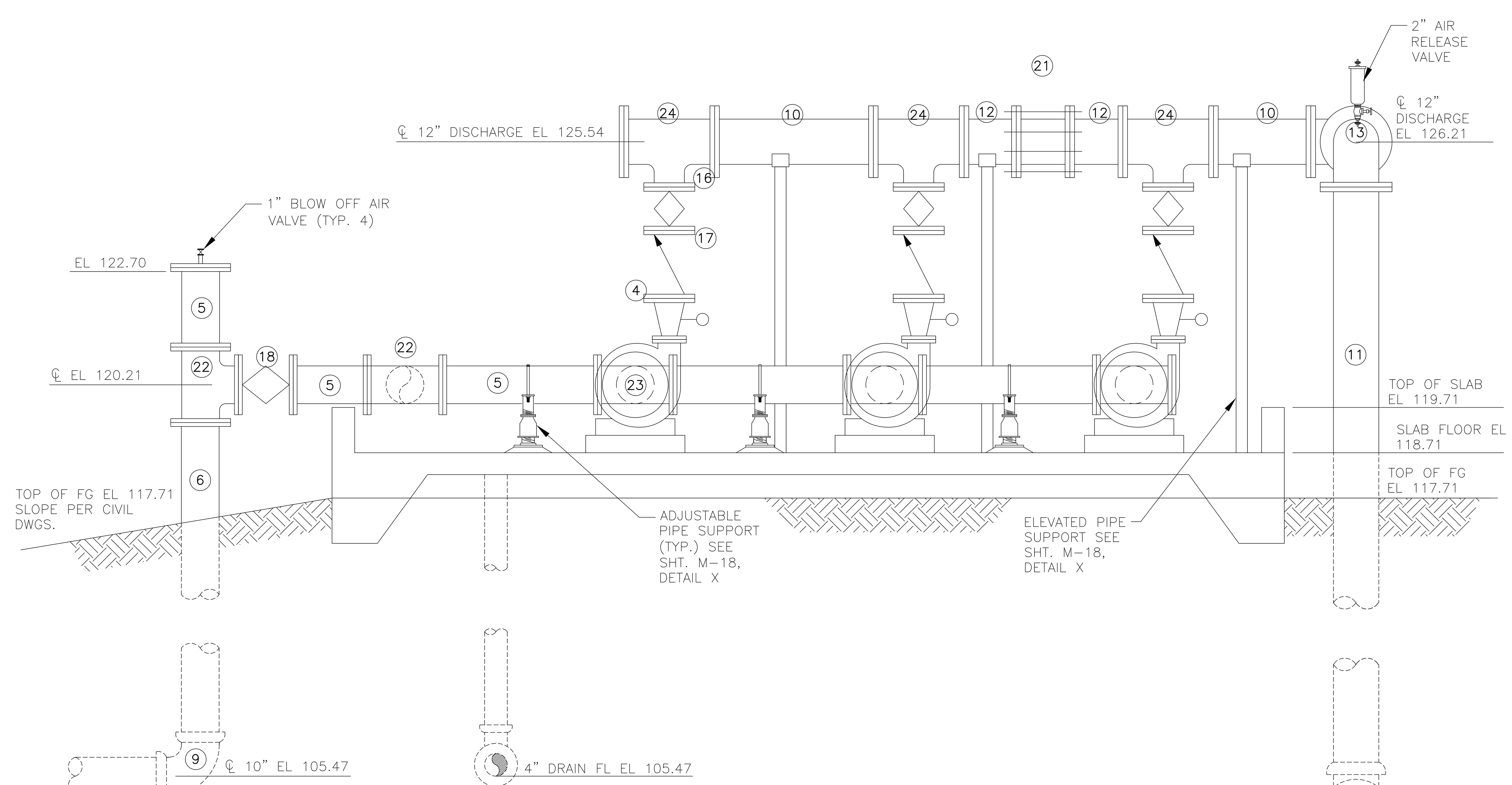
NO	QTY	FITTING DESCRIPTION	NO	QTY	PIPING DESCRIPTION
1	3	4" SPOOL PIECE, FJ X FJ	13	2	12" 90° BEND, FJ X FJ
2	6	6" SPOOL PIECE, FJ X PE	14	1	12" 90° BEND, MJ X MJ
3	3	6" TO 4" REDUCER, FJ X FJ	15	3	4" PLUG VALVE, FJ X FJ
4	3	8" TO 4" REDUCER, FJ X FJ	16	3	8" PLUG VALVE, FJ X FJ
5	13	10" *SPOOL PIECE, FJ X FJ	17	3	8" CHECK VALVE, FJ X FJ
6	4	10" SPOOL PIECE, MJ X FJ	18	4	10" PLUG VALVE, FJ X FJ
7	2	10" SPOOL PIECE, FJ X PE	19	3	6" DRESSER COUPLING, RESTRAINED
8	1	10" 90° BEND, FJ X FJ	20	1	10" DRESSER COUPLING, RESTRAINED
9	4	10" 90° BEND, MJ X MJ	21	1	12" DRESSER COUPLING, RESTRAINED
10	4	12" *SPOOL PIECE, FJ X FJ	22	7	10" TEE, FJ X FJ
11	1	12" SPOOL PIECE, FJ X MJ	23	3	10" X 6" TEE, FJ X FJ
12	2	12" SPOOL PIECE, FJ X PE	24	3	12" X 8" TEE, FJ X FJ

*LENGTHS OF SPOOL PIECES WILL VARY.



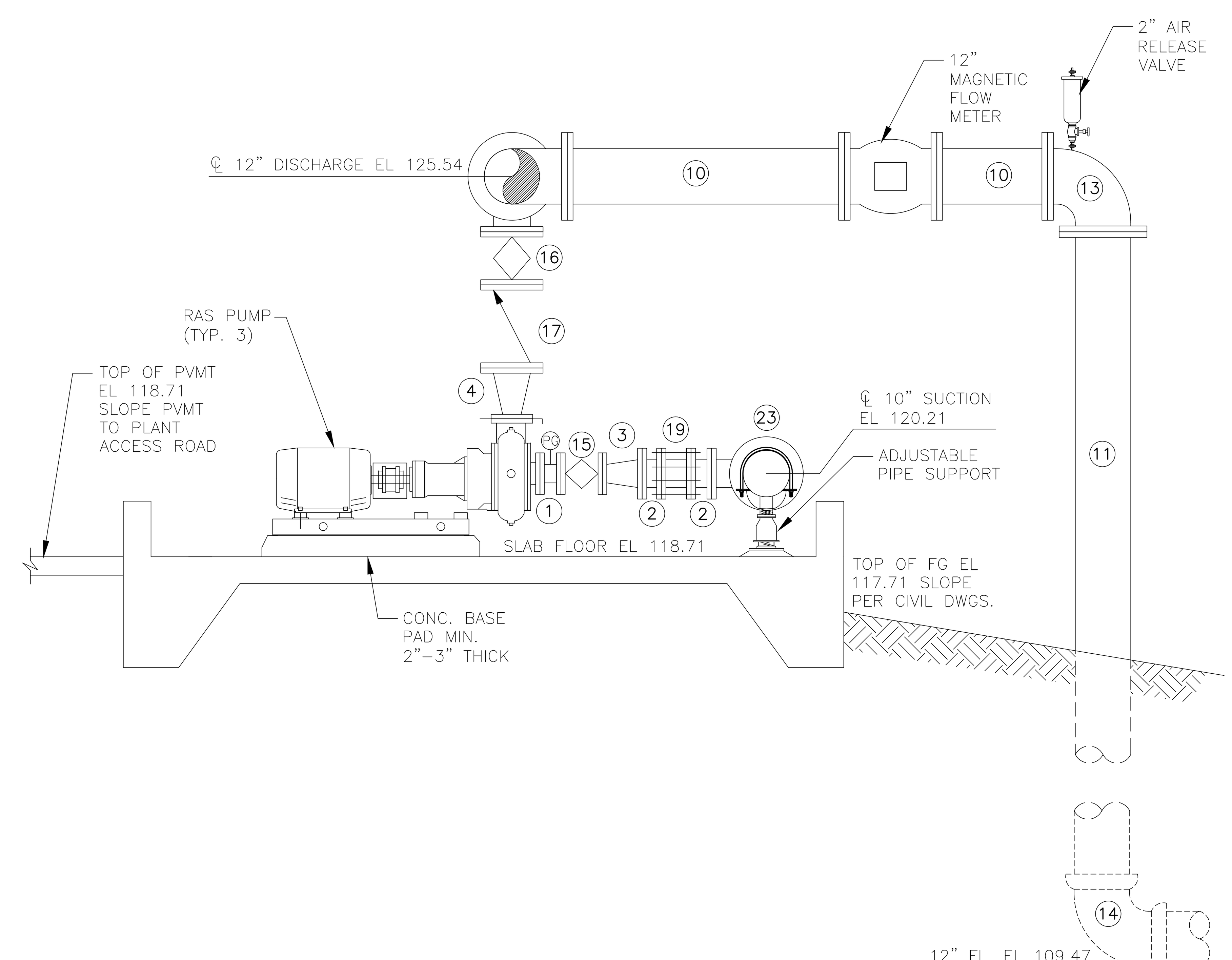
R.A.S. PUMP PLAN VIEW

SCALE: 1/2" = 1'-0"



R.A.S. PUMP SECTION (A)

SCALE: 1/2" = 1'-0"



R.A.S. PUMP SECTION (B)

SCALE: 1/2" = 1'-0"

60% SUBMITTAL

PRELIMINARY

NOT FOR CONSTRUCTION

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PRIVATE UTILITY LINES SHOWN

N/A
RELIANT ENERGY ENTEX, INC.

N/A
S.B.C. VALID FOR ONE YEAR
APPROVED FOR UNDERGROUND CONDUIT FACILITIES UNLESS NOTED

N/A
RELIANT ENERGY
HOUSTON LIGHTING & POWER CO.
APPROVED ONLY FOR CROSSING UNDERGROUND DUCTLINES UNLESS NOTED. VALID AT TIME OF REVIEW ONLY.

N/A
CABLE COMPANY

WESTON SOLUTIONS
5599 SAN FELIPE, SUITE 700
HOUSTON, TEXAS 77056
713-985-6600
TBPE REGISTRATION NO. F-3123

SURVEYED BY: LANDTECH
FB NO. -

CITY OF HOUSTON
DEPARTMENT OF PUBLIC WORKS AND ENGINEERING

WILLOWBROOK WWTP IMPROVEMENTS

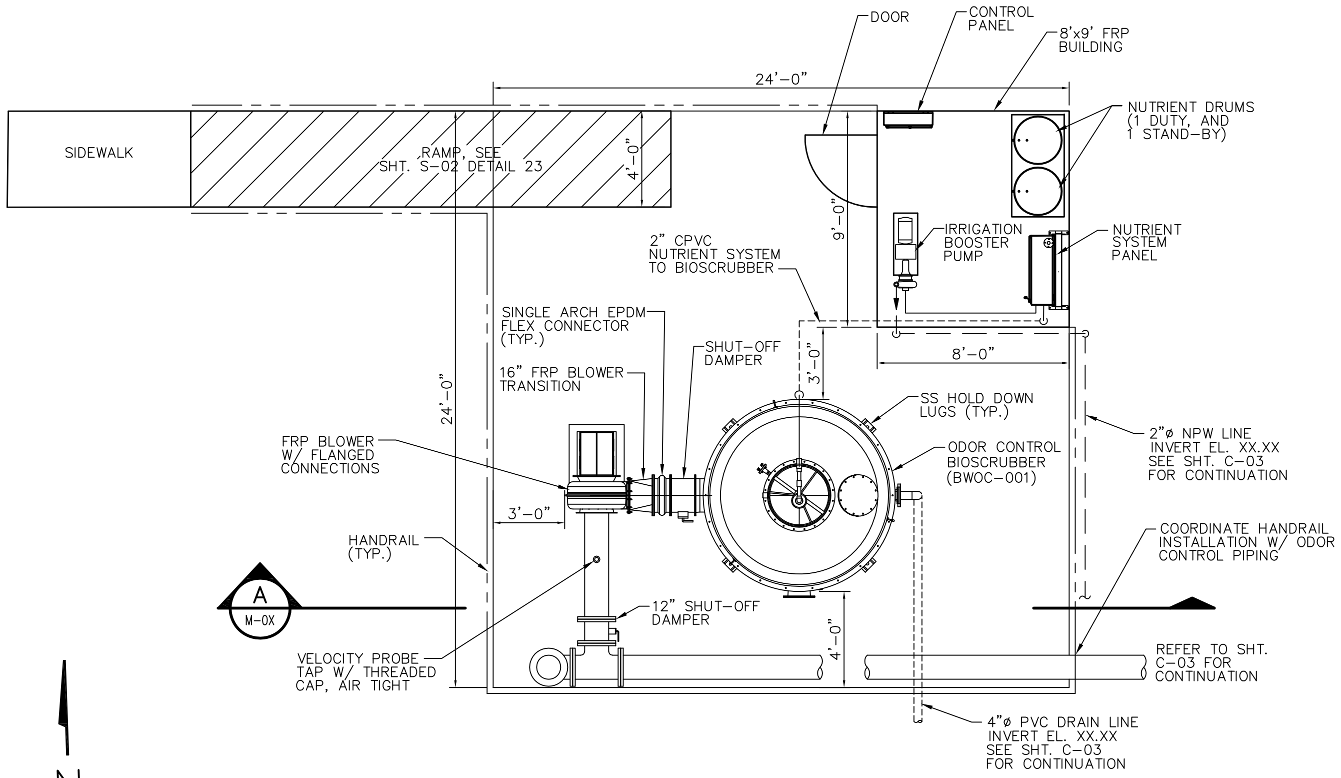
RAS PUMP STATION PLAN AND SECTIONS

WBS NUMBER
R-000265-0104-3
DRAWING SCALE
AS SHOWN
CITY OF HOUSTON PM
TANU HIREMATH, P.E.
DWG. NO.: M-14
SHEET NO. XX OF XXX

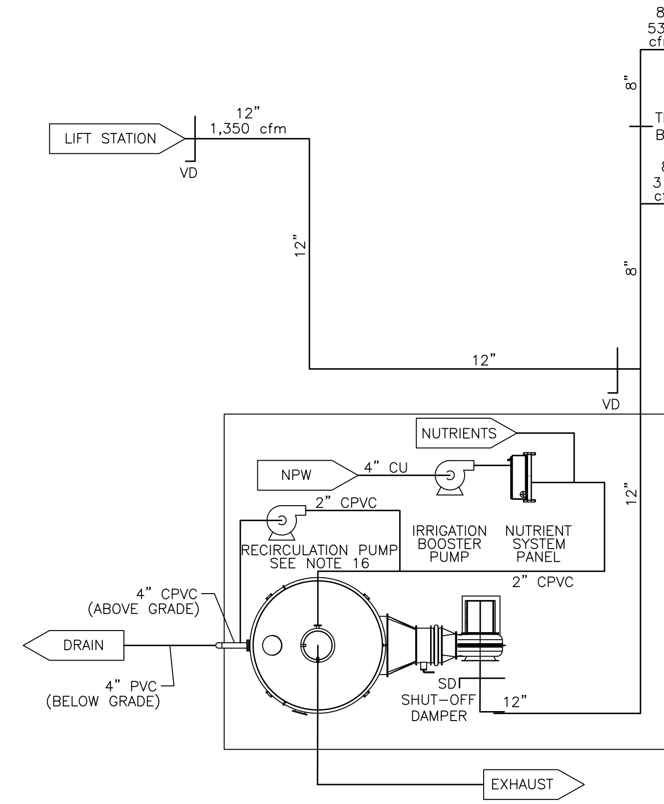
FILE NAME:
PLOTTED ON:

No.	Date	Revisions	App.

H:\Dwg\COH Willowbrook WWTP\Design Package\WBLSR_M-AerationProp.dwg Jun 24,2014 - 1:28pm staudts



PLAN VIEW
SCALE: 1/4"=1'=0"



BASIS OF DESIGN:

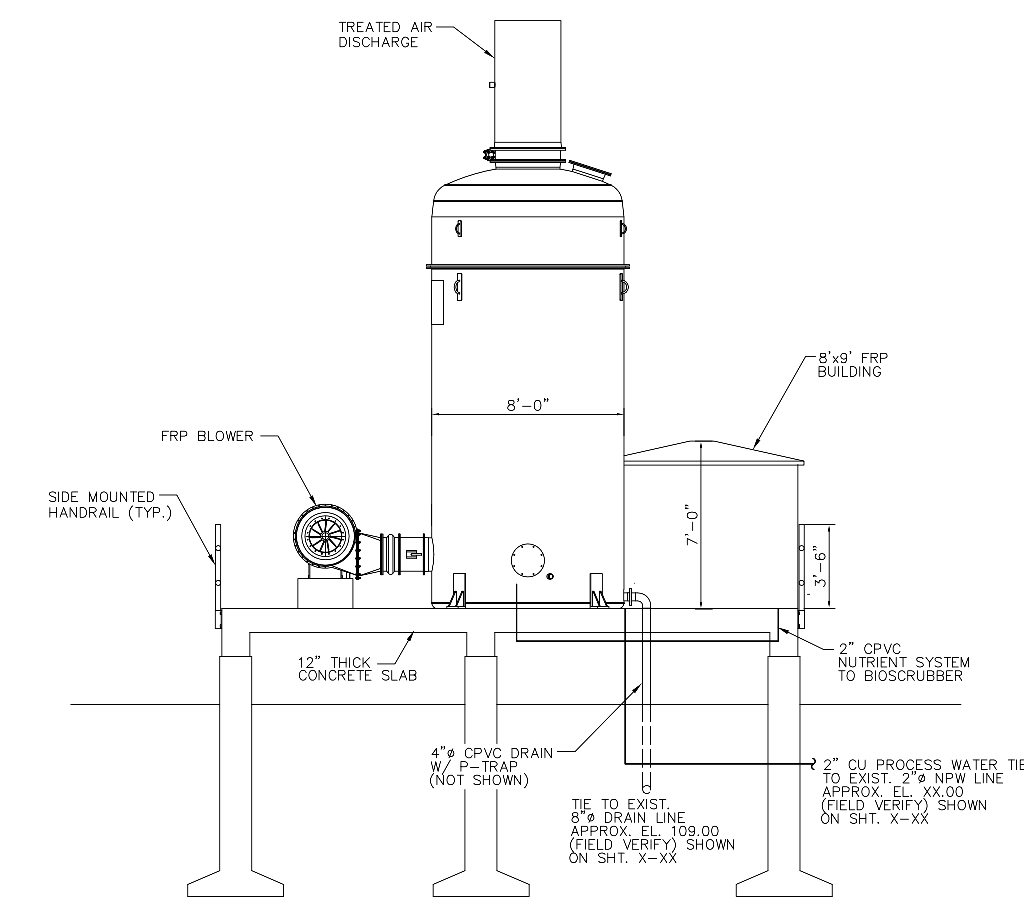
6 AIR CHANGES PER HOUR AT LIFT STATION
10 AIR CHANGES PER HOUR AT HEADWORKS
10 AIR CHANGES PER HOUR AT GRIT CHAMBERS (FUTURE)
10 AVG. H2S PPM LOADING
35 MAX. H2S PPM LOADING
55 PEAK H2S PPM LOADING
FLOW RATE: 2,200 CFM

ODOR CONTROL LINE DIAGRAM
NTS

NOTES:

1. CONTRACTOR SHALL FIELD VERIFY ALL EXISTING UTILITY AND CLEARANCE REQUIREMENTS PRIOR TO INSTALLATION.
2. 1/2-INCH DRAIN VALVES SHALL BE INSTALLED AT LOWEST POINTS IN EACH DUCT SECTION TO FACILITATE DRAINAGE OF ACCUMULATED MOISTURE. CONTRACTOR SHALL PROVIDE ALL VALVE PIPING AND APPURTENANCES REQUIRED. REFER TO SHEET XXXXXXX, NOTE XXXXXXXX.
3. ALL DUCTWORK SHALL BE CORROSION RESISTANT DUCT MATERIAL AS SPECIFIED.
4. PROVIDE WYE TEES (NOT TO EXCEED 45°) FOR ALL LATERAL DUCTWORK CONNECTION POINTS.
5. FOR CLARITY NOT ALL EQUIPMENT, PIPES, OR DUCTS ARE SHOWN. THE CONTRACTOR SHALL FIELD VERIFY ACTUAL CONDITIONS PRIOR TO COMMENCING WORK.
6. ODOR CONTROL DUCT SHALL BE PROVIDED WITH EXPANSION JOINTS AS NOTED AND REQUIRED. SUBMIT DUCT PLAN FOR APPROVAL.
7. TRANSITIONS/EXPANSIONS SHALL BE TAPERED. TAPER SHALL BE AT LEAST 5 UNITS LONG FOR EACH ONE UNIT CHANGE IN DIAMETER OR 30° INCLUDED ANGLE.
8. MAXIMIZE STRAIGHT DUCT LENGTH TO FAN INLET.
9. ALL 90° DUCT BENDS SHALL BE SMOOTH OR MINIMUM SEVEN PIECE CONSTRUCTION. ALL 90° DUCT BENDS SHALL BE LONG RADIUS TYPE.
10. LOCATE ALL BIOSCRUBBER PANELS WITH MINIMUM 4-FT CLEARANCE FROM ANY OBSTRUCTION.
11. BIOSCRUBBER PAD SHALL BE SIZED TO PROVIDE NOTED CLEARANCE REQUIREMENTS. PAD DIMENSIONS MAY VARY BASED ON SELECTED EQUIPMENT. MINIMUM DIMENSIONS OF PAD NOTED.
12. INSTALL BIOSCRUBBER AND DAMPERS IN ACCORDANCE WITH MANUFACTURER'S RECOMMENDATIONS.
13. PROVIDE VELOCITY PROBE TAP WITH THREADED CAP, AIR TIGHT FOR BALANCING AS SHOWN AND RECOMMENDED BY BIOSCRUBBER MANUFACTURER.
14. REFER TO SHEET XXXXXX FOR GENERAL STRUCTURAL NOTES.
15. ALL EXPOSED IRRIGATION AND RECIRCULATION PIPING SHALL BE INSULATED.
16. RECIRCULATION SYSTEM AS REQUIRED BY MANUFACTURER. PUMP SHALL BE SUITABLE FOR OUTDOOR INSTALLATION, AND ALL INTERCONNECTING PIPING AND VALVES SHALL BE PROVIDED.
17. NUTRIENT DRUMS SHALL BE PROVIDED WITH 2-DRUM SPILL CONTAINMENT PALLET WITH MINIMUM CAPACITY OF 66 GALLONS (EAGLE MODEL # EM-1260 OR APPROVED EQUAL).
18. PROVIDE ELECTRIC DRUM TRANSFER PUMP CHEMICALLY COMPATIBLE WITH NUTRIENT SOLUTION. PUMP SHALL BE CONTINUOUS DUTY, OPERATE ON 120 VAC, 60 Hz, AND PROVIDED WITH POWER CORD AND WALL BRACKET FOR STORAGE. (McMUSTER MODEL #42895K82 OR APPROVED EQUAL). MOUNT WALL BRACKET TO FRP BUILDING INTERIOR WALL.
19. THE SOIL IS ASSUMED TO BE CLAY/SANDY CLAY (CL, ML, MH, & CH). AS SUCH A MINIMUM BEARING PRESSURE OF 1500 PSF AND LATERAL PRESSURE OF 100 PSF/FT ARE ASSUMED AS STIPULATED IN TABLE 1804.2 OF IBC 2006.
20. THE FEMA NAVD88 100-YEAR SITE FLOOD PLAIN BASE FLOOD ELEVATION IS 116.50 FEET AND THE 500-YEAR SITE FLOOD PLAIN ELEVATION IS 117.50 FEET. DRAWINGS ARE SHOWN IN CORS ELEVATIONS. TO OBTAIN FEMA (NAVD88 2001 ADJUSTMENT) SUBTRACT 0.79' FROM DRAWING ELEVATION.

No.	Date	Revisions	App.



SECTION A
SCALE: 1/4"=1'=0" BW-M05

60% SUBMITTAL
PRELIMINARY
NOT FOR CONSTRUCTION

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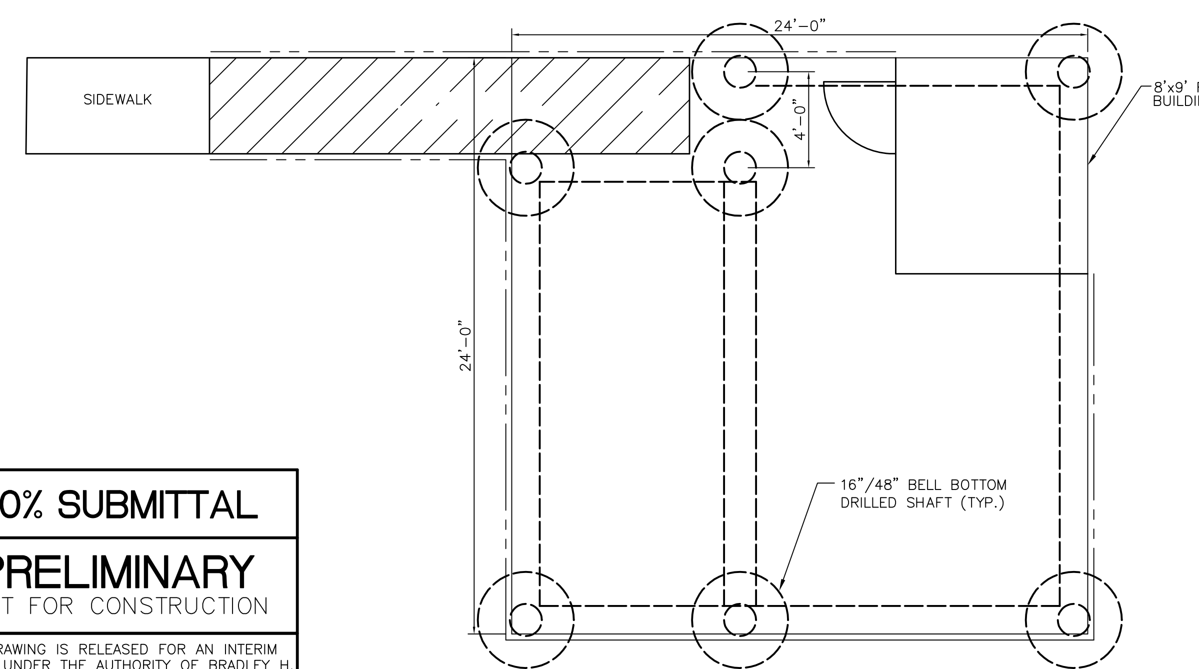

STRUCTURAL PLAN VIEW
SCALE: 1/4"=1'=0"

PRIVATE UTILITY LINES SHOWN

N/A
RELIANT ENERGY ENTEX, INC.

N/A
S.B.C. VALID FOR ONE YEAR
APPROVED FOR UNDERGROUND CONDUIT FACILITIES UNLESS NOTED

N/A
RELIANT ENERGY HOUSTON LIGHTING & POWER CO.
APPROVED ONLY FOR CROSSING UNDERGROUND DUCTLINES UNLESS NOTED. VALID AT TIME OF REVIEW ONLY.

N/A
CABLE COMPANY

WESTON SOLUTIONS
5599 SAN FELIPE, SUITE 700
HOUSTON, TEXAS 77056
713-985-6600
TBPE REGISTRATION NO. F-3123

SURVEYED BY: LANDTECH
FB NO. -

CITY OF HOUSTON
DEPARTMENT OF PUBLIC WORKS AND ENGINEERING

WILLOWBROOK WWTP IMPROVEMENTS

ODOR CONTROL PLAN AND SECTIONS

WBS NUMBER
R-000265-0104-3
DRAWING SCALE
AS SHOWN
CITY OF HOUSTON PM
TANU HIREMATH, P.E.
DWG. NO.: M-06
SHEET NO. XX OF XXX

FILE NAME:

H:\Dwg\COH Willowbrook WWTP\Design Package\WBLSR_Odor.dwg May 18,2014 - 4:00pm staudts