Appendix D

Geotechnical Investigation Report

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CITY OF LOS ANGELES
DEPARTMENT OF PUBLIC WORKS
Bureau of Engineering
GEOTECHNICAL ENGINEERING DIVISION

October 20, 2017

ENGINEER OF RECORD – BOYLE HEIGHTS SPORTS CENTER PROJECT
933 S MOTT STREET AND 2510 E WHITTIER BLVD, LOS ANGELES
TRACT: M.L. WICKS' STEPHENSON AVENUE TRACT NO. 2, BLOCK: --, LOT: 12 &
TRACT: TR 5299, BLOCK: --, LOT: 21
W.O. E170192B **GED FILE NO. 17-086**

Reference: Willdan Geotechnical "Geotechnical Investigation Report, Proposed Boyle Heights Sports Center Project, 933 South Mott Street, Los Angeles, California" dated October 17, 2017.

The Los Angeles Department of Public Works, Bureau of Engineering, Geotechnical Engineering Division has reviewed the referenced geotechnical investigation report by Willdan Geotechnical. We concur with the information presented in it and take full responsibility for the use of its contents. We also accept the role of Geotechnical Engineer of Record for the project.

Supplemental Recommendations

The supplemental recommendations provided in this Engineer of Record report supercede those in the referenced report. All other recommendations, except those specifically modified herein, remain applicable.

Section 8.2.1 Site Preparation

The existing soil beneath structural footings, including building and retaining walls, shall be removed to a depth of at least 3 feet beneath the bottom of footing. The compacted fill thickness, including the subgrade preparation, shall result in 42 inches of compacted fill beneath the footings. The lateral over-excavation shall extend 5 feet beyond the edges of the perimeter footings.

The over-excavation and replacement beneath new pavement areas shall result in at least 12 inches of non-expansive import fill material beneath the baserock.

Section 8.2.2 Fill Materials

The existing clayey soil is suitable for reuse provided the expansion index of the blended stockpile material does not exceed 50. This material shall not be used within the upper 12 inches of subgrade beneath interior slabs and pavement sections. This material shall be compacted to within 1 and 4 percent above the optimum moisture content.

Section 8.2.4 Temporary Excavation

Excavations up to 10 feet high may be sloped back at a maximum inclination of 1:1. Excavation greater than 10 feet and up to 15 feet high shall be sloped back no steeper than 1.5:1.

Section 8.2.5 Shoring Design

Cantilever or braced shoring may be considered at this site as an alternative to temporary excavations. Cantilever shoring shall only be utilized if some deflection is acceptable; therefore, it is not recommended adjacent to existing structures or utilities that cannot tolerate at least ½-inch of lateral and/or vertical movement. Sheet piles, box shoring, and/or trench shields (i.e. speed shores) are not acceptable.

Settlement of structures founded adjacent to the shoring will occur in proportion to both the distance between the shoring and the structure, and the amount of horizontal deflection of the shoring system. The vertical settlement will be a maximum at the shoring face and decrease as the horizontal distance from the shoring increases. Beyond a distance from the shoring equal to the height of the shoring, the settlement is expected to be negligible. The maximum vertical settlement is expected to be about 75 percent of the horizontal deflection of the shoring system.

Cantilever or braced shoring shall be designed for the lateral earth pressures shown on Figure 1. These values are based on the assumption that (1) the shored soil material is level at ground surface, (2) the exposed height of the shoring is no greater than 15 feet, and (3) the shoring is temporary, and will not be required to support the soil longer than about six months. Surcharge coefficients of 0.30 and 0.50 may be used with uniform vertical surcharges for cantilever and braced shoring lateral earth pressures, respectively. These surcharge pressures should be added to the lateral earth pressures.

Section 8.5 Cast-in-Drilled-Hole (CIDH) Pile

Security lights taller than 30 feet and shade structures, if proposed, may be supported on CIDH piles with a minimum diameter of 24 inches.

Axial Capacity in Compression

Axial compression capacities are presented on Figure 2 for 24-inch, 30-inch, and 36-inch diameter CIDH piles. The minimum pile embedment depth shall be 10 feet below the lowest adjacent grade. The actual depths may be deeper and will likely depend on the required lateral and tensile loads. We anticipate the piles will be isolated (i.e. spaced at least 3 pile diameters on center), and therefore, group effects are not anticipated.

The axial compression capacities presented on Figure 2 assume the CIDH piles develop their capacity solely from side resistance (i.e. skin friction).

Axial Capacity in Tension

The allowable axial tensile capacity may be assumed to be ½ the axial capacity in compression for the 24-inch, 30-inch and 36-inch diameter CIDH piles (Figure 2). The weight of the concrete shaft may be added to the tensile capacity.

Lateral Load Behavior

The lateral load behavior of the CIDH piles was evaluated using the LPILE (Ensoft, 2016) software program. LPILE (2016) uses load deflection (p-y) curves to approximate the relationship between soil resistance and pile deflection. The lateral load behavior was evaluated for both a free head deflection of ½-inch and a fixed head deflection of ½-inch. Also, we assumed a perfectly elastic pile and a cracked section. The modulus of elasticity for the cracked section was estimated to be 1802500 pounds per square inch (i.e. FS = 2).

The main inputs in the LPILE software for each soil layer are the unit weight and soil shear strength. The existing native soil was assumed to behave as "sand" with a total unit weight of 105 pcf, effective friction angle of 30 degrees, and no cohesion. The results of the LPILE analyses are attached to this report.

<u>Section 8.6.3 Lateral Earth Pressures</u>

The design lateral earth pressures for permanent retaining structures are presented on Figure 3 in this letter. The design lateral earth pressures for gravel-sand mixtures are presented on Figure 4 in this letter. The lateral earth pressures presented on Figure 4 may only be used instead of those on Figure 3 if the entire theoretical failure wedge is backfilled using select sand-gravel material. The lateral earth pressures shown on Figures 3 and 4 are applicable for backfill inclinations no steeper than 5:1 (horizontal:vertical).

For basement (i.e. restrained) walls, a seismically induced pressure increment of 10 pounds per cubic foot (pcf) may be used instead of 25 pcf. This reduced value of 10 pcf was estimated using the provisional recommendations by Lew et al. (2010) and the Mononobe-Okabe method.

If surcharge loads (live or dead) are applied, they should be added to the active or at-rest earth pressure by applying a uniform (rectangular) pressure. The lateral earth pressure coefficient for a uniform vertical surcharge is 0.33 and 0.50 for an active and at-rest condition, respectively.

<u>Section 8.6.4 Wall Foundation</u>

Retaining wall foundations shall be designed in accordance with the recommendations in Section 8.3. The minimum footing width shall remain as 2 feet. The actual footing dimensions will be based on the lateral load analysis, which shall be performed by the structural engineer.

Section 8.9 On-Site Stormwater Disposal

The infiltration tests were performed in the lower portion of the site, and not in the proposed building / parking area. We expect infiltration pits will be located in close proximity to where our infiltration tests were performed. If infiltration pits are proposed in other areas, additional testing is required. A supplemental report will be prepared following completion of the testing. Infiltration pits shall be set back at least 20 feet from structures and adjacent property boundaries. Furthermore, infiltration pits shall be set back at least 15 feet from the toe of the slope.

Section 8.10 Pavement Design

Table 5 should be replaced as follows:

ASPHALT PAVEMENT SECTION LAYER THICKNESSES (INCHES) – OPTION 1

Layer	Traffic Index ≤ 5.0	Traffic Index = 6.0	Traffic Index = 7.0	Traffic Index = 8.0	Traffic Index = 9.0
Asphalt Concrete Surface	2.0	3.0	3.5	4.0	4.5
CAB / CMB	4.0	4.5	6.0	7.0	8.0

ASPHALT PAVEMENT SECTION LAYER THICKNESSES (INCHES) – OPTION 2

Layer	Traffic Index ≤ 5.0	Traffic Index = 6.0	Traffic Index = 7.0	Traffic Index = 8.0	Traffic Index = 9.0
Asphalt Concrete Surface	2.5	3.0	4.0	4.5	5.0
CAB / CMB	3.0	4.5	4.5	6.0	7.0

CAB, CMB, and asphalt concrete shall conform to Sections 203 and 302 of the latest edition of the Standard Specifications for Public Works Construction ("Greenbook").

Portland cement concrete (PCC) may be used as an alternative to asphalt concrete. For Traffic Indexes between 6 and 7, a section of 6 inches of PCC over 8 inches of CAB/CMB is recommended. For Tis of 8 and 9, the PCC section shall be increased to 7 and 8 inches, respectively. The PCC shall have a minimum modulus of rupture of 650 psi at 28 days.

<u>Non-Structural Foundations</u>

Spread footing foundations are suitable for support of non-structural foundations, including fences, planter walls, and accessory walls less than 8 feet high. The earthwork recommendations presented in Section 8.2.1 remain applicable.

Non-structural footings shall be embedded at least 18 inches below the lowest adjacent grade. Continuous footings shall have a minimum width of 12 inches and isolated footings shall have a minimum width of 24 inches. Footings may be designed for an allowable bearing capacity of 1,500 pounds per square foot (psf). Bearing values indicated above are for total dead-load and frequently applied live-loads. The above vertical bearing may be increased by one-third for short durations of loading which will include the effect of wind or seismic forces.

The recommendations for lateral load resistance provided in Section 8.3.3 of Willdan's report remain applicable.

CLOSURE

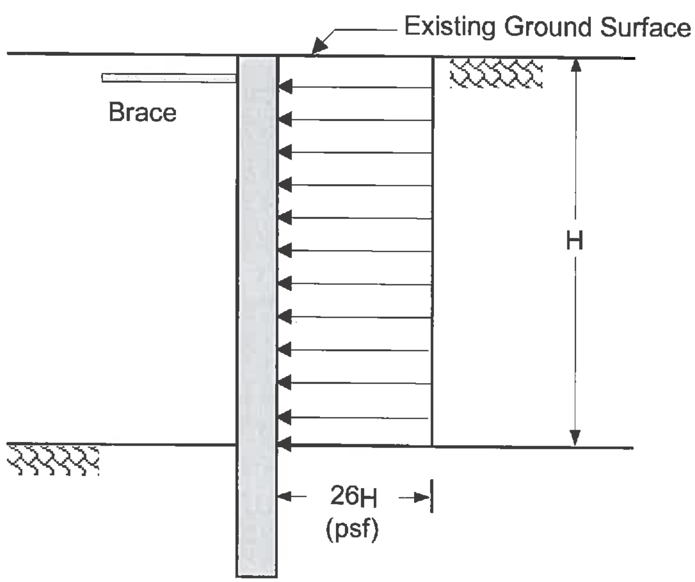
Any questions or clarification of the contents of the report shall be directed to Easton Forcier at (213) 847-0476.

REGISTERED PROFESSIONAL ENGINEER
EASTON RENE FORCIER
No. GE 2948
Exp. 9-30-18
GEOTECHNICAL
STATE OF CALIFORNIA

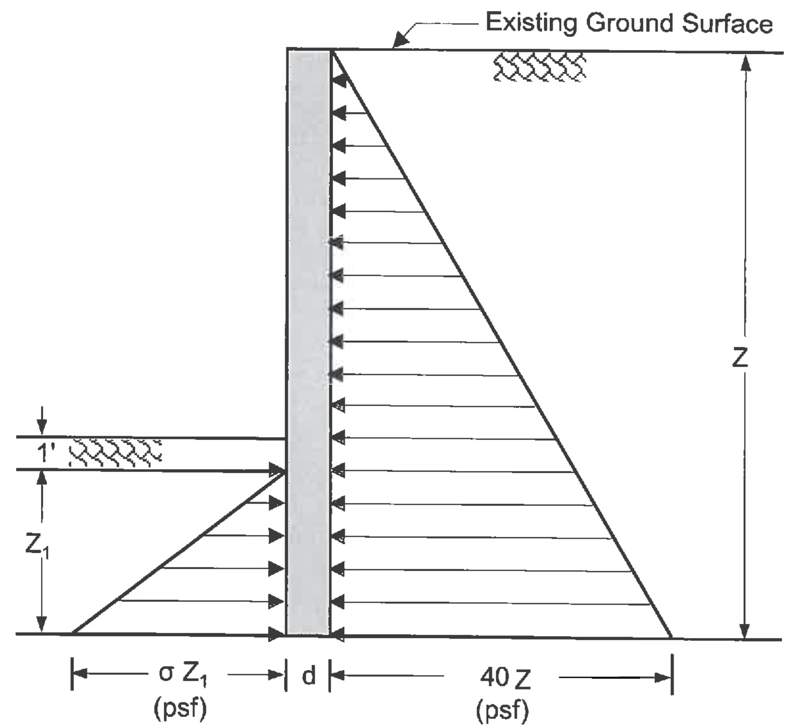
Easton F. 10-20-17

Easton R. Forcier, GE 2948
Geotechnical Engineer I

Attachments:
Figure 1 – Lateral Earth Pressures for Temporary Shoring Systems
Figure 2 – Allowable Downward Capacity of CIDH Pile vs. Depth
Figure 3 – Lateral Earth Pressures for Retaining Walls Onsite Soil Backfill
Figure 4 – Lateral Earth Pressures for Retaining Walls Select Sand / Gravel Mix Backfill
LPILE Results for 24-inch, 30-inch, and 36-inch diameter CIDH Piles
Geotechnical Investigation Report by Willdan Geotechnical dated October 17, 2017



BRACED SHORING

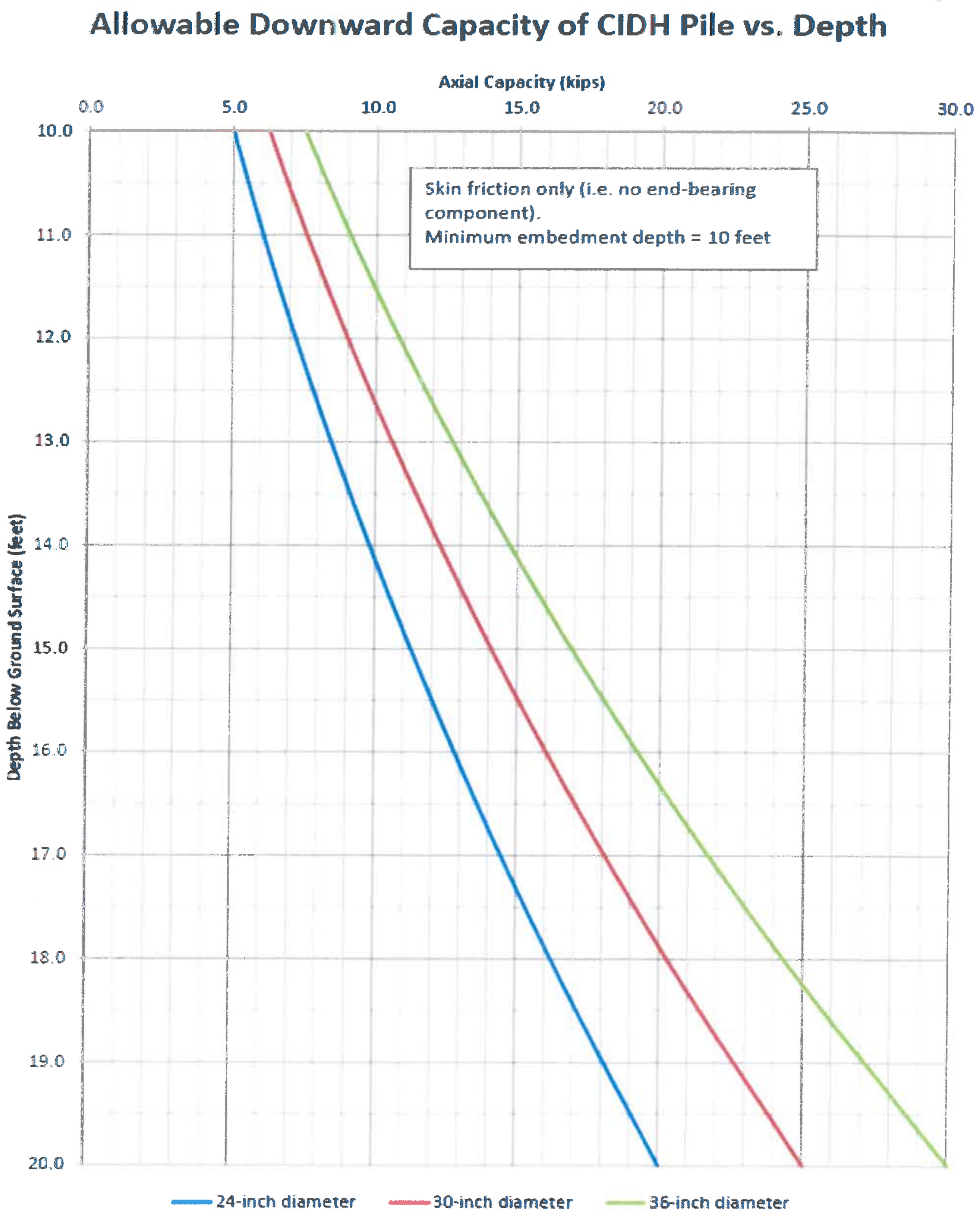


CANTILEVER SHORING

σ = 240 pcf for soldier piles spaced at least 3d apart applied over 2d
σ = 240 pcf for soldier piles spaced less than 3d apart
The maximum passive pressure shall not exceed 2,400 psf.

Notes:
1. Not to scale.
2. Dimensions are in feet.
3. Earth pressures shown are based on level backfill conditions behind shoring elements and groundwater below bottom of shoring elements.

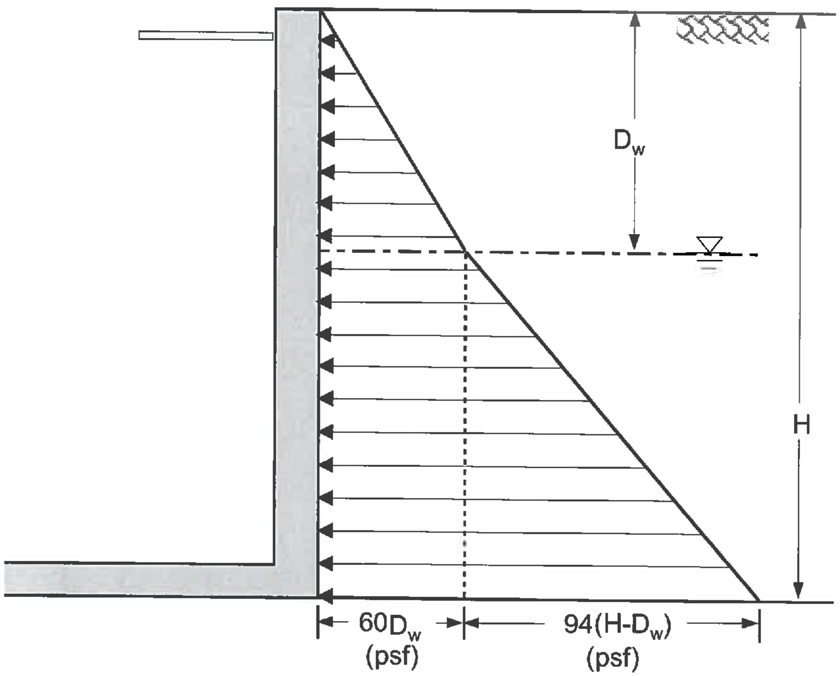
LATERAL EARTH PRESSURES FOR TEMPORARY SHORING SYSTEMS Boyle Heights Sports Center 2510 E Whittier Blvd and 933 S Mott Street Los Angeles, California		
By: ERF	Date: 10/17/17	GED File No.: 17-086
City of Los Angeles, DPW, BOE, Geotechnical Engineering Division		Figure 1



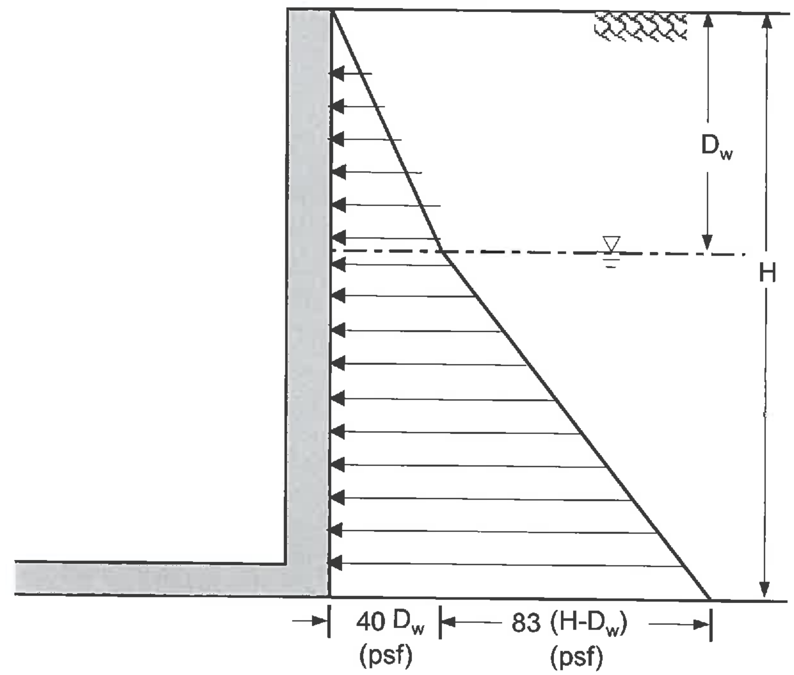
BOYLE HEIGHTS SPORTS CENTER
2510 E WHITTIER BLVD
LOS ANGELES, CALIFORNIA

BUREAU OF ENGINEERING
GEOTECHNICAL ENGINEERING GROUP
(GEO)
GEO FILE No.: 17-086
October 2017

Figure
No. 2



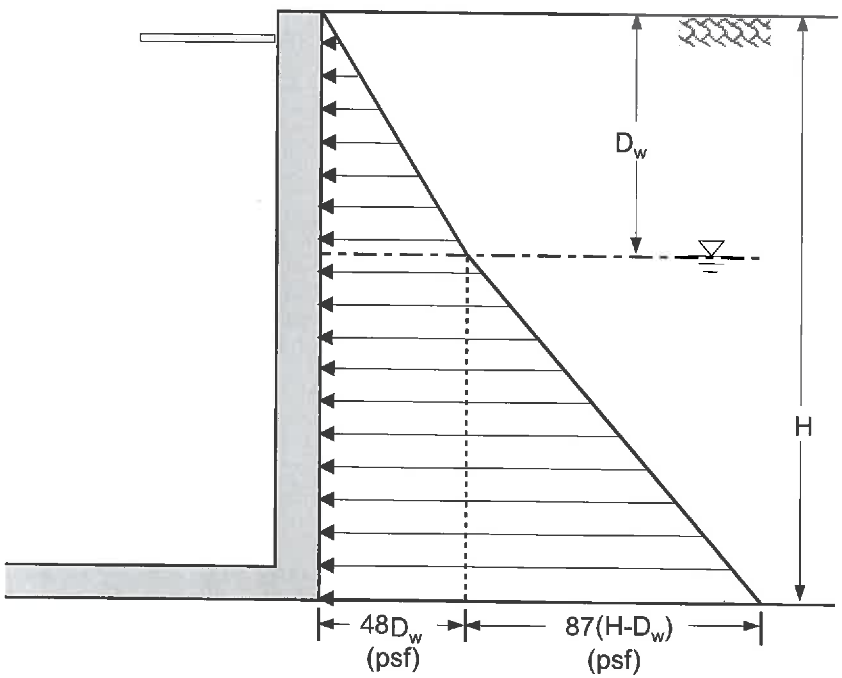
AT REST LATERAL EARTH PRESSURE UNDER STATIC CONDITIONS - RESTRAINED WALL CONDITIONS



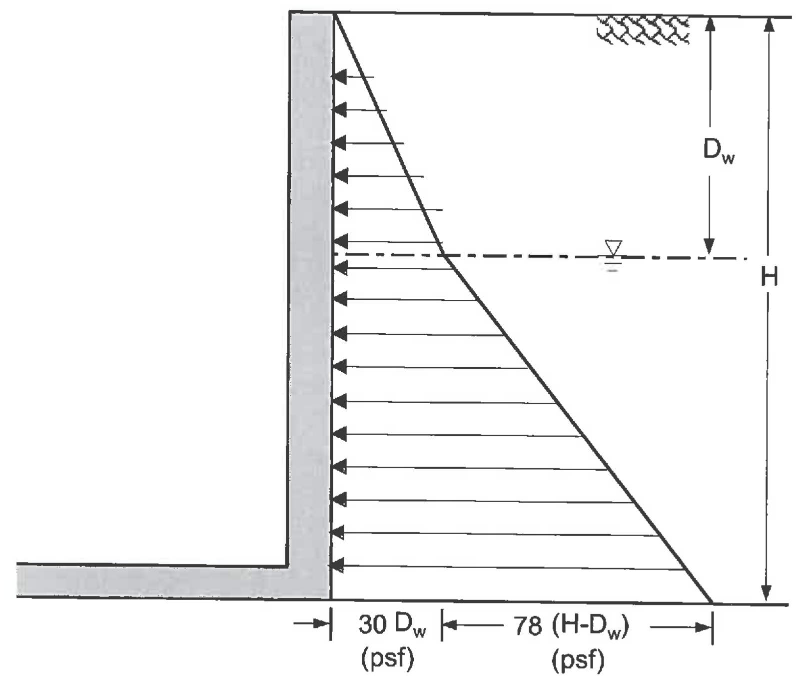
ACTIVE LATERAL EARTH PRESSURE UNDER STATIC CONDITIONS - CANTILEVERED WALL CONDITIONS

Notes:
1. Dimensions are in feet
2. If groundwater is not present, Dw should be taken as the historical high GW depth
3. The earth pressures shown are based on level backfill conditions behind wall

LATERAL EARTH PRESSURE FOR RETAINING WALLS **ONSITE SOIL BACKFILL** Boyle Heights Sports Center 2510 E Whittier Blvd Los Angeles, California		
By: ERF	Date: 10/17/17	GED File No.: 17-086
The Los Angeles Department of Public Works, Bureau of Engineering, GED		Figure **3**



AT REST LATERAL EARTH PRESSURE UNDER STATIC CONDITIONS - RESTRAINED WALL CONDITIONS

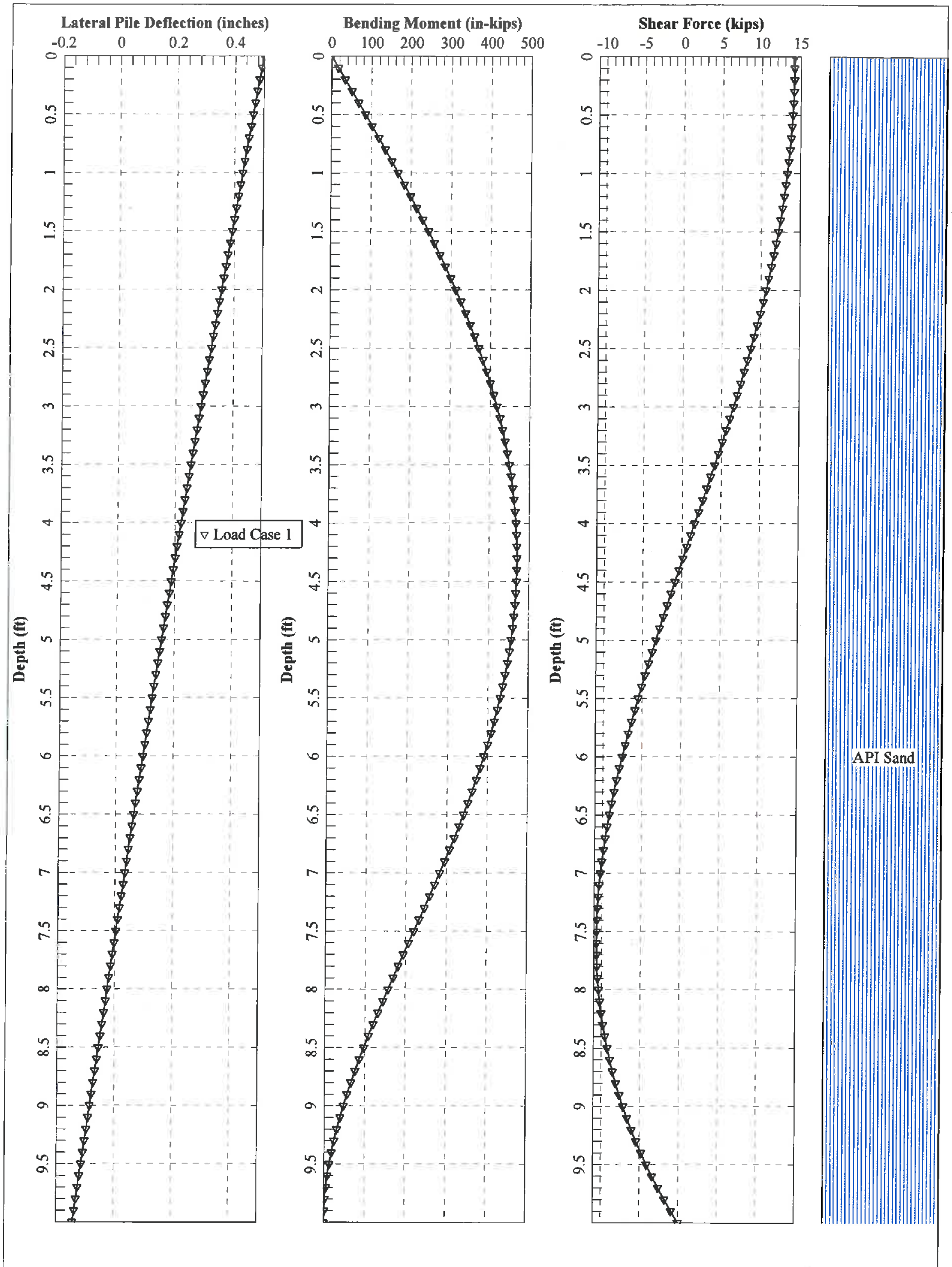


ACTIVE LATERAL EARTH PRESSURE UNDER STATIC CONDITIONS - CANTILEVERED WALL CONDITIONS

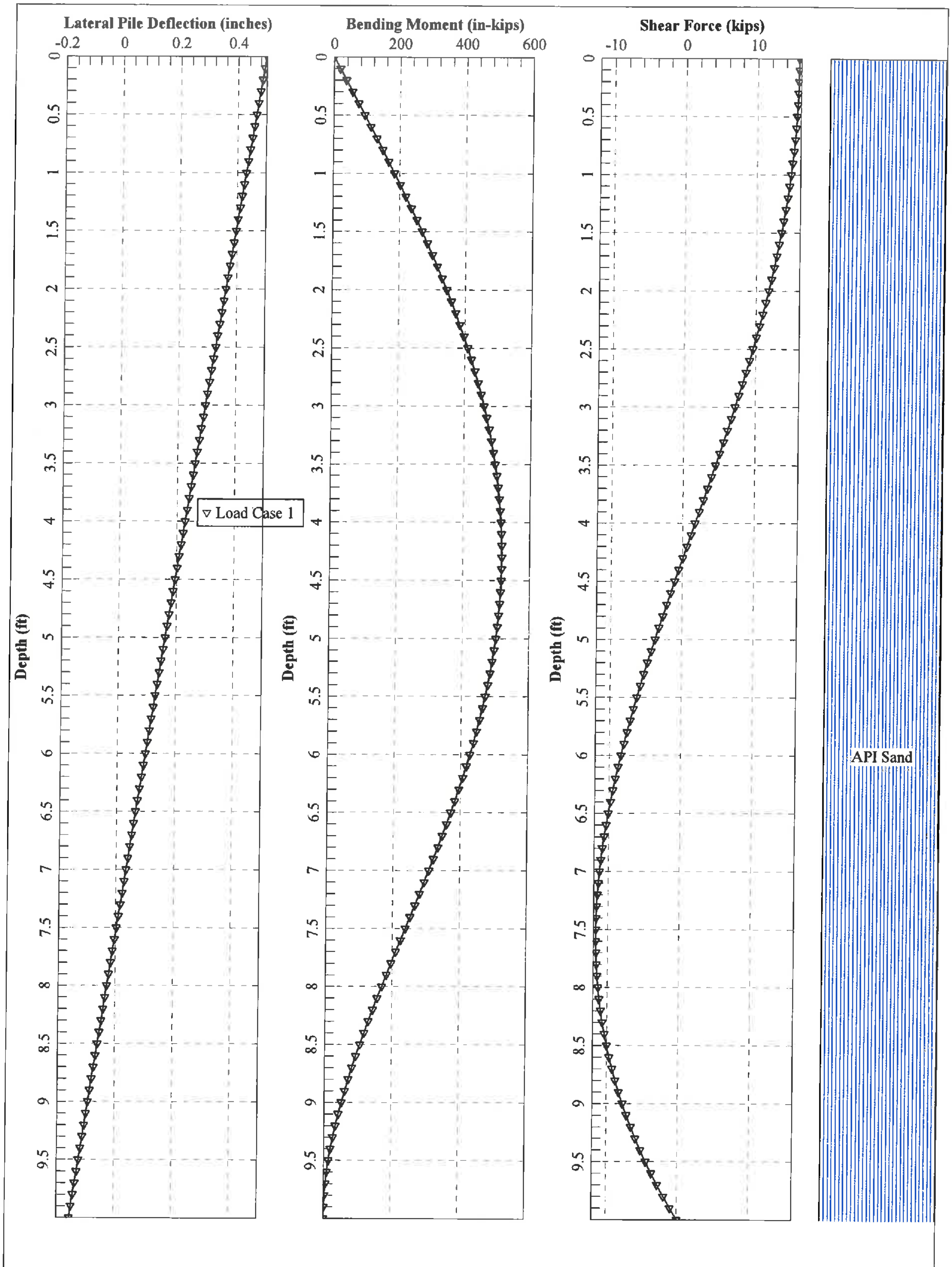
Notes:

1. Dimensions are in feet
2. If groundwater is not present, Dw should be taken as the historical high GW depth.
3. The earth pressures shown are based on level backfill conditions behind wall

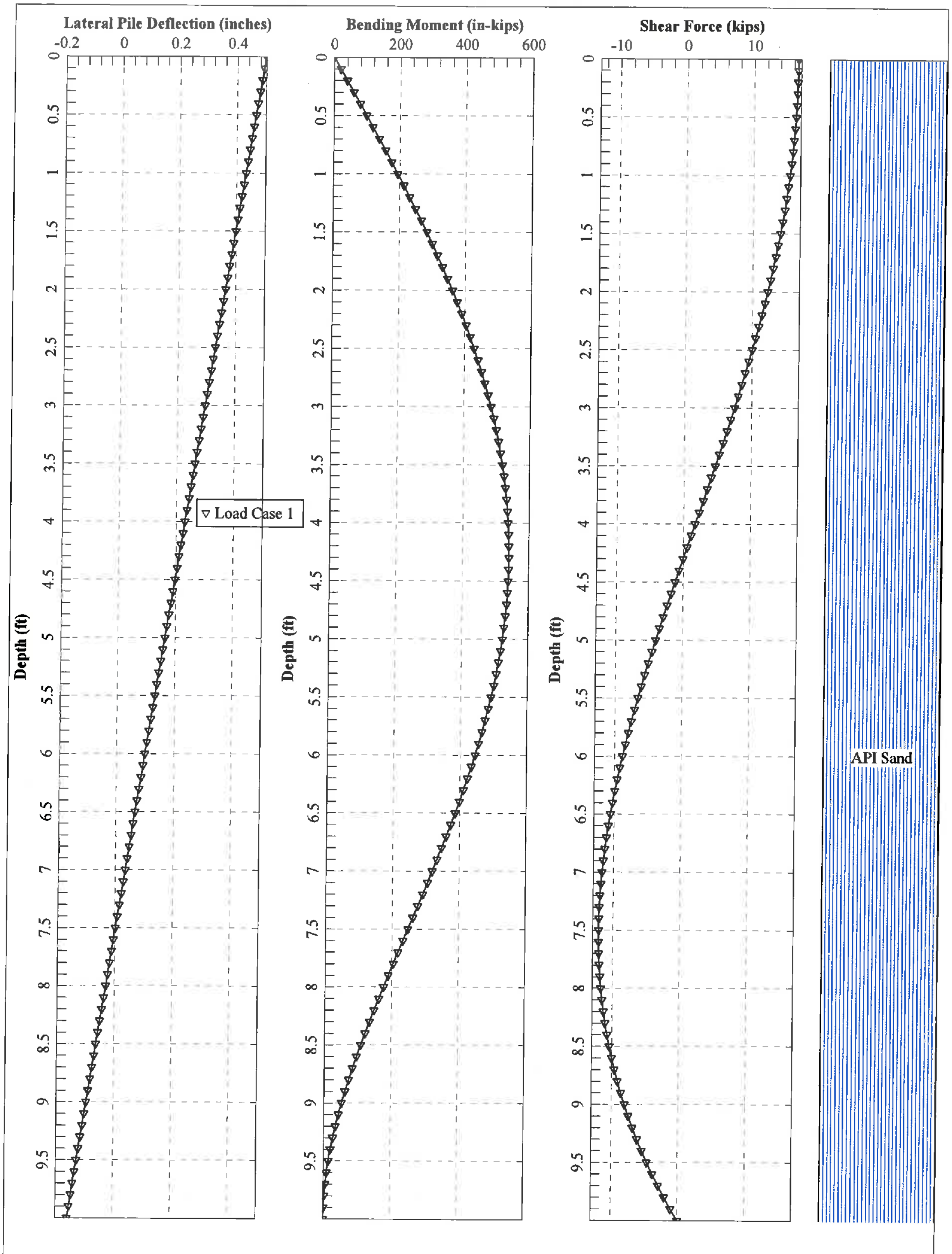
LATERAL EARTH PRESSURES FOR RETAINING WALLS **SELECT SAND / GRAVEL MIX BACKFILL** Boyle Heights Sports Center 2510 E Whittier Blvd Los Angeles, California		
By: ERF	Date: 10/17/17	GED File No.: 17-086
The Los Angeles Department of Public Works, Bureau of Engineering, GED		Figure **4**



24-inch diameter CIDH pile



30-inch diameter CIDH pile



36-inch diameter CIDH pile

GEOTECHNICAL INVESTIGATION REPORT
PROPOSED BOYLE HEIGHTS SPORTS CENTER PROJECT
933 SOUTH MOTT STREET
LOS ANGELES, CALIFORNIA

PREPARED FOR

CITY OF LOS ANGELES
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ANAHEIM, CALIFORNIA 92806
WILLDAN GEOTECHNICAL PROJECT NO. 106965-2000

OCTOBER 17, 2017

WILLDAN Geotechnical | extending your reach

October 17, 2017

Mr. Patrick J. Schmidt, PE, GE
City of Los Angeles
Geotechnical Engineering Group
1149 S. Broadway, Suite 120
Los Angeles, CA 90015-2213

Subject: Geotechnical Investigation Report
Proposed Boyle Heights Sports Center Project, Los Angeles, California
Willdan Geotechnical Project No. 106965-2000

Dear Mr. Schmidt,

Willdan Geotechnical is pleased to submit this report for the proposed Boyle Heights Sports Center project located at 933 South Mott Street in the City of Los Angeles, California. This report presents our geotechnical findings, conclusions and recommendations for the design and construction of the proposed developments. Based on the results of our investigation, the proposed development is feasible from a geotechnical standpoint, provided the recommendations in this report are followed.

We appreciate the opportunity to assist you and look forward to future projects. If you have any questions, please contact us.

Respectfully submitted,
WILLDAN GEOTECHNICAL

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Principal Engineer

Distribution: Addressee (4 unbound wet signed sets and one PDF copy via e-mail)

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1. INTRODUCTION

This report presents the findings from our geotechnical field exploration, field percolation and laboratory testing performed for the proposed Boyle Heights Sports Center project located at 933 South Mott Street in the City of Los Angeles, California. Our services were performed in general accordance to our Proposals No. 17-049 dated May 25, 2017 and 17-049R dated August 7, 2017.

This report includes the descriptions of scope of our services, drilling, logging and sampling procedures, laboratory testing procedures, field percolation testing procedures and results, as well as our recommendations for the design and construction of the proposed developments from a geotechnical standpoint.

2. SCOPE OF SERVICES

This investigation was conducted to explore and evaluate the site soil engineering conditions to the depths that may be significantly influenced by the proposed developments. Our scope of services included the following:

- A site reconnaissance by a member of our engineering staff to evaluate the surface conditions at the project site.
- Review of selected published geologic maps, reports and literature pertinent to the site and surrounding areas.
- A field exploration consisting of drilling a total of five (5) hollow-stem auger (HSA) borings. The borings were drilled to depths between approximately 26 and 36.5 feet below ground surface (bgs) to evaluate the subsurface soils conditions.
- Performing two (2) field percolation tests in two borings, at depths of approximately 5 and 10 feet bgs.
- Performing laboratory tests on representative soil samples obtained from the borings to evaluate the physical and engineering properties of the subsurface soils.
- Engineering evaluation of the data obtained from field investigation and laboratory testing.
- Preparation of this report summarizing our findings, results of geotechnical laboratory and field testing, and our conclusions and recommendations for the geotechnical aspects of project design and construction.

3. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The project site is located in the north portion of Boyle Heights Park, south of Whittier Boulevard between South Mathews Street and South Mott Street in the City of Los Angeles, California. The latitude and longitude at the approximate center of the project site are 34.0331° N and 118.2138° W, respectively. The project site location is shown on Figure 1, Site Location Map.

The project site comprises two relatively flat areas separated by a sloped area. The higher and lower flat areas are located in the northwest and southeast portions of the project site with approximate average elevations of 282 and 252 feet, respectively. The more detailed project site topography is shown on Figure 2, Boring, Percolation Test and Cross Section Location Plan. Currently, the higher flat area is covered by asphalt concrete (AC) pavement and includes two one-story buildings, and the lower flat area is occupied by an AC paved basketball court and a playground area. The slopes are dirt areas covered by trees and have an approximate average height of 30 feet. There is also a drainage swale that extends from Whittier Boulevard to the playground and a green belt area to the west.

We have been provided with the plans of two preliminary alternatives of the building, which are provided in Appendix G of this report. According to these plans, the project includes construction of a new 10,000 square feet gymnasium building consisting of a high school standard full sized basketball court, offices, storage rooms, restrooms, and a parking lot. The gymnasium building will be located on the higher flat area and may extend beyond the slope area. There will also be some grading work at the slope area for construction of new landscape area and an ADA ramp to allow people to access the existing basketball court and synthetic soccer field. Although, the grading plan is not available at this time, it is anticipated that the cut and fill thicknesses will not exceed 3 feet.

By the time this report was prepared, we had not been provided with the anticipated structural loads applicable on the foundations for the proposed structure. We assume that the imposed column loads will be less than 50 kilo pounds (kips), and imposed continuous footing loads will be less than 5 kips per foot (kpf) for the structure.

4. GEOLOGY

4.1. GEOLOGICAL SETTING

The subject site is located south of the Santa Monica Mountains, east of the Los Angeles River, and in the northeastern portion of the Los Angeles Basin locally known as Boyle Heights. The basin is located within the Peninsular Ranges geomorphic province and bounded on the east and southeast by the Santa Monica Mountains. The Peninsular Ranges are characterized by a series of northwest-southeast oriented fault blocks and sediment-floored valleys Major fault zones within this province include the Newport Inglewood, Elsinore, San Jacinto and San Andreas fault zones.

The site appears to be located within the bottom of an old stream channel, and the surrounding area generally slopes down towards the site. Locally, the site is covered by alluvial deposits. The alluvium underlying the project area ranges from younger, Holocene age alluvium consisting mainly of loose to medium dense sand, silt, and gravel to older Pleistocene age alluvium consisting mainly of dense to very dense sand, silt and gravel.

4.2. REGIONAL AND LOCAL FAULTS

The project site is located in seismically active Southern California. The California Geological Survey defines active and potentially active faults in the Alquist Priolo (AP) Geologic Hazard Zone Act (1994). For the purpose of the Act, active and potentially active faults are defined as those that have ruptured during Holocene (11,000 years ago) and Quaternary (1.5 million years ago) respectively. Maps of Earthquake Fault Zones have been published by the California Geological Survey in accordance with the AP Geologic Hazard Zone Act, 1994, which regulates developments near active faults. Based on our review of these maps, the site does not lie within an AP Zone. However seismic risk is considered high as compared to other areas of California because of the proximity to active faulting. Active and potentially active faults in California have been mapped by Jennings and Bryant (2010). Elysian Park Blind Thrust fault (FPFT) is the closest fault with surface projections of potential rupture area located at distances of approximately 3 miles from the site. Although EPFT might generate strong motion at the site, it is not considered to be capable of generating surface rupture. The closest potentially active/ potentially active fault to the project site is the Raymond Fault a left lateral riverse-oblique fault that has been reported as mostly Holocene and Quaternary in part. This fault is located approximately 5.9 miles north of the site and is capable of producing earthquakes with moment magnitude range of 6.8 to 8.0.

5. GEOTECHNICAL INVESTIGATIONS

5.1. FIELD EXPLORATION

Field exploration for this project consisted of drilling and sampling five (5) HSA borings to depths between approximately 26 and 36.5 feet bgs. Willdan also conducted drilling and sampling two (2) HSA borings, one to 5 feet bgs and the other to 10 feet bgs for the purposes of percolation testing. Approximate locations of borings are shown on Figure 2.

Prior to field exploration, a site visit was performed to mark the boring locations and evaluate access conditions for drilling equipment. Underground Service Alert (USA) of Southern

California was then notified for clearance of underground utilities in the vicinity of the subsurface exploration locations.

Soil borings were advanced using a truck-mounted CME 75 rig equipped with 8-inch diameter hollow-stem augers. Bulk, disturbed and relatively undisturbed soil samples were collected from each soil boring during drilling. Bulk samples were collected from auger cuttings obtained from within the upper 5 feet soils. At selected intervals throughout the boring depths, relatively undisturbed soil samples were collected by driving a 3-inch outside diameter Modified California sampler lined with brass rings, and disturbed samples were collected by driving a 1⅜ inch inside diameter Standard Penetration split-spoon sampler. The samplers were driven into the underlying soil to a depth of 18 inches, or the interval noted on the boring logs, with a 140-pound hammer falling 30 inches. The number of blows required to drive the sampler was recorded for each 6-inch penetration interval and is shown on the boring logs. Soil samples were retained for possible laboratory testing. The number of blows required to drive the sampler the last 12 inches was used to estimate the in-situ relative density of granular soils and to a lesser degree of accuracy, the consistency of cohesive soils. The samples were also screened using a photo-ionization detector (PID) to detect the presence of volatile gases, an indication of potential soil contamination. The PID readings are shown on the boring logs.

Classification of the soils encountered in our exploratory borings was made in general accordance with the Unified Soil Classification System (USCS), using visual-manual procedure (ASTM D2488) and/or based on laboratory testing (ASTM D2487). A key for the classification of the soils (USCS classification) along with the boring logs are provided in Appendix A.

Upon completion of drilling, the borings were backfilled with soil cuttings, tamped, and patched with cold asphalt as appropriate. Soil samples collected from the field were delivered to Willdan Geotechnical's laboratory for testing.

5.2. LABORATORY TESTING

As requested by the Geotechnical Engineering Group (GEO), laboratory tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. Laboratory testing included determination of in-situ moisture content and dry density, percent passing #200 sieve, gradation, Atterberg limits, direct shear, consolidation, compaction curve, expansion index (EI), R-value and corrosion potential for soil samples collected from various depths. Laboratory tests were conducted in general accordance with American Society for Testing of Materials (ASTM) Standards or California Test Methods (CTM). The in-situ dry density and moisture content test results are shown on the boring logs. The remaining laboratory test results are provided in Appendix B, Laboratory Test Results. The laboratory test results indicate that:

- The shallow subsurface clayey soils within the proposed building area have an EI of 55 and according to ASTM D4829 are classified as soils with medium expansion potential. As such, the recommendations provided in Section 8.2.2 of this report shall be incorporated in design and construction of the project.

- The soils encountered in the borings have in-situ dry densities ranging from 88 to 126 pounds per cubic foot (pcf) and moisture contents ranging from 2.5% to 11.5%, with an exception for the sample in Boring B-5 at depth of 20 feet that has a moisture content of 28.4%.

- Based on the consolidation test results on a soil sample (B-1@5') and the criteria addressed in the Naval Facilities Engineering Command Design Manual 7.01 (NAVFAC DM 7.01), the existing shallow subsurface soils within the proposed building area have a collapse potential of 7% and considered as Trouble Soils.

- The subsurface soils have peak cohesion ranging from 5 to 390 pounds per square foot (psf), and ultimate cohesion ranging from 5 to 225 psf. The internal friction angle of soils ranges from 28.5 to 32 degrees for peak value, and from 27.5 to 32 degrees for ultimate value. The following shear strength parameters have been used for the subsurface soils at the slope areas.

Table 1. Soils Profile

Layer No.	Depth (ft)	Unit Weight (pcf)	Peak		Ultimate	
			Cohesion (psf)	Friction Angle (degree)	Cohesion (psf)	Friction Angle (degree)
1	0 to 10	100	10	30.0	5	30.0
2	Below 10	125	390	28.5	225	27.5

5.3. SUBSURFACE CONDITIONS

Uncertified fill was not encountered in the borings. Based on the results of the field exploration, it can be concluded that the subsurface material within the subject project site predominantly consists of Holocene to older Pleistocene-age alluvium, which mainly includes medium dense to very dense sandy materials to the maximum drilled depth of 36.5 feet bgs. The sandy materials are interbedded with silt and clay layers.

This appears typical of those found in the geologic region of the site. The above is a general description of soil conditions encountered at the site in the borings drilled for this investigation. For a more detailed description of the soil conditions encountered, refer to the boring logs in Appendix A.

5.4. GROUNDWATER

The site is in the southwest portion of the Los Angeles Quadrangle, where the historically highest groundwater has been identified as about 150 to 200 feet (CGS Seismic Hazard Zone Report 029, 1998). The borings conducted for the current investigation were monitored for visible signs of free groundwater during and immediately after completion of the borehole. Groundwater was not encountered during the drilling operations on June 28, 2017.

Depth to groundwater can be expected to fluctuate both seasonally and from year to year. Fluctuations in the groundwater level may occur due to variations in precipitation, irrigation practices at the site and in the surrounding areas, climatic conditions, pumping from wells, and possibly as the result of other factors that were not evident at the time of our investigation. Because of the type of the proposed developments, it is unlikely that groundwater would be encountered during the course of construction for the proposed developments.

5.5. FIELD PERCOLATION TESTING

The average infiltration rate for the on-site shallow subsurface soils was measured by two (2) falling head percolation tests conducted at the locations of Borings TW-1 and TW-2 as shown on Figure 2. The percolation tests were performed in accordance with the boring percolation testing procedures presented in Low Impact Development Best Management Practice, Manual GS200.1, published by County of Los Angeles.

Borings TW-1 and TW-2 were drilled to depths of approximately 5 and 10 feet bgs, respectively. The tests were conducted on June 28, 2017. Perforated PVC pipes, 3 inches in diameter, were placed in the boreholes. The bottom of the test hole and the annular space were filled with free draining gravel. The holes were first pre-saturated by filling with water to the depth of approximately 4 inches and topping off the water when it was necessary. The holes were presoaked 4 hours before conducting the infiltration tests. Then the water level was monitored by measurements taken every 30 minutes based on the permeability of soils within the borehole, until the rate of fall in the water level became steady. The test data and calculations are included in Appendix D, and the test results are summarized in Table 2.

Table 2. Percolation Tests Results

Test Location (See Figure 2)	Boring Depth (ft)	Soil Encountered	Adjusted Infiltration Rate (in/hr)
TW-1	5.0	0' to 5': Clayey Sand (SC)	1.88
TW-2	10.0	0' to 6': Clayey Sand (SC); 6' to 10': Silty Sand (SM)	3.00

6. SEISMIC CONSIDERATIONS

6.1. SITE CLASS

The subsurface soil profile at the site can be classified from a seismic standpoint based on the conditions encountered in our exploratory borings, and anticipated within the upper 100 feet of the site based on geologic mapping, as being a very dense soil and soft rock with undrained shear strength of more than 2,000 pounds per square foot (psf) and SPT N-values of more than 50 blows per foot. Based on the soils encountered in the borings drilled within the subject site and with consideration of the geologic units mapped in the area, it is our opinion that the site soil profile corresponds to Site Class C in accordance with Section 1613.3.2 of the California Building Code (CBC 2016).

6.2. 2017 LABC SEISMIC DESIGN PARAMETERS

The site class per Section 1613.3.2 of the CBC 2016 is based upon the site soil conditions. It is our opinion that Site Class C is most consistent with the subject site soil conditions. For design of the structures based on the seismic provisions of the CBC 2016, we recommend the parameters in the following Table 3.

Table 3. Seismic Design Parameters

Seismic Item	Value	CBC Reference
Site Class	C	Section 1613.3.2
F_a	1.0	Table 1613.3.3(1)
S_s	2.336	Figure 1613.3.1(1)
S_{MS}	2.336	Section 1613.3.3
S_{DS}	1.557	Section 1613.3.4
F_v	1.3	Table 1613.3.3(2)
S_1	0.815	Figure 1613.3.1(2)
S_{M1}	1.059	Section 1613.3.3
S_{D1}	0.706	Section 1613.3.4

Site Coordinates: Latitude: 34.0331° N Longitude: 118.2138° W

6.3. SOIL LIQUEFACTION

Soil liquefaction is a state of temporary soil particle suspension caused by loss of strength due to pore pressure increase resulting from cyclic stress induced by earthquakes, and the resultant drop in effective stress and soil shear strength. Liquefaction normally occurs in saturated granular soils, such as sands, in which the strength is purely frictional. Soils most susceptible to liquefaction are saturated, loose, uniformly graded, fine-grained sand deposits. However, liquefaction has occurred in soils other than clean sands. Silty sands and sandy silts have also been reported to be susceptible to liquefaction or partial liquefaction. The occurrence of liquefaction is generally limited to soils located within about 50 feet of the ground surface. Primary factors affecting the potential for a soil to undergo liquefaction include:

1) Depth to groundwater;
2) Soil type;
3) Relative density of the soil and initial confining (overburden) pressure; and
4) Intensity and duration of ground shaking.

Potential problems associated with soil liquefaction include ground surface settlement, loss of foundation bearing support strength, and lateral spreading. Ground surface settlement due to densification of the liquefied soils can be approximated using procedures developed by Tokimatsu and Seed (1987), Ishihara and Yoshimine (1992), or Idriss and Boulanger (2008). While liquefaction occurs in confined sand layers, a phenomenon referred to as sand boils may occur at the ground surface. Sand boils occur when the sudden compression of groundwater in a layer of saturated clean loose sand builds up sufficient pressure to rupture up through the upper soil mantle to the ground surface. When this occurs, displacement of the liquefied sand results in the sudden loss of support of structures supported by shallow foundations.

The project site has not been mapped as being within a zone susceptible to liquefaction as designated by the State of California (CGS, 1999). The soils underlying the project site consist of very dense soils, and the groundwater table at the project site is expected to be very deep (below 150 feet bgs). Therefore, it is our opinion that liquefaction is not a potential hazard at the project site.

6.4. SEISMICALLY INDUCED SETTLEMENT OF UNSATURATED SANDS

In addition to the settlement of saturated sand deposits due to liquefaction, strong seismic shaking can also cause settlement or densification of sands above the groundwater as well. Seismic-induced settlement of sands above the groundwater can potentially result in settlement of the ground surface. Due to the fact that the project site is underlain by very dense soils, the settlement within the project site is considered very low to nil and within the tolerable limit for the type of proposed structure, and is not expected to pose significant impact to near surface foundation of the structure.

6.5. LATERAL SPREADING

Liquefaction may lead to lateral spreading. Lateral spreading happens when surficial soil moves in a direction parallel to the ground surface due to liquefaction of underlying subsurface soils layers. Lateral spreading generally moves down gentle slopes, usually less than 6%, (Naeim

1989) or slip toward a free face such as an incised river channel. The site is not liquefiable, hence lateral spreading is not likely to occur at the project site.

6.6. GROUND LURCHING

Ground lurching is movement of the ground surface during seismic event, resulting in cracks and ridges developing perpendicular to the slope face. Areas underlain by thick alluvium with loose granular soils or clay soils with high moisture are susceptible to ground lurching. Ground lurching often causes damage to lightly loaded structures such as pavements, walkways, pipelines and other near-surface improvements located within the failure zone. Since the site is mainly underlain by stiff to very stiff sandy clay and/or medium dense to dense clayey sand, it is not subject to earthquake-induced ground lurching.

6.7. LANDSLIDING

The project site has not been mapped as being within a zone susceptible to landsliding as designated by the State of California (CGS, 1999). No evidence for landsliding was observed on or in the immediate vicinity of the site. As such, and due to the lack of significant topographic changes at the project site, landsliding is not a potential hazard on the site.

6.8. TSUNAMI AND SEICHING

The project site is not located near any enclosed bodies of water and therefore, tsunami and seiching are not considered to be potential hazards on the site.

7. SLOPE STABILITY

7.1. GLOBAL SLOPE STABILITY

Stability of the existing southeast and south descending slopes with a maximum relief of approximately 28 feet high and slope ratio ranges of 2.8H:1V to 2.2H:1V has been evaluated in accordance to the guidelines of LADBS Information Bulletin P/BC 2017-049. Selected cross sections A-A' on southeast and B-B' on south descending slopes are shown on Figure 2.

Ultimate and peak shear strength parameters, as provided in Table 1, were used to evaluate the existing slopes under static and pseudo-static conditions, respectively. The seismic coefficient, k_{eq}, used in pseudo-static stability analyses were determined in accordance with the guidelines addressed in Section 4.b of LADBS IB P/BC 2017-049, assuming PGA_M of 0.88g, fault distance (r) of 9.22 km, earthquake magnitude (M) of 6.62, and threshold (u) of 15 cm. The r and M values were obtained using the USGS Unified Hazard Tool website.

Our analyses indicate that the existing slopes have a minimum factor of safety of 2.59 and 1.85 under static and pseudo-static conditions, respectively. Based on our field observation and slope stability analyses, the existing slopes are considered stable. Slope stability analyses are provided in Appendix C, and the cross sections are shown on Figure 2.

7.2. SURFICIAL SLOPE STABILITY AND LANDSCAPING

Surficial stability of the existing descending slopes at cross section B-B' with a slope ratio of 2.2H:1V has been evaluated in accordance to the guidelines of LADBS Bulletin P/BC 2017-049 and using a weighted average value of the ultimate shear strength parameters, as provided in Table 1. Our analyses indicate that the existing slopes have a minimum factor of safety of 1.69 for surficial stability under static condition. Based on our field observation and slope stability analyses, the existing slopes are considered surficially stable. Surficial slope stability analyses are provided in Appendix C.

All slopes will be subject to surficial erosion. Therefore, slopes should be protected from surface runoff by means of concrete interceptor drains. All slopes should be landscaped with a suitable plant material requiring irrigation water in order to thrive. Overwatering and subsequent saturation of slope surfaces should be avoided. Slope maintenance is required during and after construction. Maintenance includes corrections of defective drainage terraces on slopes, elimination of burrowing rodents, and corrections of defective irrigation facilities. Irrigation programs for all landscapes slopes should be well controlled and minimized. Seasonal adjustments should be made to prevent excess moisture in the slope soils.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1. GENERAL

Based on our geotechnical investigation, the proposed developments are feasible from a geotechnical point of view, provided the recommendations contained in this report are implemented in the design and construction of the project.

8.2. EARTHWORK

8.2.1. Site Preparation

The proposed new Gymnasium Building will be constructed in place of the existing buildings which will be demolished. After demolishing and prior to construction of the new building, all the demolished material, vegetation, trash, and debris should be cleared and disposed of offsite. During grading, the contractor should take all necessary measures to protect existing utilities within the grading limits. All abandoned utilities encountered should be removed or otherwise drained for all content, if any, and properly capped. Any soils disturbed during site clearing operations in the construction areas should be removed down to the required depth within the suitable undisturbed soils.

Reworking of the earth materials beneath the designated footprint of the proposed developments shall be performed as recommended below:

- **Structural Footings:** As mentioned in Section 5.2, the shallow subsurface soils within the area of the proposed new building are collapsible. As such, we recommend that the entire footprint area of the proposed building be over-excavated and replaced with at least 5 feet of engineered fill below the bottom of footings or engineered fill that extends to a minimum depth of 6.5 feet below the lowest adjacent finished grade, whichever provides the deeper fill. Over-excavation shall laterally extend at least 5 feet from outer faces of the perimeter footings in all directions, where possible.

- **Non-Structural Footings:** The soils below non-structural footings shall be over-excavated and replaced with at least 2 feet of compacted fill below the bottom of footings. Over-excavation shall laterally extend at least 2 feet from outer faces of the footing in all directions, where possible.

- **Interior Concrete Slab-On-Grade:** The interior slab-on-grade for the new building shall be supported on engineered fill as recommended for structural footings.

- **Exterior Concrete Slab-On-Grade:** It is recommended that the upper 12 inches of soils below exterior concrete flatworks or hardscapes located around and within the vicinity of the proposed developments and subject to pedestrian loads only, be over-excavated and replaced with compacted fill. Over-excavation shall laterally extend at least 2 feet beyond the perimeter of the slab, where possible.

- **Pavement:** It is recommended that the upper 12 inches of soils below pavements be over-excavated and replaced with compacted fill. Over-excavation shall laterally extend at least 2 feet beyond the perimeter of the pavement, where possible.

After removal of unsuitable soils and prior to placement of fill, the bottom of removal shall be observed and confirmed to be competent by the Geotechnical Engineer of Record. Following the over-excavation, the areas to receive engineered fill shall be scarified to a minimum depth of 8 inches, moisture-conditioned within 3% above optimum moisture content and compacted to at least 90% relative compaction of the maximum density as determined by the ASTM D1557.

For structural fill, all clayey materials should be placed in loose lifts of 8 inches or less, moisture-conditioned within 3% above optimum moisture content and compacted to at least 90% relative compaction of the maximum density as determined by the ASTM D1557. Granular fill materials with less than 15% finer than 0.005 mm, including over-excavation bottoms, should be compacted to at least 95% relative compaction of the maximum density as determined by the ASTM D1557. For other fills, the fill materials should be placed in loose lifts of 8 inches or less, moisture-conditioned within 3% above optimum moisture content and compacted to at least 90%

relative compaction of the maximum density as determined by the ASTM D1557. Compaction should be verified by observation, probing, and testing by a geotechnical consultant's representative.

Once the subgrade and fill soil have been moisture conditioned and compacted, the soil should not be allowed to dry out prior to additional fill placement or concrete placement at finished grade. If it is dried out prior to compaction of the fill or prior to foundation and slab-on-grade construction, reprocessing of the soil is required to reestablish the recommended soil moisture content.

When the work is interrupted by heavy rains, fill operations shall not be resumed until the Geotechnical Engineer indicates that the moisture content, density and stability of previously placed fill are as specified. All soft or wet subgrade soil encountered during construction should be stabilized prior to the placement of new fill and further construction. Wet to saturated soils may become unstable or "pump" under dynamic loading such as equipment movement during grading and may not respond to densification techniques. Typical remedial measures include discing and aerating the soil during dry weather, mixing the soil with dryer materials, removing and replacing the soil with an approved fill material, or mixing the soil with an approved lime or cement product. Our firm should be consulted prior to implementing remedial measures to observe the unstable subgrade conditions and provide appropriate recommendations.

8.2.2. Fill Materials

The shallow subsurface clayey soils at the project site are classified as soils with medium expansion potential. They are suitable for reuse as backfill material provided they are free of organic materials, debris and cobbles larger than 3 inches, and compacted within 3% to 5% of the optimum moisture content.

Imported granular soils may be used in the required compacted fills within the subject project site. Imported materials should contain sufficient fines (binder material) to be relatively impermeable and result in a stable subgrade when compacted. The imported materials should also be non-expansive, with an EI less than 35 and free of organic materials, debris and cobbles larger than 3 inches, with no more than 25 percent of materials being larger than 2 inches in size and no more than 25 percent passing #200 Sieve. Within the upper 2 feet of fills the materials should be free of particles greater than 2 inches in size. A bulk sample of potential import material, weighing at least 30 pounds, should be submitted to the Geotechnical Consultant at least 48 hours before fill operations. All proposed import materials should be approved by the Geotechnical Consultant prior to being placed at the site.

8.2.3. Utility Trench Bedding and Backfill

Bedding materials consisting of sand, gravel, or crushed aggregate should be used to backfill around utility pipes to approximately one foot above the top of the pipe. Onsite soils which have a Sand Equivalent (SE) of 30 or greater can also be used as bedding material. Prior to placing the pipes, the pipe trench subgrade should be observed by a representative of the project geotechnical engineer. If the exposed subgrade is loose or unstable, the unsuitable subgrade soil must be excavated and replaced with bedding material. Bedding must be placed uniformly on each side of the pipe and mechanically compacted. Flooding or jetting to densify the bedding materials is not allowed. The fill should be placed in loose lifts not to exceed 8 inches, moisture-conditioned within optimum and 3% above optimum moisture content, and mechanically compacted to at least 90% relative compaction in accordance with ASTM D1557. Thinner lifts may be necessary to achieve the recommended level of compaction of the backfill due to equipment limitations.

8.2.4. Temporary Excavation

Temporary excavations must be properly sloped or shored. Based on the earth materials encountered in our borings, excavation of 5 feet or less in depth may be performed with vertical sidewalls. Deeper excavation up to a depth of 15 feet can be accomplished in accordance with the Occupational Safety and Health Administration (OSHA) requirements for Type B soils, and shall be laid back at 1H:1V gradient.

The contractor is responsible for maintaining the stability of the cuts and personnel safety in the field during construction. All excavations shall be performed in accordance with applicable requirements established by the State, County, or local government. The regulatory requirement may supersede the recommendations presented in this section. The Geotechnical Engineer of Record’s representative should be present during all excavations.

8.2.5. Shoring Design

Typical cantilever shoring up to 20 feet should be designed based on an active fluid pressure of 40 pounds per cubic foot (pcf), assuming level ground above the shoring. If excavations are braced at specific design intervals, the active pressure may then be approximated by a uniform soil pressure distribution with the pressure per foot of width equal to 25H, where H is the depth of the excavation. Surcharge loads within a 1H:1V plane extending up from the base of the excavation should be included in the design lateral pressures by taking 35% of the surcharge pressure applied as a uniform load along the shoring system.

For a soldier beam shoring system, the soldier piles should be spaced at a maximum of 8 feet on-center. For design purposes, the lagging should be designed using a uniform pressure of 300 psf. The passive pressure used to design the soldier pile may be taken as 300 psf per foot of depth. The maximum passive pressure should not be taken more than 3,000 psf. The space between the soil and the soldier beam should be backfilled with concrete with a minimum compressive strength of 2,500 pounds per cubic inch (psi).

All shoring should be designed in accordance with the latest edition of the Caltrans Trenching and Shoring Manual. The geotechnical consultant should review the contractor's shoring design. The shoring design must consider support of the proposed adjacent traffic lanes, parking, structures and/or underground utilities. Also, the effects of shoring deflection on supported pipelines and structures should be considered. Prior to excavating, all adjacent existing structures should be photo documented, and any existing cracks or other distress should be noted. Adjacent structure response should be monitored during excavation. A licensed surveyor should be retained to establish monuments on the shoring and the surrounding ground prior to excavation. Such monuments should be monitored for horizontal and vertical movement during construction. Results of the monitoring program should be provided immediately to the project structural/shoring engineer and Willdan Geotechnical for review and evaluation. It is recommended that Willdan Geotechnical review the shoring plans for conformance with our recommendations and that a Willdan Geotechnical representative observe the installation of shoring.

8.3. FOUNDATION DESIGN

8.3.1. General

It is our opinion that the proposed building may be supported on conventional spread and/or strip footings. As mentioned in Section 3, by the time of preparation of this report, we have not been provided with the order of the anticipated structural loads applicable on the foundations for the proposed structures. The following recommendations are based on the assumption that the imposed column loads will be less than 50 kilo pounds (kips), and imposed continuous footing loads will be less than 5 kips per foot (kpf) for the structure.

8.3.2. Bearing Capacity

Column and strip footings should be at least 24 and 18 inches wide, respectively, and embedded at least 18 inches below the lowest adjacent grade. The footings, supported on structural fill prepared as recommended in Section 8.2.1, may be designed to impose a maximum allowable pressure of 2,500 psf due to dead plus live loads. The bearing capacity may be increased by one-third for transient loads such as seismic or wind.

In order to maintain adequate support for the foundations located adjacent to utility trenches, including existing utility trenches or other footings, the footings should be deepened as necessary so that their bearing surfaces are below an imaginary plane having an inclination of 1H:1V, extending upward from the bottom edge of the adjacent trench or footing.

8.3.3. Resistance to Lateral Loads

Lateral soil resistance will be provided by a combination of frictional resistance between the bottom of the footings and the underlying soils and by passive soil resistance acting against side of the footing. For frictional resistance between concrete and soil, a frictional coefficient of 0.35 may be used. For passive resistance, an allowable fluid pressure of 300 pounds per cubic foot (pcf) may be used for a level ground surface condition in front of the footing. When combining both frictional and passive resistance, the passive resistance should be reduced by one-third. The recommended value for passive resistance may be increased by one-third for short-term loading.

8.3.4. Settlements

Based on the results of our investigation, total settlements due to building loads are expected to be less than one (1) inch, and maximum differential settlements are expected to be of the order of ½ inch over a 50-foot span.

8.3.5. Foundation Setback

All the foundations for the proposed buildings located on or near the descending slopes at the north side of the project site should be setback as recommended in LADBS Information Bulletin P/BC 2017-001. Also, all the recommendations and requirements addressed in Section 1808.7 of CBC 2016 shall be implemented in design and construction of the foundations and buildings on or adjacent to the slopes.

8.4. INTERIOR CONCRETE SLAB-ON-GRADE

Interior concrete slab(s)-on-grade shall be supported on compacted fill, as discussed in Section 8.2.1 of this report. The minimum slab thickness, slab reinforcement, concrete mix design, curing, and control joints shall be determined by the structural engineer. The need for waterproofing shall be determined by the architect/designer.

8.5. CAST-IN-DRILLED-HOLE (CIDH) PILE

8.5.1. Axial Capacity

Allowable downward and uplift capacities for piles with different diameters were evaluated using SHAFT 2012 program and the graphs are presented in Appendix E. The presented graphs are provided for 18 and 24-inch diameter piles, and similar graphs for different diameters other than provided ones will be provided upon request. The capacities are based on frictional resistance of the piles. For frictional pile design using the attached graphs, the weight of the shaft can be assumed to be taken by end-bearing resistance of the pile and it is not necessary to add the weight of the shaft to the structural loads. Uplift capacity of the pile may be assumed as half of the downward capacity of the pile. The actual length of the drilled piles shall be calculated by the structural engineer for the project, considering the recommendations provided herein. The provided capacities are based on the strength of the soils, not the pile section, which should be designed and checked by the project structural engineer.

8.5.2. Pile Spacing Group Efficiency

Piles in group should be spaced at least 3 diameters on centers. For this recommended spacing, there is no reduction in axial capacity. If the spacing is smaller than this value, following group efficiency should be incorporated to obtain the group capacity. The axial load capacity of piles group may be calculated as follows:

$$P_{ag} = \eta \ N \ P_{as}$$

where:

P_{ag} = allowable downward or uplift capacity of pile group
η = group efficiency factor
N = number of piles in group
P_{as} = allowable downward or uplift capacity of a single isolated pile

The group efficiency factor may be calculated using the following formula:

$$0.7 \leq \eta = \frac{2s(m+n)+4B}{\pi mnB} \leq 1.0$$

where:

m = number of rows of piles
n = number of piles per row
B = diameter of a single pile
s = center to center spacing of piles

8.5.3. Lateral Resistance

Lateral loads can be resisted by passive pressure developed against the vertical shafts. The lateral capacity of the pile depends on the permissible deflection and the degree of fixity at the top of the pile. For this project, lateral resistance of a free-head and a fixed-head single pile were evaluated using LPILE 2016 program.

A lateral deflection of ½ inch has been applied to the top of the pile, and the lateral capacity graphs of lateral deflection, shear force and bending moment vs. depth for a 30-feet long pile with 50 kips axial load are presented within Appendix E. The provided capacities are based on the strength of the soils, not the pile section, which should be designed and checked by the project structural engineer.

The presented lateral capacities are for a single pile and do not consider a reduction for group action. Lateral load reduction factors shall be applied when the pile spacing is less than 3 times and 8 times of the pile diameter in normal and parallel to loading direction, respectively. The following Table 4 provides the lateral load reduction factors, to be applied for various pile spacing for piles in line with loading direction.

Table 4. Lateral Load Group Reduction Factors

Center to Center Pile Spacing Line Loading	Group Reduction Factor*
3D	0.70
4D	0.75
5D	0.82
6D	0.88
7D	0.94
8D	1.00

*Ratio of lateral resistance of pile in a group to a single pile

8.5.4. Drilled Pile Installation

The borings for the purpose of site exploratory work were drilled using truck mounted hollow stem auger drilling rig, so it is difficult to evaluate the caving potential. Based on our experience and according to the material encountered, as well as existence of very deep groundwater table, the likelihood of caving is considered low. Precautions should be taken during the drilling operation to minimize caving of the drilled holes. To minimize caving potential, it is recommended to keep pile diameter as small as possible. Other means and methods such as using casing may be employed by contractor when necessary. Experienced contractors shall be retained to install drilled pile foundations. It is necessary to perform continuous observation during piling operation by a project geotechnical engineer's representative.

Piles close to each other shall be drilled and filled with concrete alternately and concrete shall be permitted to set at least 8 hours before drilling an adjacent pile. The drilled hole shall be inspected and filled with concrete as soon as possible. The holes should not be left open overnight. The concrete shall be poured using tremie method.

To evaluate the caving potential of the site soils, we recommend excavating one drilled pile hole to the design tip elevation. The diameter of the hole shall be same as the designed pile diameter. The hole shall be excavated under the geotechnical engineer's observation. The hole then should be left open for a sufficient amount of time to evaluate the long-term caving and raveling potential. The holes shall be backfilled as soon as possible, not to leave them open overnight. If the holes are left open, they shall be secured not to create a safety hazard. The hole could be backfilled with the soils or slurry mix.

8.6. RETAINING WALLS

8.6.1. General

Retaining walls may be designed for active or at-rest lateral soil pressures. Active pressure should be used in computations for a retaining wall which is free to rotate at the top. At-rest pressures should be utilized if the wall is restrained from moving at the top, such as below-grade basement walls. The following recommendations should be followed for design and construction of the retaining walls.

8.6.2. Wall Backfill

The backfill behind the walls should be placed and compacted per recommendations provided in Section 8.2.1 of this report. Retaining wall backfill and typical subdrain details for conditions of native soil, imported sand, or crushed rock are provided in Appendix F.

8.6.3. Lateral Earth Pressure

For design of retaining walls where the surface of the backfill is level, it may be assumed that drained on-site soils will exert a pressure equal to that developed by an equivalent fluid pressure with a density of 40 and 60 pounds per cubic foot (pcf) for active and at-rest conditions, respectively. The recommended lateral pressure may be considered as service loads. A drainage system per details provided in Appendix F, or similarly acceptable product, should be provided behind the walls to reduce the potential for development of hydrostatic pressure. If a drainage system is not installed, the walls should be designed to include a hydrostatic pressure and the combined pressure for a level backfill may be assumed to be equal to that developed by an equivalent fluid with a density of 80 and 90 pcf for active and at-rest conditions, respectively, for the full height of the wall.

When imported gravelly or sandy material is used for backfill behind the retaining wall, the density of equivalent fluid for active pressure may be reduced to 30 and 75 pcf for drained and undrained level backfill, respectively. For imported gravelly or sandy material, the density of equivalent fluid for at-rest pressure may be reduced to 50 and 85 pcf for drained and undrained level backfill, respectively.

In addition to static lateral earth pressure, the walls supporting more than 6 feet of backfill height shall be designed for a seismic lateral pressure equal to that developed by an equivalent fluid pressure with a density of 25 pcf. The seismic pressure may be assumed to act as an equivalent fluid pressure on the wall.

Also, the retaining walls should be designed to resist any lateral surcharges due to the traffic, nearby buildings or construction loads. Surcharge loads within a 1H:1V plane extending up from the base of the wall should be included in the design lateral pressures by taking 35% of the surcharge pressure applied as a uniform load along the height of the wall.

8.6.4. Wall Foundation

Bearing Capacity: The walls may be supported on conventional strip footings. The footings should have a minimum embedment of 18 inches below adjacent lowest finished grade and a minimum width of 2 feet. The wall footings, supported on non-structural fill prepared as recommended in Section 8.2.1, may be designed using a maximum allowable bearing pressure of 1,500 psf. The recommended value may be increased by one-third for short-term loading, such as wind and earthquake.

Resistance to Lateral Loads: Lateral soil resistance will be provided by a combination of frictional resistance between the bottom of the footings and the underlying soils and by passive soil resistance acting against side of the footing. For frictional resistance between concrete and soil, a frictional coefficient of 0.35 may be used. For passive resistance, an allowable fluid pressure of 150 pcf may be used for a level ground surface condition in front of the footing. The first 12 inches of the soil should not be considered in passive resistance. The recommended passive resistance may be increased by one-third for short-term loading. The frictional resistance and the passive resistance may be combined provided that the passive resistance is reduced by one-third.

Settlements: Based on the results of our investigation, total settlements due to wall loads are expected to be less than 1.0 inch, and maximum differential settlements are expected to be of the order of ½ inch over a 50-foot span.

8.7. SURFACE DRAINAGE

Inadequate control of run-off water and/or heavy irrigation after construction of the proposed developments may lead to adverse conditions. Maintaining adequate surface drainage, proper disposal of run-off water, and control of irrigation will help reduce the potential for future moisture related problems and differential movements from soil heave/settlement.

Surface drainage should be carefully taken into consideration during grading, landscaping and building construction. Positive surface drainage should be provided to direct surface water away from wall and toward a suitable drainage device.

8.8. SOIL CORROSIVITY AND SULFATE ATTACK POTENTIAL

Two (2) samples obtained from the borings drilled within the subject project site were tested for pH, minimum resistivity, soluble chloride content and soluble sulfate content. The test results indicate that the onsite soils show moderate sulfate exposure. As such, for concrete in contact with onsite soils, Type II or V Portland cement should be used. The measured resistivity and pH indicate that onsite soils are severely corrosive to buried ferrous metals. Further interpretation of the corrosivity test results and providing corrosion design and construction recommendations are referred to corrosion specialists.

8.9. ON-SITE STORMWATER DISPOSAL

We performed two percolation tests at the site and the test results are provided in Section 5.5 of this report. The test results indicate that the on-site soil has adequate permeability to accommodate onsite infiltration. Furthermore, the historical groundwater table at the site is very deep. As such, the stormwater infiltration at the site is feasible from the geotechnical standpoint.

For design purposes, an infiltration rate of 1.5 inch per hour (in/hr) may be used. The infiltration system shall be designed in accordance with the minimum design requirements, as presented in LADBS Information Bulletin P/BC 2017-118 Guidelines for Storm Water Infiltration. It is our opinion that, if the drainage system is designed in accordance with the LADBS' requirements, infiltration will not result in ground settlement that could adversely impact structures, either on or adjacent to the site. Furthermore, infiltration is not expected to result in soil saturation that could adversely impact retaining walls and/or basements.

8.10. PAVEMENT DESIGN

Pavement sections have been designed in accordance with the procedures presented in Caltrans Highway Design Manual (HDM). Laboratory testing of a bulk sample from the shallow subsurface soil of the proposed pavement area indicates a minimum R-value of 52, however, according to the HDM's recommendation an R-value of 50 has been used for design. A flexible section consisting of asphalt concrete (AC) over aggregate base (AB), or a full-depth AC section may be used. The pavement sections listed in Table 5 have been developed for a range of traffic index (TI) values.

Table 5. Flexible Pavement Design

TI	AC/AB (in/in)	Full Depth AC (in)
4	2.5/4.5	3.5
5	3.0/4.5	4.5
6	3.5/4.5	5.5

The pavement section shall be supported on the subgrade prepared per recommendations of Section 13.0 of this report. The base material shall consist of crushed aggregate base (CAB) or crushed miscellaneous base (CMB) as specified in the Greenbook, and compacted to a minimum of 95% of maximum dry density.

8.11. REVIEW OF CONSTRUCTION PLANS

Recommendations contained in this report are based on preliminary plans. The geotechnical consultant should review the final construction plans and specifications in order to confirm that the general intent of the recommendations contained in this report have been implemented into the final construction documents. Recommendations contained in this report may require modification or additional recommendations may be necessary based on the final design.

8.12. GEOTECHNICAL OBSERVATION AND TESTING

It is recommended that inspection and testing be performed by the geotechnical consultant during the following stages of construction:

- Grading operations, including over-excavation and placement of compacted fill;
- Observation of foundation excavation;
- Retaining wall footing excavation and subdrain installations;
- Excavations and backfilling for retaining walls and utility trenches; and
- When any unusual subsurface conditions are encountered.

9. CLOSURE

This report is intended for the use by Geotechnical Engineering Group and its consultants for design and construction associated with the proposed Boyle Heights Sports Center project located in Los Angeles, California, as shown on Figure 1, Site Location Map.

The findings and recommendations contained in this report are based on the results of the field investigation, laboratory tests, and engineering analyses, combined with an extrapolation of subsurface conditions between and beyond the boring locations.

Services performed by Willdan Geotechnical have been conducted in accordance with generally accepted professional geotechnical engineering principles and practices at this time. No other representation, express or implied, and no warranty or guarantee is included or intended.

10. REFERENCES

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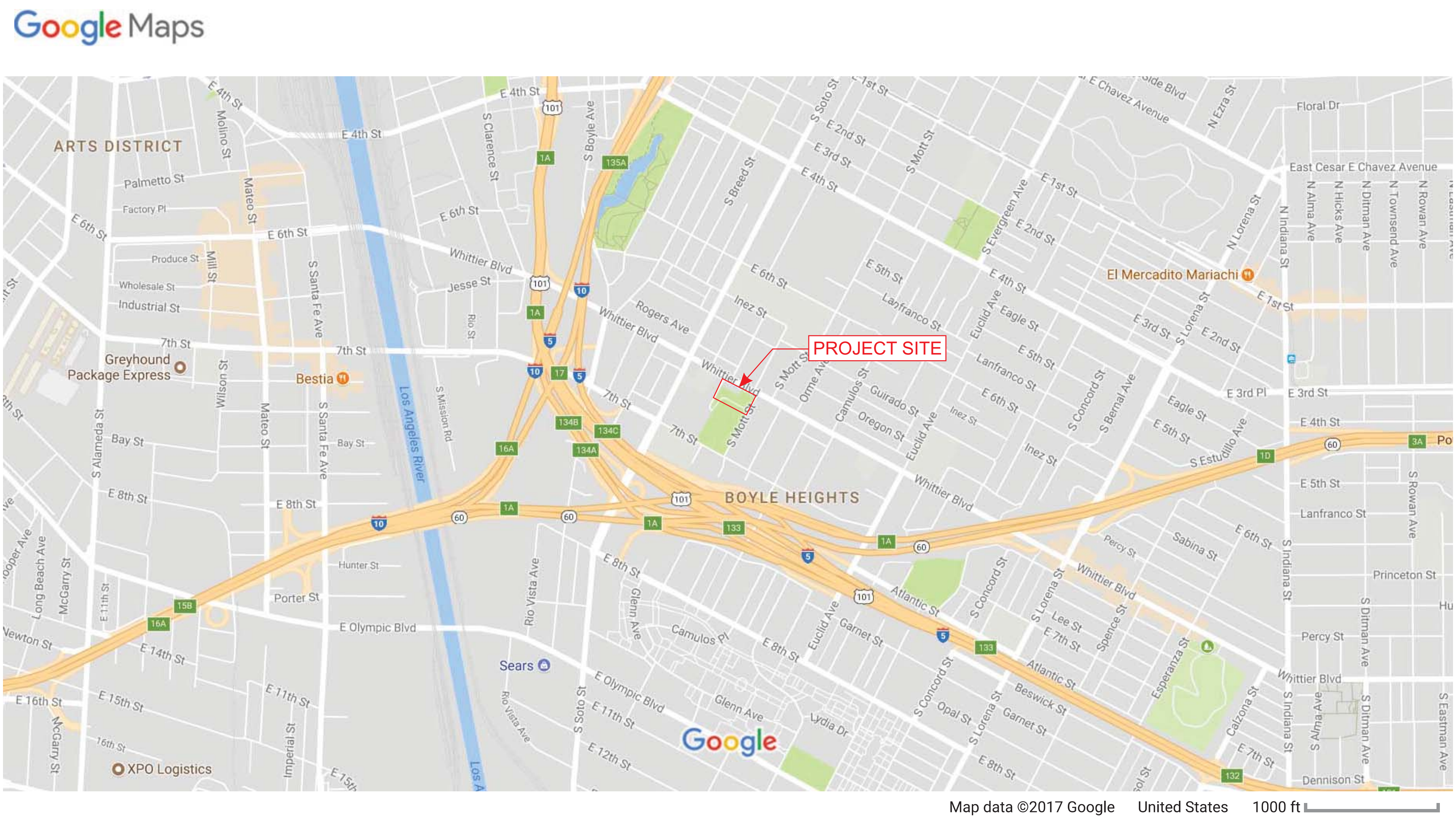
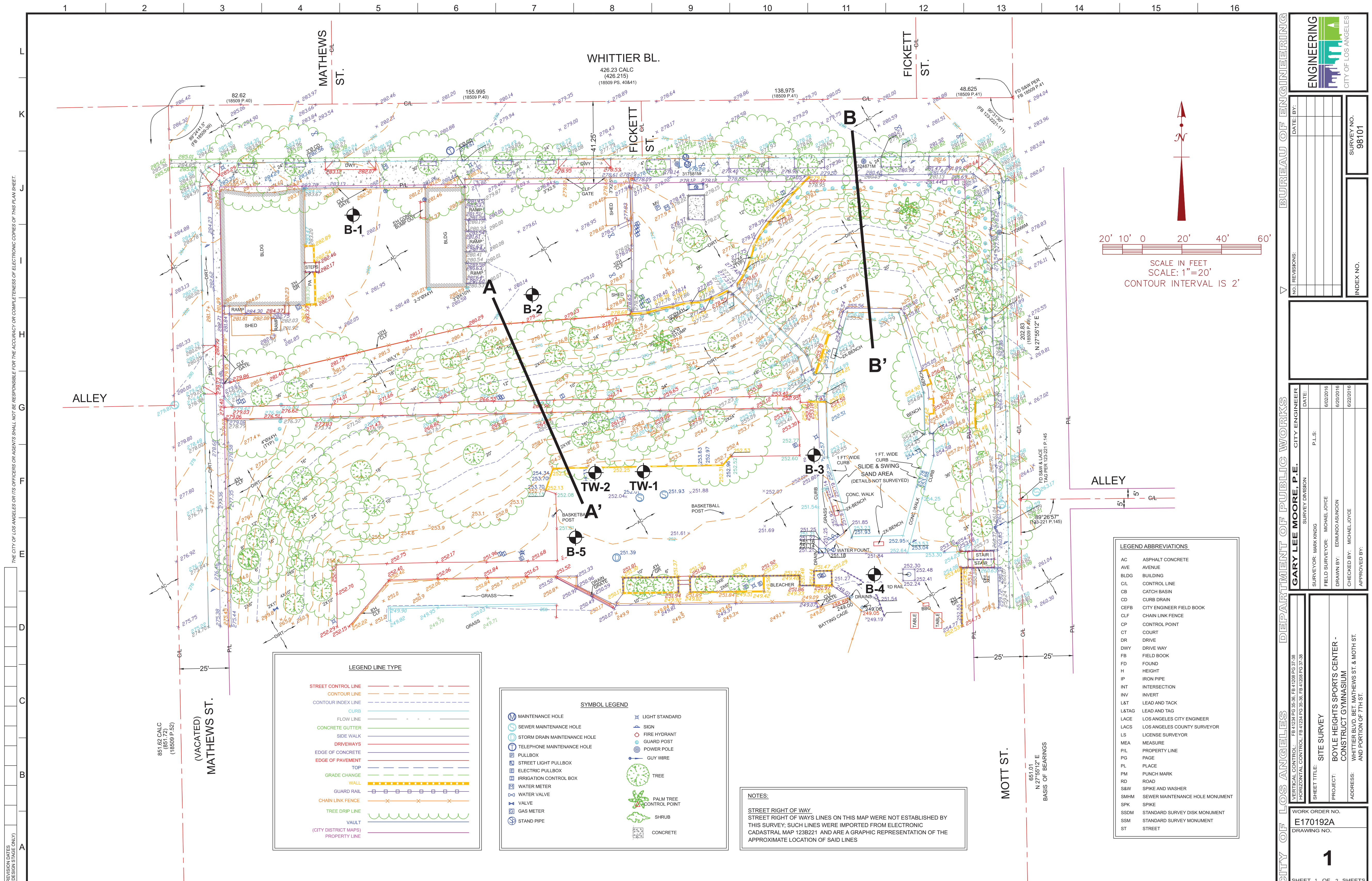


FIGURE 1. SITE LOCATION MAP	WILLDAN Geotechnical *extending your reach*
BOYLE HEIGHTS SPORTS CENTER	Drawn By: MR Date: 08-01-2017
LOS ANGELES, CALIFORNIA	Approved By: MR Project No.: 106965-2000



B-5 Approximate Boring Location

TW-2 Approximate Percolation Test Location

B——B' Approximate Cross Section Location

FIGURE 2. BORING, PERCOLATION TEST & CROSS SECTION LOCATION PLAN

BOYLE HEIGHTS SPORTS CENTER
LOS ANGELES, CALIFORNIA

WILLDAN Geotechnical extending your reach

Drawn By: MR Date: 09-28-2017

Approved By: MR Project No.: 106965-2000

APPENDIX A. BORING LOGS

MAJOR DIVISIONS			SYMBOLS		TYPICAL NAMES
COARSE GRAINED SOILS Half is larger than no. 200 sieve	GRAVELS More than half coarse fraction is larger than no. 4 sieve	Clean gravels with little or no fines	GW		Well graded gravels, gravel-sand mixtures
			GP		Poorly graded gravels, gravel-sand mixtures
		Gravels with over 12% fines	GM		Silty gravels, poorly graded gravel-sand-silt mixtures
			GC		Clayey gravels, poorly graded gravel-sand-clay mixtures
	SANDS More than half coarse fraction is smaller than no. 4 sieve	Clean sands with little or no fines	SW		Well graded sands, gravelly sands
			SP		Poorly graded sands, gravelly sands
		Sands with over 12% fines	SM		Silty sands, poorly graded sand-silt mixtures
			SC		Clayey sands, poorly graded sand-clay mixtures
FINE GRAINED SOILS Half is smaller than no. 200	SILTS AND CLAYS Liquid limit less than 50		ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL		Organic clays and organic silty clays of low plasticity
	SILTS AND CLAYS Liquid limit greater than 50		MH		Inorganic silts, micaceous or diatomaceous fine, sandy or silty soils, elastic silts
			CH		Inorganic clays of high plasticity, fat clays
			OH		Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			Pt		Peat and other highly organic soils

GRANULAR SOILS

RELATIVE DENSITY	BLOWS/FOOT* SPT	CD
VERY LOOSE	0 - 4	0 - 8
LOOSE	5 - 10	9 - 18
MEDIUM DENSE	11 - 30	19 - 54
DENSE	31 - 50	55 - 90
VERY DENSE	OVER 50	OVER 90

FINE-GRAINED SOILS

CONSISTENCY	BLOWS/FOOT* SPT	CD
SOFT	0 - 4	0 - 4
FIRM	5 - 8	5 - 9
STIFF	9 - 15	10- 18
VERY STIFF	16 - 30	19 - 39
HARD	OVER 30	OVER 39

*Conversion between California Drive (CD) and Standard Penetration Test (SPT) blow count has been calculated using "Foundation Engineering Hand Book" by H.Y. Fang.

STANDARD PENETRATION TEST SAMPLE
Split Barrel sampler in accordance with

MODIFIED CALIFORNIA SAMPLE
2.416" inside diameter

SHELBY TUBE SAMPLE

BULK SAMPLE

WATER TABLE

TEST TYPE Results shown in Appendix B	OTHER
Corrosion Analysis	CA
Sieve Analysis	SA
Unconfined Compression	UC
Hydrometer Analysis	HA
Expansion Index	EI
California Bearing Ratio	CBR
% Passing #200 Sieve	W
Pocket Penetrometer	PP
Direct Shear	DS
Direct Shear (Remolded)	DS_R
Atterberg Limits	AL
Consolidation	CN
Consolidation (Remolded)	CN_R
R-Value	R
Undrained-Unconsolidated Shear	UU
Maximum Density Curve	Max

EXPLORATION LOG KEY

BORING LOG B-1

Borehole Location: **See Figure 2**	Approximate Grade Elevation:	Sheet **1** of **2**
Borehole Coordinates: **34.0335N 118.2141W**	Date Started: **06/28/17**	Date Finished: **06/28/17**
Drilling Equipment: **CME 75**	Total Depth: **36.5 ft**	Depth to Groundwater: **GW Not Encountered.**
Drilling Method: **Hollow Stem Auger Boring**	Borehole Diameter: **8"**	
Driller: **Choice Drilling, Inc.**	Logged By: **RC**	Checked By: **AM**
Hammer Information: **140 lb and 30" Drop Height**		

Elevation (ft)	Depth (ft)	Lithology	Description	Remarks	Sampler	Number	Blows/6"	Moisture Content (%)	Dry Density (pcf)	Additional Tests
	0		2.5" Asphalt							
			Sandy Lean CLAY (CL), stiff, brown, moist							
				PID=2.7 ppm		Bulk 1 S-1	5/5/7			SA, AL, EI, CN_R, DS_R, Max, CA
	5		very stiff	PID=2.5 ppm		R-2	6/9/12	5.2	91	CN, DS
			Clayey SAND/Silty SAND (SC/SM), medium dense, brown, moist	PID=3.0 ppm		S-3	5/5/6			
	10		Silty SAND (SM), dense, brown, moist	PID=5.1 ppm		R-4	20/32/40	6.7	112	DS
			very dense	PID=5.0 ppm		S-5	29/50/(6")			
	15			PID=3.0 ppm		R-6	50/(6")			
				PID=1.4 ppm		S-7	28/50/(5")			
	20			PID=5.0 ppm		R-8	50/(6")			
	25			PID=2.8 ppm		S-9	50/(6")			

BORING LOG B-1

Borehole Location: **See Figure 2**	Approximate Grade Elevation:	Sheet **2** of **2**
Borehole Coordinates: **34.0335N 118.2141W**	Date Started: **06/28/17**	Date Finished: **06/28/17**
Drilling Equipment: **CME 75**	Total Depth: **36.5 ft**	Depth to Groundwater: **GW Not Encountered.**
Drilling Method: **Hollow Stem Auger Boring**	Borehole Diameter: **8"**	
Driller: **Choice Drilling, Inc.**	Logged By: **RC**	Checked By: **AM**
Hammer Information: **140 lb and 30" Drop Height**		

Elevation (ft)	Depth (ft)	Lithology	Description	Remarks	Sampler	Number	Blows/6"	Moisture Content (%)	Dry Density (pcf)	Additional Tests
	30		Silty SAND (SM), very dense, brown, moist	PID=3.2 ppm		R-10	50/(6")			
	35		Sandy SILT/Sandy CLAY (ML/CL), hard, brown, moist	PID=3.3 ppm		S-11	12/16/24			
			Total Depth 36.5 ft GW Not Encountered. Backfilled with Excavated Spoils and Patched with Cold Asphalt.							
	40									
	45									
	50									
	55									

BORING LOG B-2

Borehole Location: **See Figure 2**	Approximate Grade Elevation:	Sheet **1** of **2**
Borehole Coordinates: **34.0332N 118.2139W**	Date Started: **06/28/17**	Date Finished: **06/28/17**
Drilling Equipment: **CME 75**	Total Depth: **35.5 ft**	Depth to Groundwater: **GW Not Encountered.**
Drilling Method: **Hollow Stem Auger Boring**	Borehole Diameter: **8"**	
Driller: **Choice Drilling, Inc.**	Logged By: **RC**	Checked By: **AM**
Hammer Information: **140 lb and 30" Drop Height**		

Elevation (ft)	Depth (ft)	Lithology	Description	Remarks	Sampler	Number	Blows/6"	Moisture Content (%)	Dry Density (pcf)	Additional Tests
	0		2" Asphalt							
			Clayey SAND (SC), dense, dark brown, moist	PID=1.3 ppm		Bulk 1 R-1	20/27/35	11.5	120	SA, Max, R
	5		Silty SAND with Gravel (SM), medium dense, brown, moist	PID=2.0 ppm		S-2	10/12/15			
				PID=1.5 ppm		R-3	13/16/25	4.4	94	DS
	10			PID=1.9 ppm		S-4	5/6/9			
			Silty GRAVEL with Sand (GM), very dense, brown, moist	PID=0.5 ppm		R-5	19/50/(6")	4	105	SA
	15			PID=1.9 ppm		S-6	50/(6")			
			Silty Sand with Gravel (SM), very dense, brown, moist (disturbed sample)	PID=1.7 ppm		R-7	25/50/(6")			
	20			PID=0.2 ppm		S-8	30/50/(5")			
	25		Sandy CLAY/Clayey SAND (CL/SC), hard/dense, brown, moist	PID=0.5 ppm		R-9	10/28/45	8.7	116	DS

BORING LOG B-2

Borehole Location: **See Figure 2**	Approximate Grade Elevation:	Sheet **2** of **2**
Borehole Coordinates: **34.0332N 118.2139W**	Date Started: **06/28/17**	Date Finished: **06/28/17**
Drilling Equipment: **CME 75**	Total Depth: **35.5 ft**	Depth to Groundwater: **GW Not Encountered.**
Drilling Method: **Hollow Stem Auger Boring**	Borehole Diameter: **8"**	
Driller: **Choice Drilling, Inc.**	Logged By: **RC**	Checked By: **AM**
Hammer Information: **140 lb and 30" Drop Height**		

Elevation (ft)	Depth (ft)	Lithology	Description	Remarks	Sampler	Number	Blows/6"	Moisture Content (%)	Dry Density (pcf)	Additional Tests
	30		SILT (ML), hard, brown, moist	PID=0.8 ppm		S-10	13/17/23			
	35		Sandy SILT/Silty SAND (ML/SM), hard/very dense, brown, moist	PID=3.1 ppm		R-11	50/(6")			
			Total Depth 35.5 ft GW Not Encountered. Backfilled with Excavated Spoils and Patched with Cold Asphalt.							
	40									
	45									
	50									
	55									

BORING LOG B-3

Borehole Location: **See Figure 2**	Approximate Grade Elevation:	Sheet **1** of **1**
Borehole Coordinates: **34.0329N 118.2137W**	Date Started: **06/28/17**	Date Finished: **06/28/17**
Drilling Equipment: **CME 75**	Total Depth: **26.5 ft**	Depth to Groundwater: **GW Not Encountered.**
Drilling Method: **Hollow Stem Auger Boring**	Borehole Diameter: **8"**	
Driller: **Choice Drilling, Inc.**	Logged By: **RC**	Checked By: **AM**
Hammer Information: **140 lb and 30" Drop Height**		

Elevation (ft)	Depth (ft)	Lithology	Description	Remarks	Sampler	Number	Blows/6"	Moisture Content (%)	Dry Density (pcf)	Additional Tests
	0		Clayey SAND (SC), medium dense, light brown, moist							
				PID=2.2 ppm		Bulk 1 S-1	7/5/8			
	5			PID=5.5 ppm		R-2	12/18/30	9.3	88	
				PID=2.9 ppm		S-3	8/9/11			
	10		Silty SAND (SM), very dense, brown, moist	PID=3.0 ppm		R-4	18/38/50(4")	2.5	112	SA
				PID=3.1 ppm		S-5	13/28/50			
	15			PID=3.9 ppm		R-6	43/50(5")	4.6	104	
			dense, light gray	PID=4.4 ppm		S-7	11/15/21			
	20		SILT/Silty SAND (ML/SM), hard/very dense, light gray, moist	PID=4.3 ppm		R-8	29/50(6")	9.5	98	
	25		SILT (ML), hard, light gray, moist	PID=0.3 ppm		S-9	18/26/30			
			Total Depth 26.5 ft GW Not Encountered. Backfilled with Excavated Spoils.							

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FIGURE A-4

BORING LOG B-4

Borehole Location: **See Figure 2**	Approximate Grade Elevation:	Sheet **1** of **1**
Borehole Coordinates: **34.0327N 118.2136W**	Date Started: **06/28/17**	Date Finished: **06/28/17**
Drilling Equipment: **CME 75**	Total Depth: **26.0 ft**	Depth to Groundwater: **GW Not Encountered.**
Drilling Method: **Hollow Stem Auger Boring**	Borehole Diameter: **8"**	
Driller: **Choice Drilling, Inc.**	Logged By: **RC**	Checked By: **AM**
Hammer Information: **140 lb and 30" Drop Height**		

Elevation (ft)	Depth (ft)	Lithology	Description	Remarks	Sampler	Number	Blows/6"	Moisture Content (%)	Dry Density (pcf)	Additional Tests
	0		5" Asphalt							
			Clayey SAND (SC), medium dense, brown, moist							
			hand augered to 5 feet and no sample was collected at 2.5 feet			Bulk 1				W, AL, Max, CA
	5			PID=2.4 ppm		S-1	4/5/7			
				PID=0.2 ppm		R-2	15/25/28	5.9	114	
	10		dense	PID=1.8 ppm		S-3	15/19/28			
			Silty SAND (SM), very dense, brown, moist	PID=1.2 ppm		R-4	35/50(3")			
	15			PID=3.1 ppm		S-5	37/50(5")			
				PID=4.7 ppm		R-6	50/(3")			
	20			PID=4.7 ppm		S-7	41/50(6")			
	25			PID=4.0 ppm		R-8	38/50(5")			
			Total Depth 26.0 ft GW Not Encountered. Backfilled with Excavated Spoils.							

BORING LOG B-5

Borehole Location:	**See Figure 2**	Approximate Grade Elevation:		Sheet **1** of **1**	
Borehole Coordinates:	**34.033N 118.2141W**	Date Started:	**06/28/17**	Date Finished:	**06/28/17**
Drilling Equipment:	**CME 75**	Total Depth:	**26.5 ft**	Depth to Groundwater:	**GW Not Encountered.**
Drilling Method:	**Hollow Stem Auger Boring**	Borehole Diameter:	**8"**		
Driller:	**Choice Drilling, Inc.**	Logged By:	**RC**	Checked By:	**AM**
Hammer Information:	**140 lb and 30" Drop Height**				

Elevation (ft)	Depth (ft)	Lithology	Description	Remarks	Sampler	Number	Blows/6"	Moisture Content (%)	Dry Density (pcf)	Additional Tests
	0		Clayey SAND (SC), medium dense, brown, moist							
				PID=20.0 ppm		Bulk 1 S-1	6/7/10			
	5		Silty SAND with Gravel (SM), very dense, brown, moist	PID=1.0 ppm		R-2	27/50/(6")	3.1	114	
				PID=9.0 ppm		S-3	18/30/40			
	10			PID=>200 ppm		R-4	38/50/(5")	8.7	107	
			dense, light gray	PID=6.9 ppm		S-5	13/16/28			
	15		SILT/ Silty SAND (ML/SM), hard/very dense, light gray, moist	PID=4.3 ppm		R-6	27/50/(6")	9	96	
				PID=3.7 ppm		S-7	20/37/41			
	20		SILT (ML), hard, reddish brown, very moist	PID=5.5 ppm		R-8	21/30/35	28.4	95	
	25		gray to reddish brown	PID=3.2 ppm		S-9	13/16/32			
			Total Depth 26.5 ft GW Not Encountered. Backfilled with Excavated Spoils. (City of Los Angeles representative was notified about high VOC concentration at 10 feet)							

Boyle Heights Sports Center Project
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FIGURE A-6

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BORING LOG TW-1

Borehole Location: **See Figure 2**	Approximate Grade Elevation:	Sheet **1** of **1**
Borehole Coordinates: **34.033N 118.2139W**	Date Started: **06/28/17**	Date Finished: **06/28/17**
Drilling Equipment: **CME 75**	Total Depth: **5.0 ft**	Depth to Groundwater: **GW Not Encountered.**
Drilling Method: **Hollow Stem Auger Boring**	Borehole Diameter: **8"**	
Driller: **Choice Drilling, Inc.**	Logged By: **RC**	Checked By: **AM**
Hammer Information: **140 lb and 30" Drop Height**		

Elevation (ft)	Depth (ft)	Lithology	Description	Remarks	Sampler	Number	Blows/6"	Moisture Content (%)	Dry Density (pcf)	Additional Tests
	0		6" Asphalt over 4.5" Aggregate Base							
			Clayey SAND (SC), dense, brown, moist			Bulk 1				SA, AL
				PID=2.5 ppm		R-1	19/21/36	6.6	126	CN
	5		Total Depth 5.0 ft GW Not Encountered. Backfilled with Excavated Spoils and Patched with Cold Asphalt.							
	10									
	15									
	20									
	25									

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FIGURE A-7

BORING LOG TW-2

Borehole Location: **See Figure 2**	Approximate Grade Elevation:	Sheet **1** of **1**
Borehole Coordinates: **34.03303N 118.21398W**	Date Started: **06/28/17**	Date Finished: **06/28/17**
Drilling Equipment: **CME 75**	Total Depth: **10.0 ft**	Depth to Groundwater: **GW Not Encountered.**
Drilling Method: **Hollow Stem Auger Boring**	Borehole Diameter: **8"**	
Driller: **Choice Drilling, Inc.**	Logged By: **RC**	Checked By: **AM**
Hammer Information: **140 lb and 30" Drop Height**		

Elevation (ft)	Depth (ft)	Lithology	Description	Remarks	Sampler	Number	Blows/6"	Moisture Content (%)	Dry Density (pcf)	Additional Tests
	0		6" Asphalt/ 4.5" Aggregate Base							
			Clayey SAND (SC), brown, moist			Bulk 1				
	5									
			Silty SAND (SM), very dense, light brown, moist	PID=0.6 ppm		Bulk 2 R-1	50(6")			
	10		Total Depth 10.0 ft GW Not Encountered. Backfilled with Excavated Spoils and Patched with Cold Asphalt.							
	15									
	20									
	25									

WILLDAN Geotechnical

Boyle Heights Sports Center Project
Los Angeles, California

Project Number: **106965-2000**

FIGURE A-8

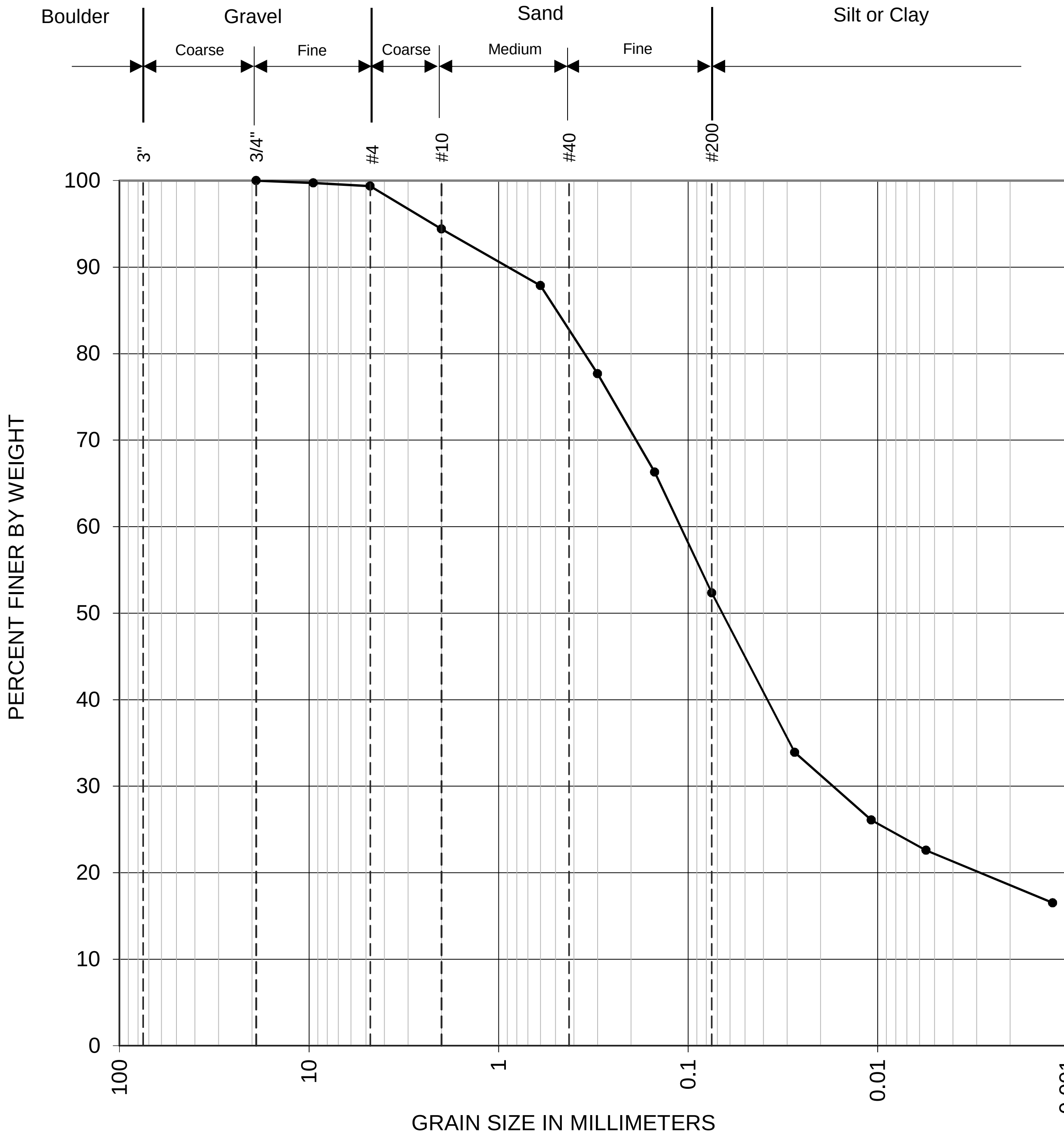
APPENDIX B. LABORATORY TEST RESULTS

TABLE B-1. SUMMARY OF LABORATORY TEST RESULTS

Boyle Heights Sports Center Project, Los Angeles, California

Willdan Geotechnical Project No. 106965-2000

Sample		USCS Soil Description	Gradation (ASTM D422)	Passing #200 (ASTM D1140)	Atterberg Limits (ASTM D4318)		Expansion Index (ASTM D 4829)	R-Value (CTM 301)	Direct Shear (ASTM D3080)				Consolidation (ASTM D2435)			Compaction (ASTM D2435)		Corrosivity (CTM 422, 417, 643)			
									Peak		Ultimate										
Boring No.	Depth (ft)		(% G : S : F)	(% F)	Liquid Limit	Plasticity Index			c (psf)	ϕ (^{o})	c (psf)	ϕ (^{o})	P_C (ksf)	C_C	C_S	Max Dry Density (pcf)	Opt. Moisture (%)	pH	Soluble Sulfate (ppm)	Soluble Chloride (ppm)	Minimum Resistivity (ohm-cm)
									Remolded to 90% RC				Remolded to 90% RC								
B-1	1.0 - 5.0	Sandy Lean CLAY (CL)	1 : 47 : 52		32	19	55		125	29.0	125	29.0	3.10	0.135	0.010	124.9	9.9	8.02	150	180	1776
B-1	5.0	Sandy Lean CLAY (CL)							5	29.5	5	29.0	1.60	0.095	0.021						
B-1	10.0	Silty SAND (SM)							145	32.0	5	32.0									
B-2	1.0 - 5.0	Clayey SAND (SC)	5 : 61 : 34					52								131.1	9.7				
B-2	7.5	Silty SAND with Gravel (SM)							10	30.5	5	30.0									
B-2	12.5	Silty GRAVEL with Sand (GM)	37 : 32 : 31																		
B-2	25.0	Sandy CLAY/Clayey SAND (CL/SC)							390	28.5	225	27.5									
B-3	12.5	Silty SAND (SM)	5 : 82 : 13																		
B-4	1.0 - 5.0	Clayey SAND (SC)		37	24	9										131.5	8.0	7.39	330	240	1523
TW-1	1.0 - 5.0	Clayey SAND (SC)	8 : 60 : 32		25	9															
TW-1	3.5	Clayey SAND (SC)											1.60	0.083	0.016						



Boring No.	Sample No.	Depth	USCS Symbol	Classification	Natural W %	LL	PL	PI
B-1		1.0' - 5.0'	CL	Sandy Lean CLAY				

% +3"	% Gravel	% Sand	% Fines
0	1	47	52

C_u	C_c

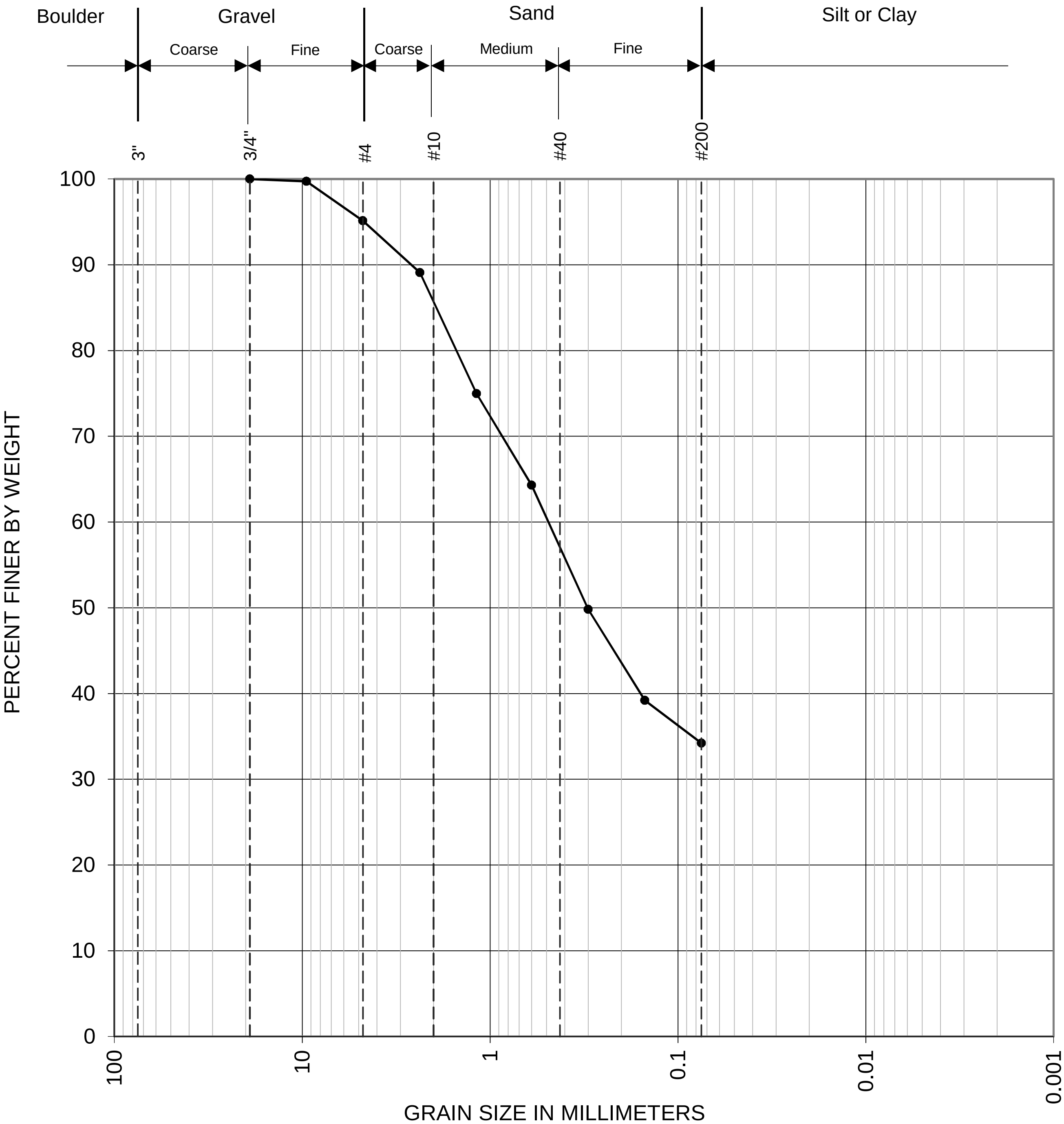
Project Name: Boyle Heights Sports Center

Project No.: 106965-2000

PARTICLE SIZE CURVE

(ASTM D422)

WILLDAN Geotechnical | extending your reach



Boring No.	Sample No.	Depth	USCS Symbol	Classification	Natural W %	LL	PL	PI
B-2		1.0' - 5.0'	SC	Clayey SAND				

% +3"	% Gravel	% Sand	% Fines
0	5	61	34

C_u	C_c

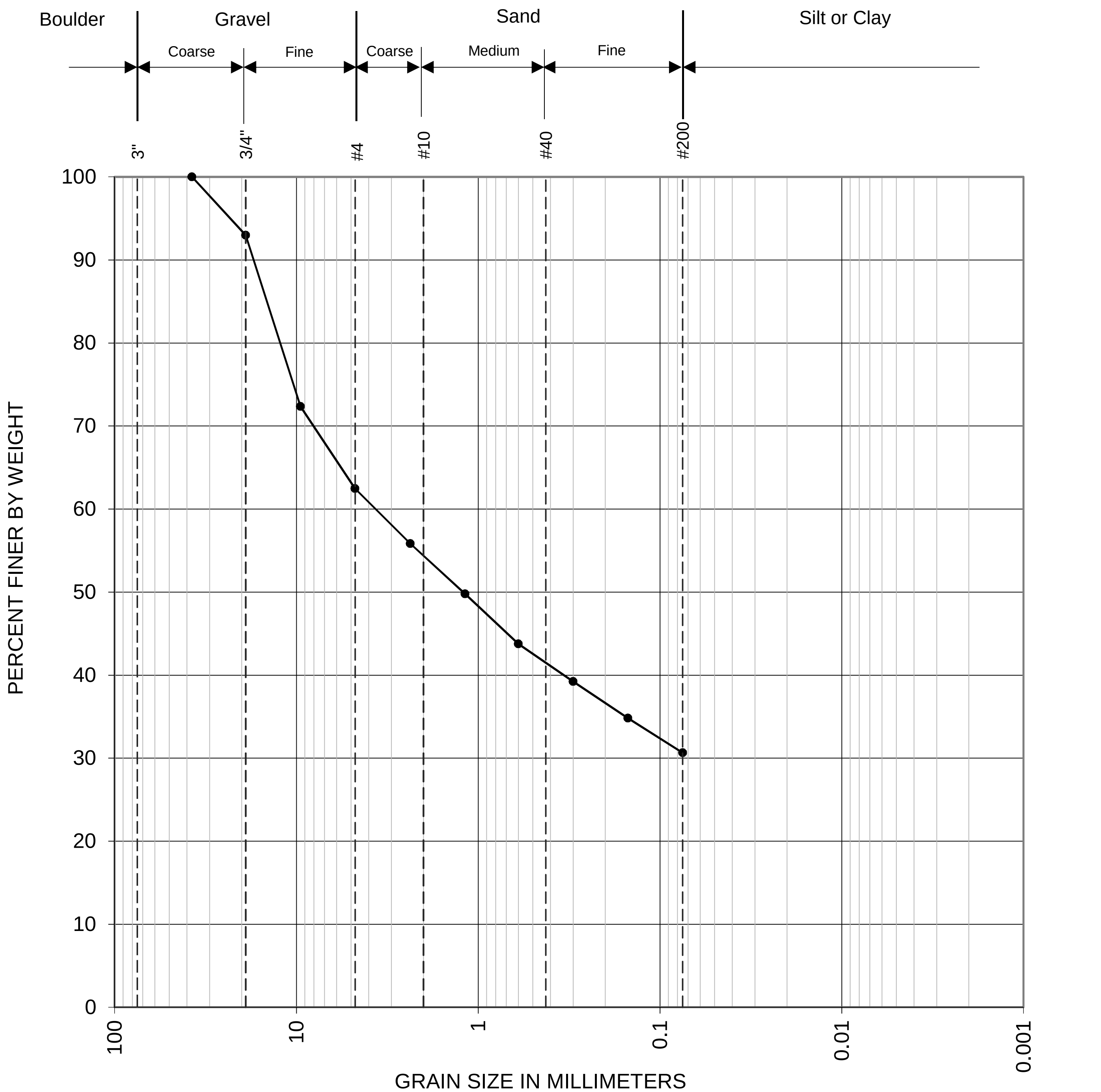
Project Name: Boyle Heights Sports Center

Project No.: 106965-2000

PARTICLE SIZE CURVE

(ASTM D422)

WILLDAN Geotechnical | extending your reach



Boring No.	Sample No.	Depth	USCS Symbol	Classification	Natural W %	LL	PL	PI
B-2		12.5'	GM	Silty GRAVEL with Sand				

% +3"	% Gravel	% Sand	% Fines
0	37	32	31

C_u	C_c

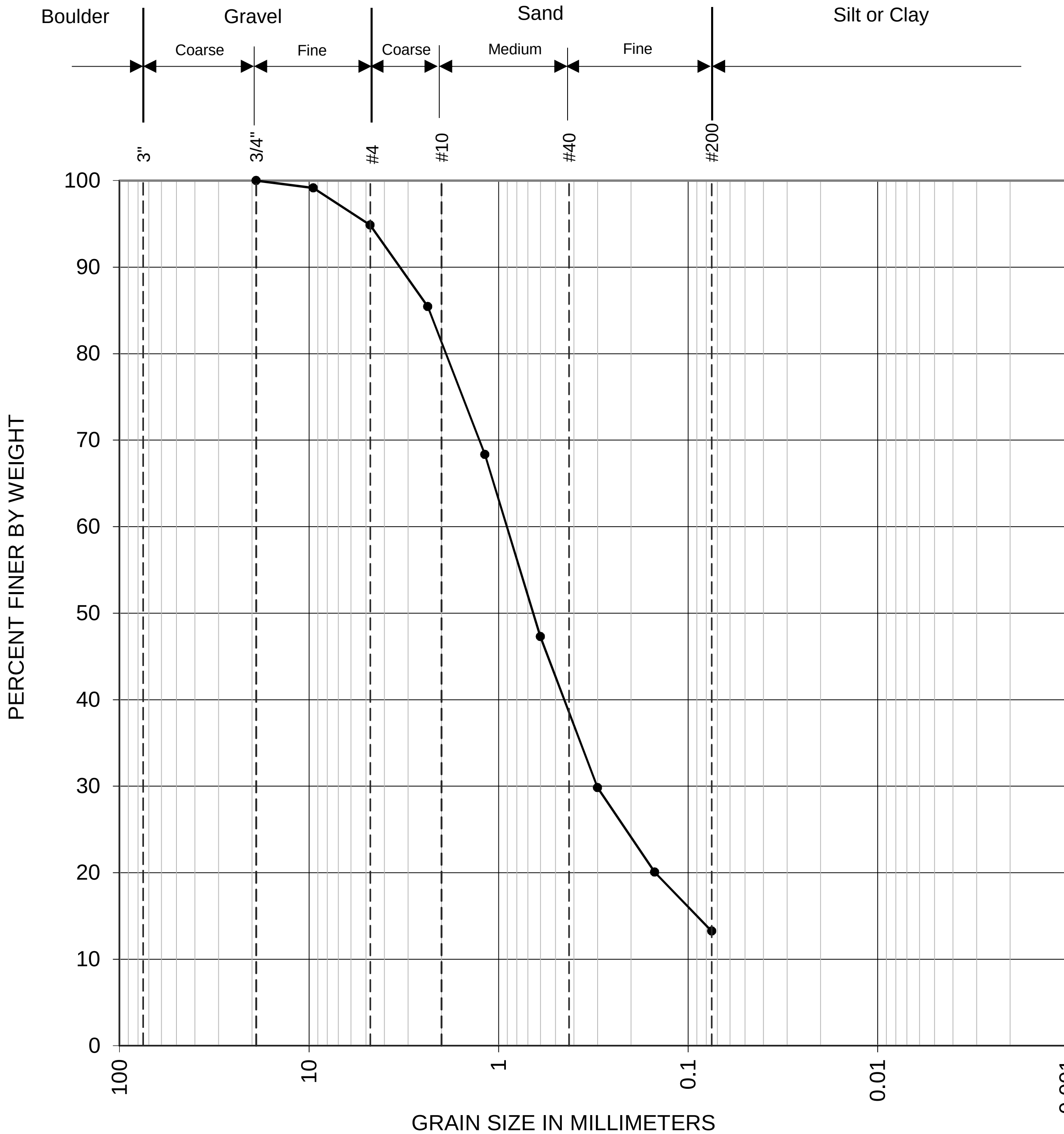
Project Name: Boyle Heights Sports Center

Project No.: 106965-2000

PARTICLE SIZE CURVE

(ASTM D422)

WILLDAN Geotechnical | extending your reach



Boring No.	Sample No.	Depth	USCS Symbol	Classification	Natural W %	LL	PL	PI
B-3		12.5'	SM	Silty SAND				

% +3"	% Gravel	% Sand	% Fines
0	5	82	13

C_u	C_c

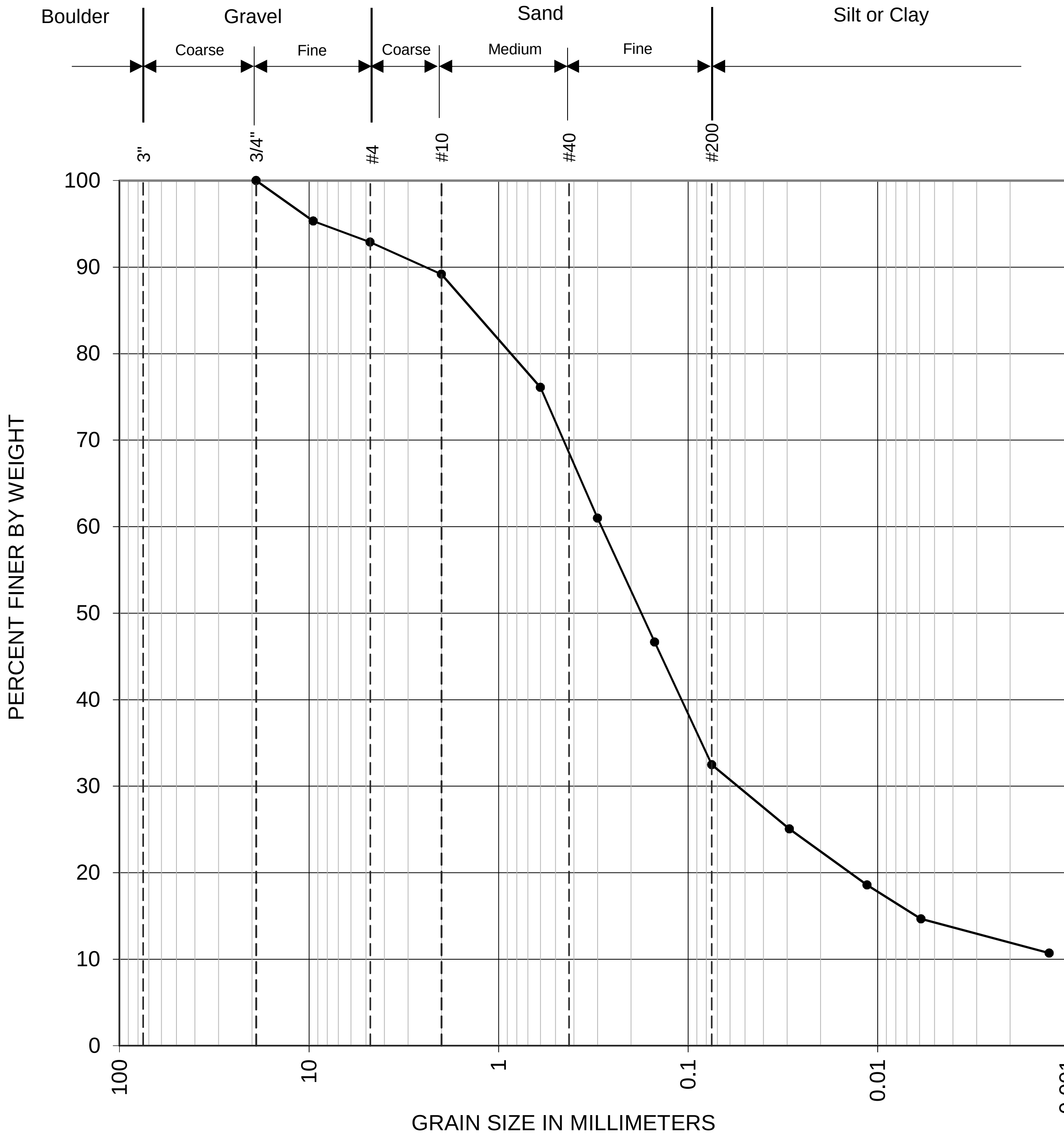
Project Name: Boyle Heights Sports Center

Project No.: 106965-2000

PARTICLE SIZE CURVE

(ASTM D422)

WILLDAN Geotechnical | extending your reach



Boring No.	Sample No.	Depth	USCS Symbol	Classification	Natural W %	LL	PL	PI
TW-1		1.0' - 5.0'	SC	Clayey SAND				

% +3"	% Gravel	% Sand	% Fines
0	8	60	32

C_u	C_c

Project Name: Boyle Heights Sports Center

Project No.: 106965-2000

PARTICLE SIZE CURVE

(ASTM D422)

WILLDAN Geotechnical

extending your reach

Project Name :	Boyle Heights Sports Center	Project No.:	106965-2000
Sample Location / Source :	B-1	Tested by :	RMC	Date: 7/26/2017
Sample Depth / No. :	1' - 5'	Sampled by:		Date:
Sample Description / Classification :	Sandy Lean CLAY (CL)		

PLASTIC LIMIT

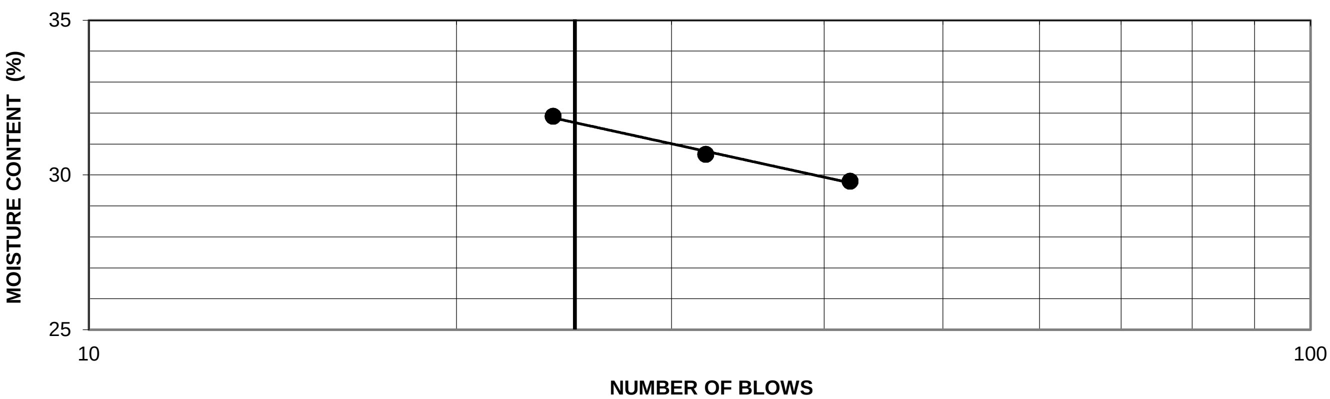
DETERMINATION NO.		1	
DISH NO.		7	
MASS, DISH + WET SOIL	(g)	39.85	
MASS, DISH + DRY SOIL	(g)	38.32	
MASS OF WATER	(g)	1.53	
MASS OF DISH	(g)	26.3	
MASS OF DRY SOIL	(g)	12.02	
MOISTURE CONTENT	(%)	12.7	

LIQUID LIMIT

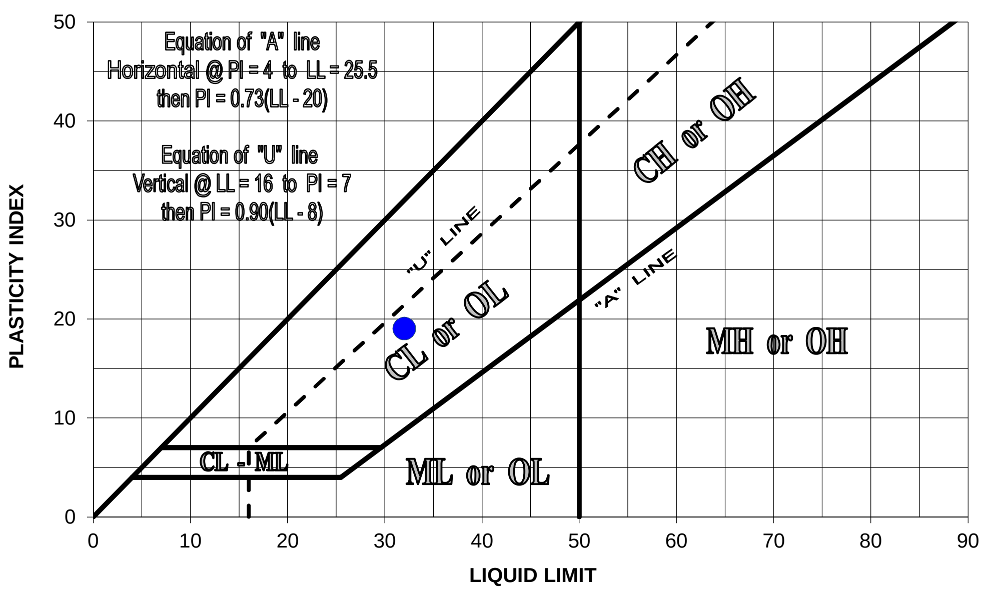
DETERMINATION NO.		1	2	3
DISH NO.		23	9	22
MASS, DISH + WET SOIL	(g)	37.78	34.50	39.75
MASS, DISH + DRY SOIL	(g)	34.38	31.36	36.38
MASS OF WATER	(g)	3.40	3.14	3.37
MASS OF DISH	(g)	23.72	21.12	25.07
MOISTURE CONTENT	(%)	31.9	30.7	29.8
NUMBER OF BLOWS		24	32	42

NATURAL MOISTURE CONTENT,%

FLOW CURVE



PLASTICITY CHART



RESULT SUMMARY

NATURAL MOISTURE CONTENT, (%)	
LIQUID LIMIT (LL)	32
PLASTIC LIMIT (PL)	13
PLASTICITY INDEX (PI)	19
SYMBOL FROM PLASTICITY CHART	CL

METHOD OF PREPARATION		METHOD OF LL DETERMINATION	
DRY	X	MULTIPOINT	X
WET		ONE-POINT	

REMARKS :

ATTERBERG LIMITS

(ASTM D4318)

WILLDAN Geotechnical | *extending your reach*

Project Name :	Boyle Heights Sports Center	Project No.:	106965-2000		
Sample Location / Source :	B-4	Tested by :	RMC	Date:	7/26/2017
Sample Depth / No. :	1' - 5'	Sampled by:		Date:	
Sample Description / Classification :	Clayey SAND (SC)				

PLASTIC LIMIT

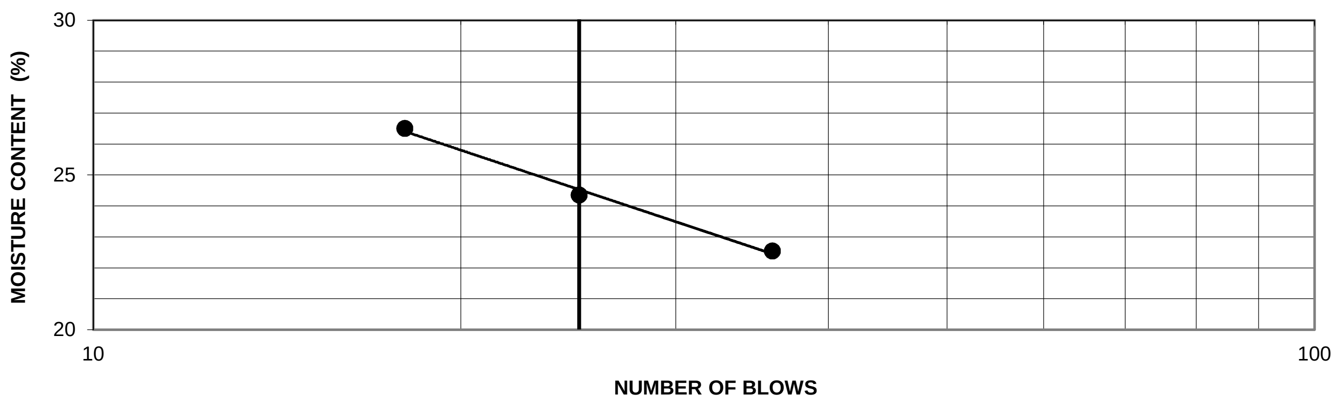
DETERMINATION NO.		1	
DISH NO.		21	
MASS, DISH + WET SOIL	(g)	37.28	
MASS, DISH + DRY SOIL	(g)	35.54	
MASS OF WATER	(g)	1.74	
MASS OF DISH	(g)	23.66	
MASS OF DRY SOIL	(g)	11.88	
MOISTURE CONTENT	(%)	14.6	

LIQUID LIMIT

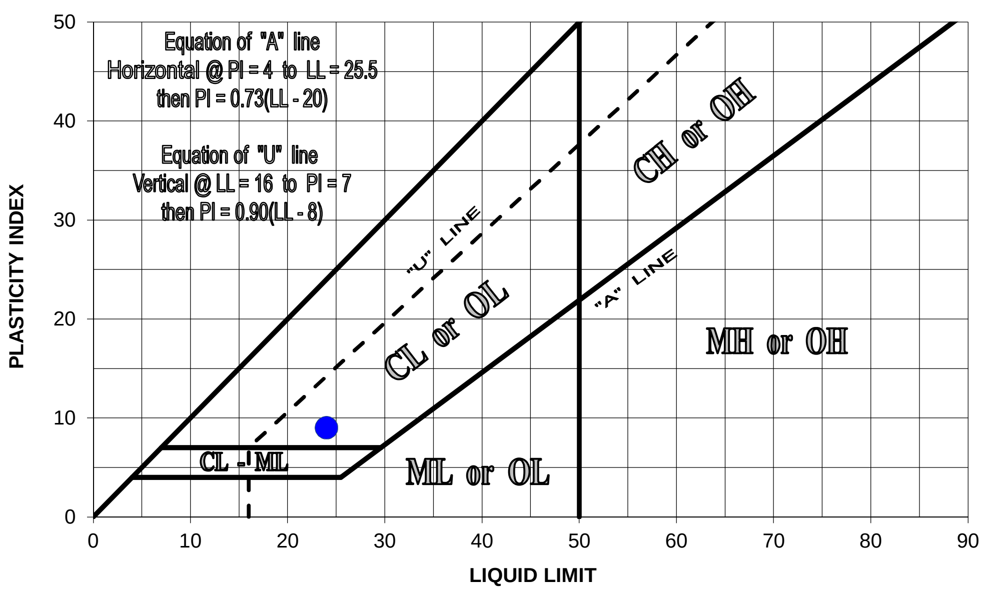
DETERMINATION NO.		1	2	3
DISH NO.		22	9	19
MASS, DISH + WET SOIL	(g)	38.53	37.25	39.06
MASS, DISH + DRY SOIL	(g)	35.22	34.09	36.52
MASS OF WATER	(g)	3.31	3.16	2.54
MASS OF DISH	(g)	22.73	21.11	25.25
MOISTURE CONTENT	(%)	26.5	24.3	22.5
NUMBER OF BLOWS		18	25	36

NATURAL MOISTURE CONTENT,%

FLOW CURVE



PLASTICITY CHART



RESULT SUMMARY

NATURAL MOISTURE CONTENT, (%)	
LIQUID LIMIT (LL)	24
PLASTIC LIMIT (PL)	15
PLASTICITY INDEX (PI)	9
SYMBOL FROM PLASTICITY CHART	CL

METHOD OF PREPARATION		METHOD OF LL DETERMINATION	
DRY	X	MULTIPOINT	X
WET		ONE-POINT	

REMARKS :

ATTERBERG LIMITS

(ASTM D4318)

WILLDAN Geotechnical | *extending your reach*

Project Name :	Boyle Heights Sports Center	Project No.:	106965-2000		
Sample Location / Source :	TW-1	Tested by :	RMC	Date:	7/26/2017
Sample Depth / No. :	1' - 5'	Sampled by:		Date:	
Sample Description / Classification :	Clayey SAND (SC)				

PLASTIC LIMIT

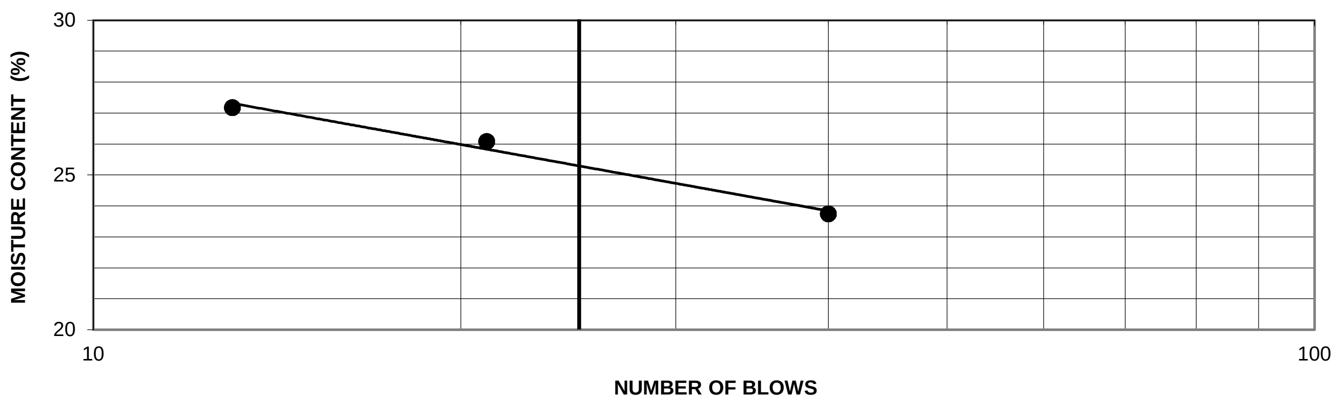
DETERMINATION NO.		1	
DISH NO.		4	
MASS, DISH + WET SOIL	(g)	36.39	
MASS, DISH + DRY SOIL	(g)	34.42	
MASS OF WATER	(g)	1.97	
MASS OF DISH	(g)	22.23	
MASS OF DRY SOIL	(g)	12.19	
MOISTURE CONTENT	(%)	16.2	

LIQUID LIMIT

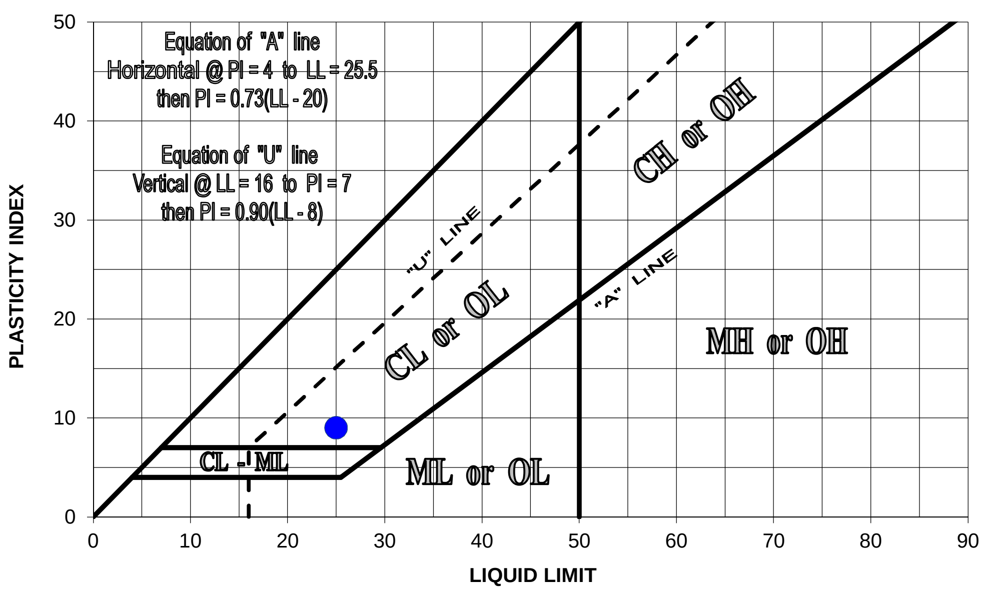
DETERMINATION NO.		1	2	3
DISH NO.		19	9	6
MASS, DISH + WET SOIL	(g)	32.78	34.03	39.44
MASS, DISH + DRY SOIL	(g)	31.09	31.36	36.16
MASS OF WATER	(g)	1.69	2.67	3.28
MASS OF DISH	(g)	23.97	21.12	24.09
MOISTURE CONTENT	(%)	23.7	26.1	27.2
NUMBER OF BLOWS		40	21	13

NATURAL MOISTURE CONTENT,%

FLOW CURVE



PLASTICITY CHART



RESULT SUMMARY

NATURAL MOISTURE CONTENT, (%)	
LIQUID LIMIT (LL)	25
PLASTIC LIMIT (PL)	16
PLASTICITY INDEX (PI)	9
SYMBOL FROM PLASTICITY CHART	CL

METHOD OF PREPARATION		METHOD OF LL DETERMINATION	
DRY	X	MULTIPOINT	X
WET		ONE-POINT	

REMARKS :

ATTERBERG LIMITS

(ASTM D4318)

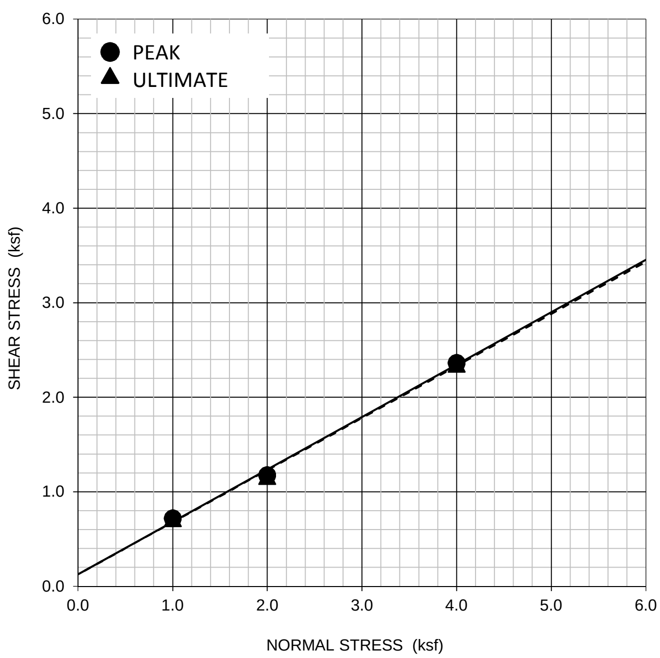
WILLDAN Geotechnical | *extending your reach*

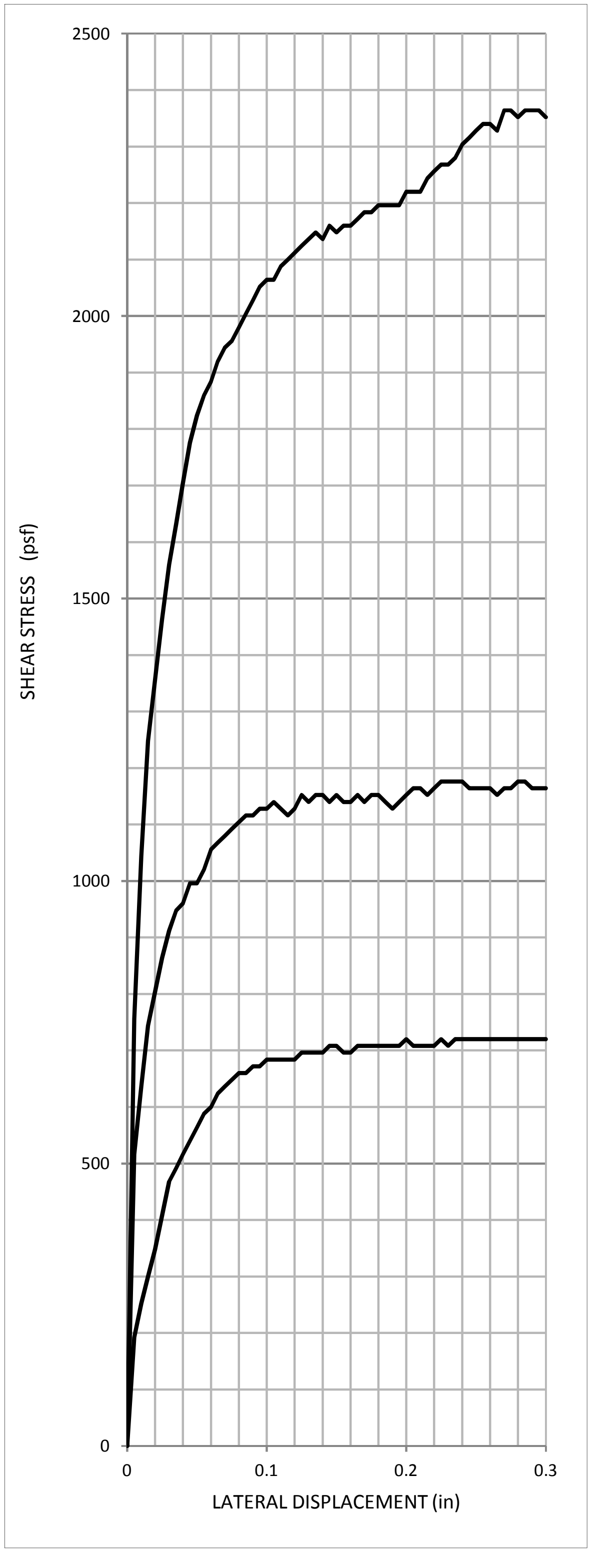
Project Name :	Boyle Heights Sports Center	Project No. :	106965-2000
Boring / Sample No :	B-1 Depth : 1' - 5'	Tested By :	RMC Date: 17-Jul-17
Sample Descriptions / Classification :	Sandy Lean CLAY (CL)	Sampled By :	Date:

Applied Normal Load (ksf)	1.0		2.0		4.0	
Shear Stress,Peak (ksf)	0.720		1.176		2.364	
Shear Stress,Ultimate(ksf)	0.720		1.164		2.352	
Density and Saturation	Initial	Final	Initial	Final	Initial	Final
Wet Wt. of Soil + Ring (g)	194.58	204.8	190.7	200.9	190.2	200.8
Dry Wt. of Soil + Ring (g)		181.2		177.2		176.8
Weight of Water (g)	13.4	23.7	13.5	23.7	13.4	24.0
Weight of Ring (g)		45.7		41.3		41.8
Weight of Dry Soil (g)		135.5		135.9		135.0
Moisture Content (%)	9.9	17.5	9.9	17.4	9.9	17.8
Wet Density (pcf)	123.6	132.2	124.0	132.5	123.1	131.9
Dry Density (pcf)		112.5		112.8		112.0
Specific Gravity (Assumed)	2.68					
Specimen Thickness (in)	1.00					
Specimen Diameter (in)	2.416					
Degree of Saturation (%)	54.5	96.2	55.0	96.8	53.9	96.6
Void Ratio		0.486		0.483		0.493

Lateral Displacement, d_h (in)		0.3
Displacement Rate, d_r (in/min)		0.03
Elapsed Time of Test, t_e (min)		10.00
Specimen	Undisturbed	-
	Remolded	90% RC
	Reconstituted	-

SHEAR STRESS	PEAK	ULTIMATE
Cohesion, c (psf)	125	125
Friction Angle, ϕ (degree)	29.0	29.0





Remarks : ______

DIRECT SHEAR TEST

(ASTM D3080)

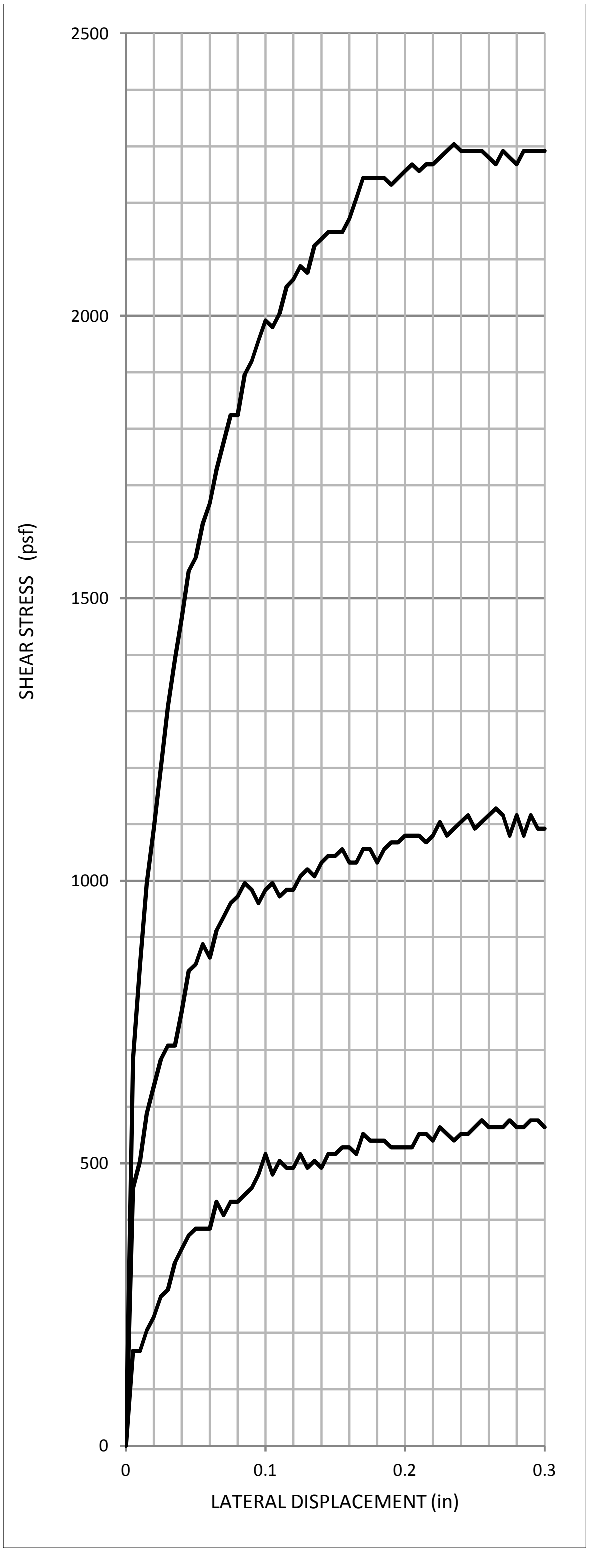
WILLDAN Geotechnical extending your reach

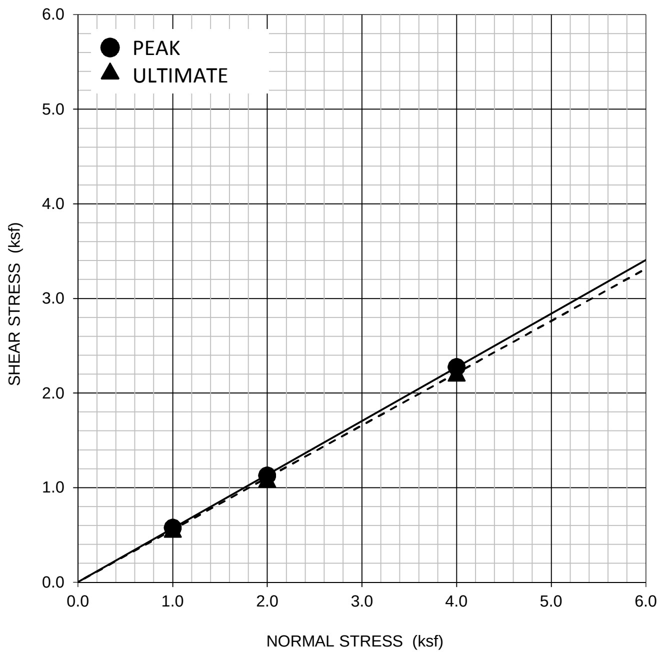
Project Name :	Boyle Heights Sports Center		Project No. :	106965-2000		
Boring / Sample No :	B-1	Depth : 5.0'	Tested By :	RMC	Date:	17-Jul-17
Sample Descriptions / Classification :	Sandy Lean CLAY (CL)		Sampled By :		Date:	

Applied Normal Load (ksf)	1.0		2.0		4.0	
Shear Stress,Peak (ksf)	0.576		1.128		2.275	
Shear Stress,Ultimate(ksf)	0.564		1.092		2.215	
Density and Saturation	Initial	Final	Initial	Final	Initial	Final
Wet Wt. of Soil + Ring (g)	161.01	185.7	160.2	184.9	161.5	187.9
Dry Wt. of Soil + Ring (g)		155.2		154.5		155.7
Weight of Water (g)	5.8	30.5	5.8	30.5	5.8	32.1
Weight of Ring (g)		43.7		43.8		44.3
Weight of Dry Soil (g)		111.6		110.7		111.4
Moisture Content (%)	5.2	27.3	5.2	27.5	5.2	28.9
Wet Density (pcf)	97.4	117.9	96.6	117.2	97.3	119.2
Dry Density (pcf)		92.6		91.9		92.5
Specific Gravity (Assumed)	2.68					
Specimen Thickness (in)	1.00					
Specimen Diameter (in)	2.416					
Degree of Saturation (%)	17.3	90.9	17.0	90.0	17.3	95.7
Void Ratio		0.806		0.820		0.808

Lateral Displacement, d_h (in)		0.3
Displacement Rate, d_r (in/min)		0.03
Elapsed Time of Test, t_e (min)		10.00
Specimen	Undisturbed	X
	Remolded	-
	Reconstituted	-

SHEAR STRESS	PEAK	ULTIMATE
Cohesion, c (psf)	5	5
Friction Angle, ϕ (degree)	29.5	29.0





Remarks :

DIRECT SHEAR TEST

(ASTM D3080)

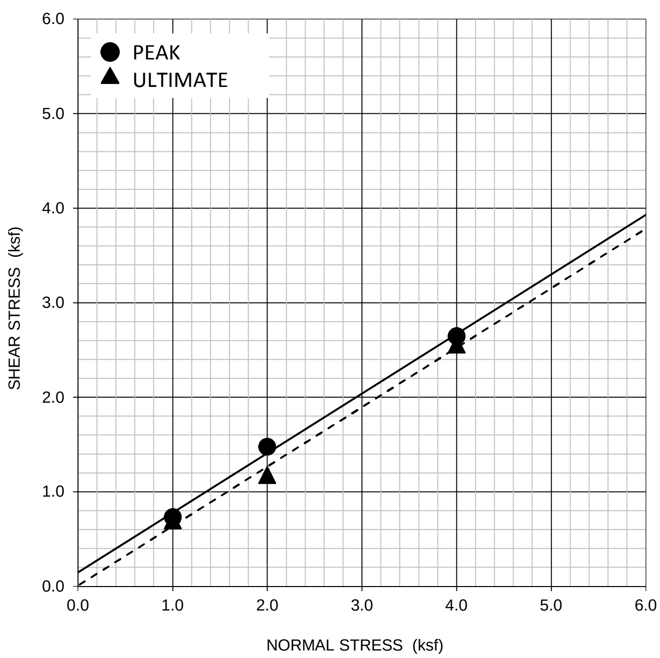
WILLDAN Geotechnical | extending your reach

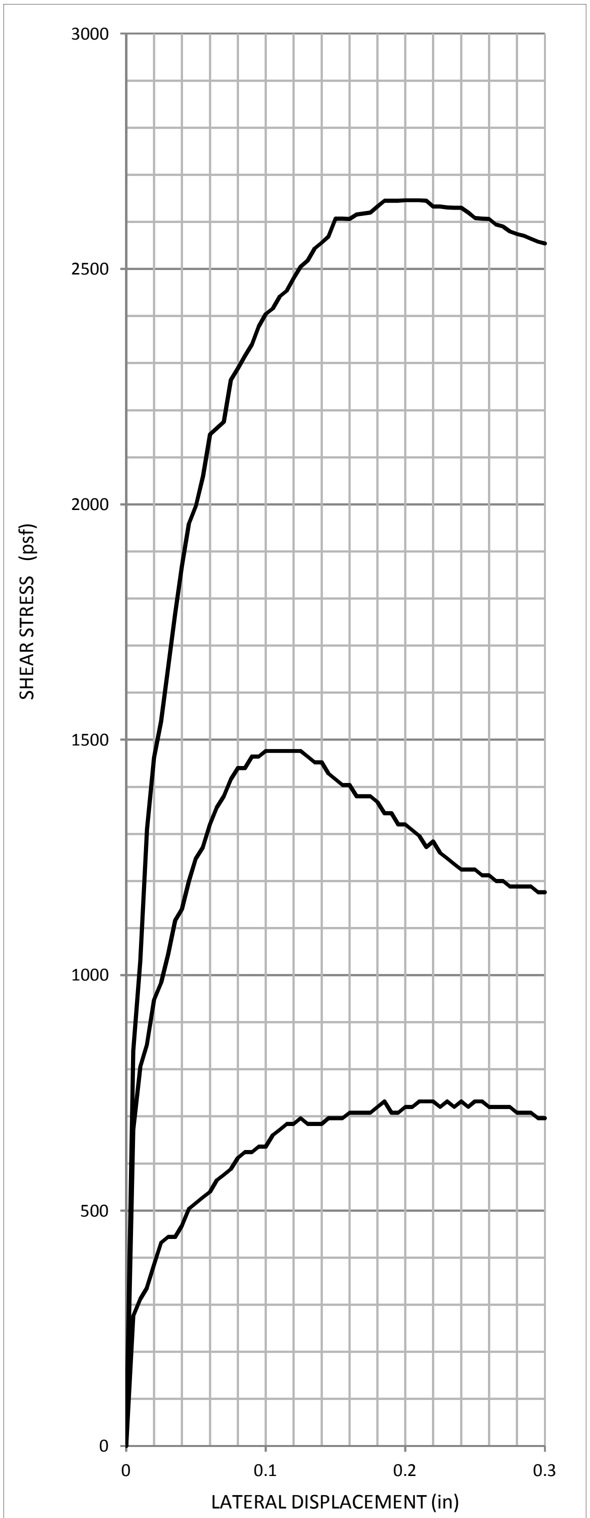
Project Name :	Boyle Heights Sports Center	Project No. :	106965-2000
Boring / Sample No :	B-1 Depth : 10.0'	Tested By :	RMC Date: 17-Jul-17
Sample Descriptions / Classification :	Silty SAND (SM)	Sampled By :	Date:

Applied Normal Load (ksf)	1.0		2.0		4.0	
Shear Stress,Peak (ksf)	0.732		1.476		2.646	
Shear Stress,Ultimate(ksf)	0.696		1.176		2.554	
Density and Saturation	Initial	Final	Initial	Final	Initial	Final
Wet Wt. of Soil + Ring (g)	187.12	200.9	193.6	206.5	188.6	202.8
Dry Wt. of Soil + Ring (g)		178.0		184.3		179.5
Weight of Water (g)	9.1	22.9	9.3	22.2	9.1	23.3
Weight of Ring (g)		42.0		45.4		43.4
Weight of Dry Soil (g)		136.0		138.9		136.1
Moisture Content (%)	6.7	16.8	6.7	16.0	6.7	17.1
Wet Density (pcf)	120.5	131.9	123.1	133.8	120.6	132.3
Dry Density (pcf)		112.9		115.3		113.0
Specific Gravity (Assumed)	2.68					
Specimen Thickness (in)	1.00					
Specimen Diameter (in)	2.416					
Degree of Saturation (%)	37.3	93.6	39.9	95.2	37.4	95.6
Void Ratio		0.481		0.450		0.480

Lateral Displacement, d_h (in)		0.3
Displacement Rate, d_r (in/min)		0.05
Elapsed Time of Test, t_e (min)		6.00
Specimen	Undisturbed	X
	Remolded	-
	Reconstituted	-

SHEAR STRESS	PEAK	ULTIMATE
Cohesion, c (psf)	145	5
Friction Angle, ϕ (degree)	32.0	32.0





Remarks :

DIRECT SHEAR TEST

(ASTM D3080)

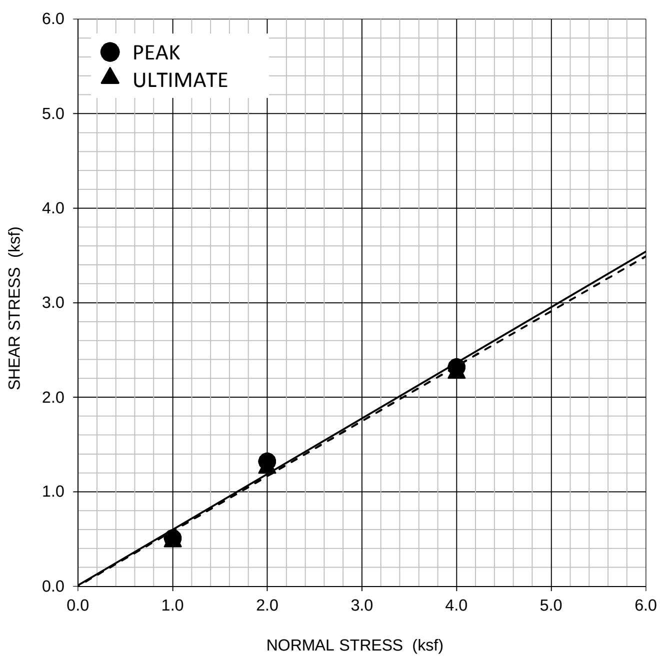
WILLDAN Geotechnical | extending your reach

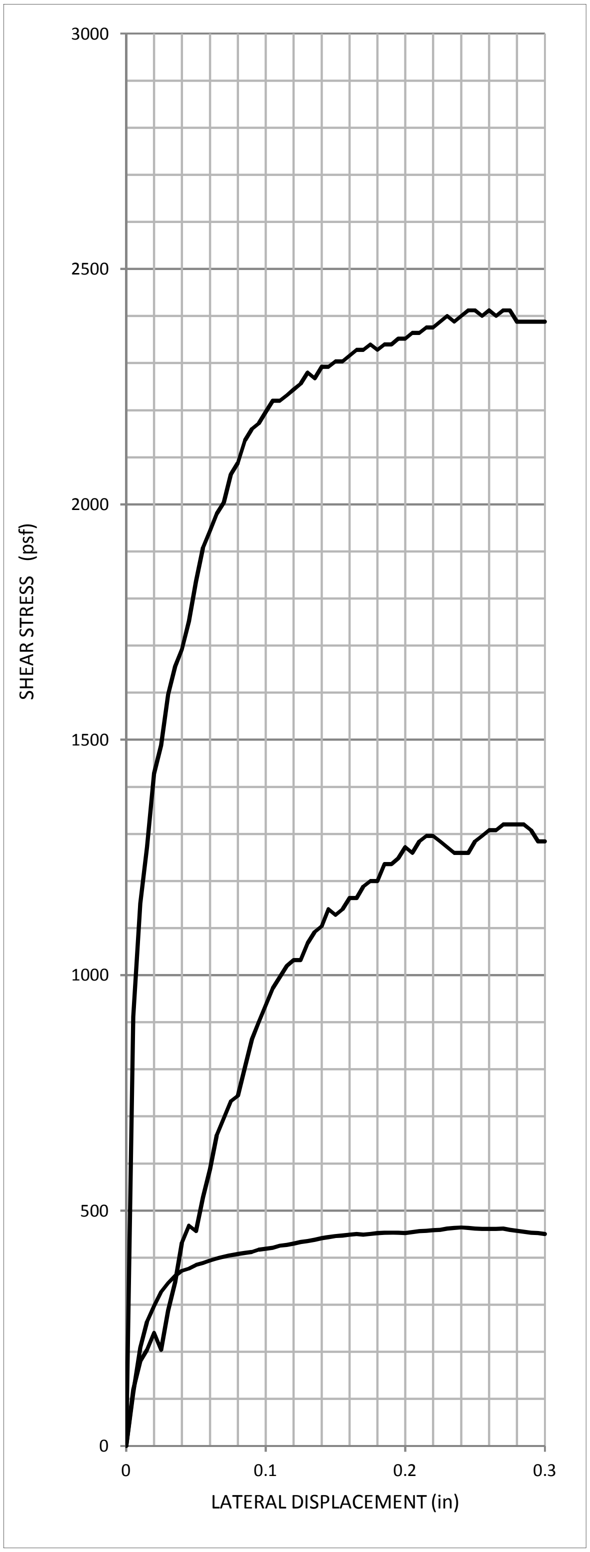
Project Name :	Boyle Heights Sports Center	Project No. :	106965-2000
Boring / Sample No :	B-2 Depth : 7.5'	Tested By :	RMC Date: 17-Jul-17
Sample Descriptions / Classification :	Silty SAND with Gravel (SM)	Sampled By :	Date:

Applied Normal Load (ksf)	1.0		2.0		4.0	
Shear Stress,Peak (ksf)	0.510		1.320		2.320	
Shear Stress,Ultimate(ksf)	0.505		1.284		2.289	
Density and Saturation	Initial	Final	Initial	Final	Initial	Final
Wet Wt. of Soil + Ring (g)	161.11	186.9	162.3	187.9	162.6	188.3
Dry Wt. of Soil + Ring (g)		156.2		157.2		157.6
Weight of Water (g)	5.0	30.7	5.0	30.6	5.0	30.7
Weight of Ring (g)		43.7		43.6		44.3
Weight of Dry Soil (g)		112.5		113.6		113.3
Moisture Content (%)	4.4	27.3	4.4	26.9	4.4	27.1
Wet Density (pcf)	97.5	118.9	98.5	119.8	98.2	119.6
Dry Density (pcf)		93.4		94.4		94.1
Specific Gravity (Assumed)	2.68					
Specimen Thickness (in)	1.00					
Specimen Diameter (in)	2.416					
Degree of Saturation (%)	14.9	92.6	15.3	93.5	15.2	93.3
Void Ratio		0.790		0.772		0.777

Lateral Displacement, d_h (in)		0.3
Displacement Rate, d_r (in/min)		0.03
Elapsed Time of Test, t_e (min)		10.00
Specimen	Undisturbed	X
	Remolded	-
	Reconstituted	-

SHEAR STRESS	PEAK	ULTIMATE
Cohesion, c (psf)	10	5
Friction Angle, ϕ (degree)	30.5	30.0





Remarks : ____________________

DIRECT SHEAR TEST

(ASTM D3080)

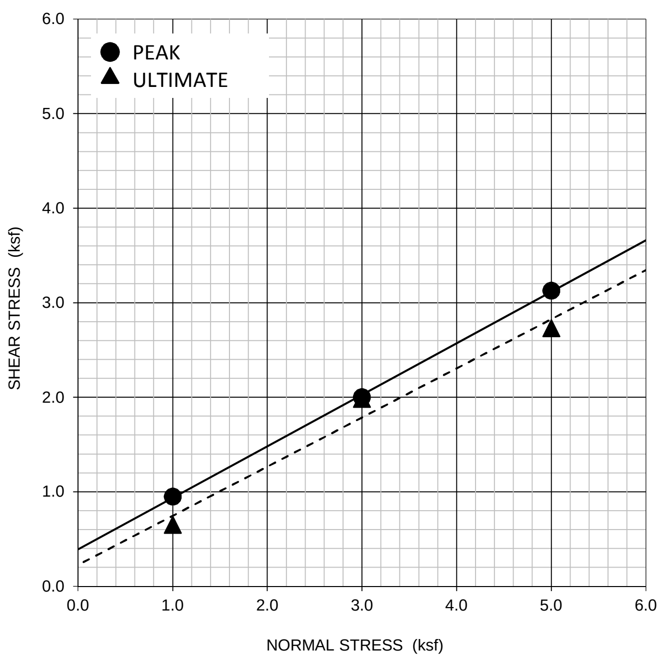
WILLDAN Geotechnical | extending your reach

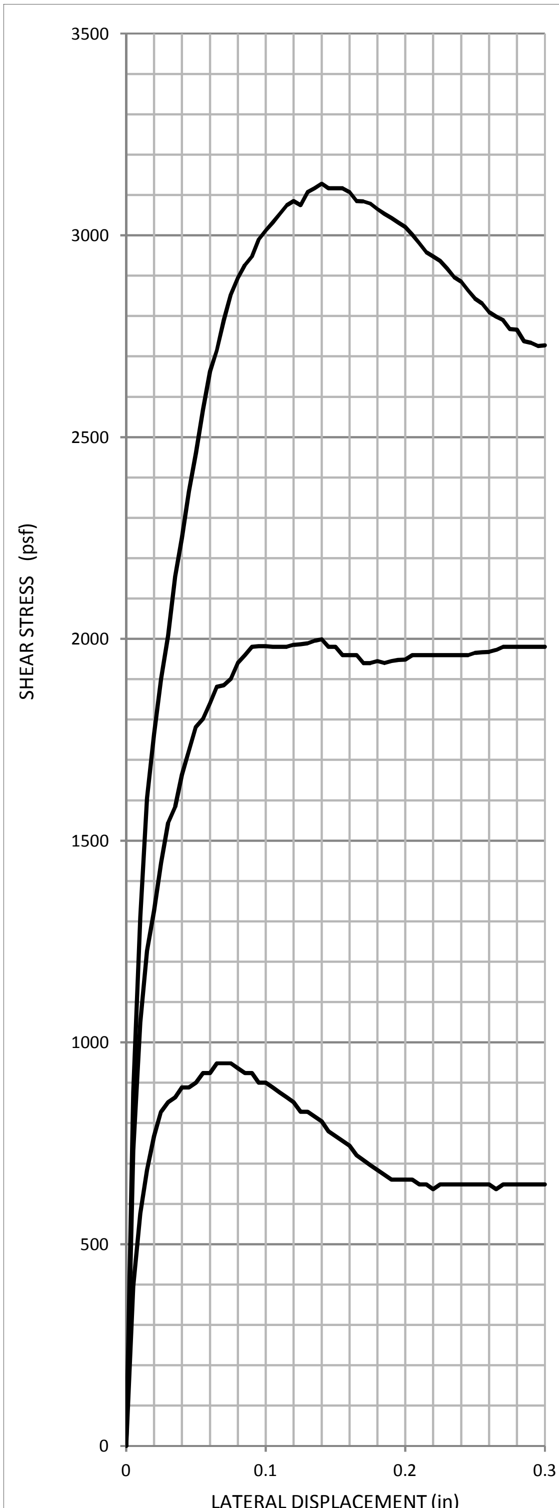
Project Name :	Boyle Heights Sports Center	Project No. :	106965-2000
Boring / Sample No :	B-2 Depth : 25.0'	Tested By :	RMC Date: 25-Jul-17
Sample Descriptions / Classification :	Sandy CLAY/Clayey SAND (CL/SC)	Sampled By :	Date:

Applied Normal Load (ksf)	1.0		3.0		5.0	
Shear Stress,Peak (ksf)	0.948		1.999		3.128	
Shear Stress,Ultimate(ksf)	0.648		1.980		2.727	
Density and Saturation	Initial	Final	Initial	Final	Initial	Final
Wet Wt. of Soil + Ring (g)	196.16	206.2	196.3	207.2	200.2	209.8
Dry Wt. of Soil + Ring (g)		183.9		184.2		187.8
Weight of Water (g)	12.3	22.3	12.1	23.0	12.3	21.9
Weight of Ring (g)		42.8		45.0		46.4
Weight of Dry Soil (g)		141.1		139.2		141.5
Moisture Content (%)	8.7	15.8	8.7	16.5	8.7	15.5
Wet Density (pcf)	127.3	135.6	125.6	134.7	127.7	135.7
Dry Density (pcf)		117.1		115.6		117.5
Specific Gravity (Assumed)	2.68					
Specimen Thickness (in)	1.00					
Specimen Diameter (in)	2.416					
Degree of Saturation (%)	54.5	98.9	52.2	99.0	55.0	98.1
Void Ratio		0.428		0.447		0.424

Lateral Displacement, d_h (in)		0.3
Displacement Rate, d_r (in/min)		0.03
Elapsed Time of Test, t_e (min)		10.00
Specimen	Undisturbed	X
	Remolded	-
	Reconstituted	-

SHEAR STRESS	PEAK	ULTIMATE
Cohesion, c (psf)	390	225
Friction Angle, ϕ (degree)	28.5	27.5

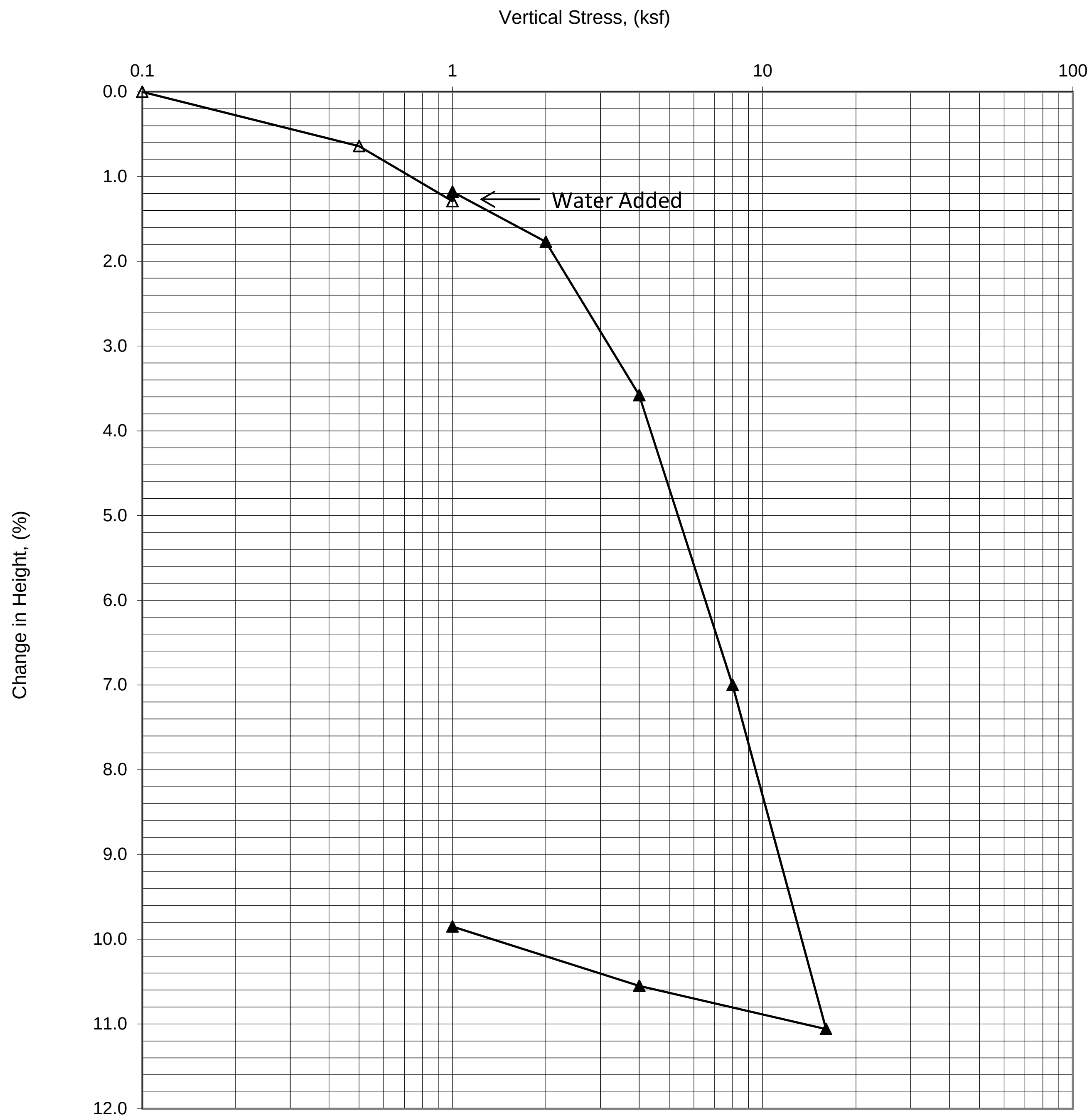




Remarks :

DIRECT SHEAR TEST
(ASTM D3080)

WILLDAN Geotechnical | extending your reach



Boring No. : B-1

Depth,(ft) : 1' - 5'

Pre-Consolidation Pressure (P_c), ksf : 3.10

Sample Descriptions / Classification: Sandy Lean CLAY (CL) (Remolded to 90% RC)

Overburden Pressure (P_o), ksf : 0.34

Sp. Gravity : 2.68 (Assumed)

Compression Index, C_c : 0.135

Swell Index, C_s : 0.010

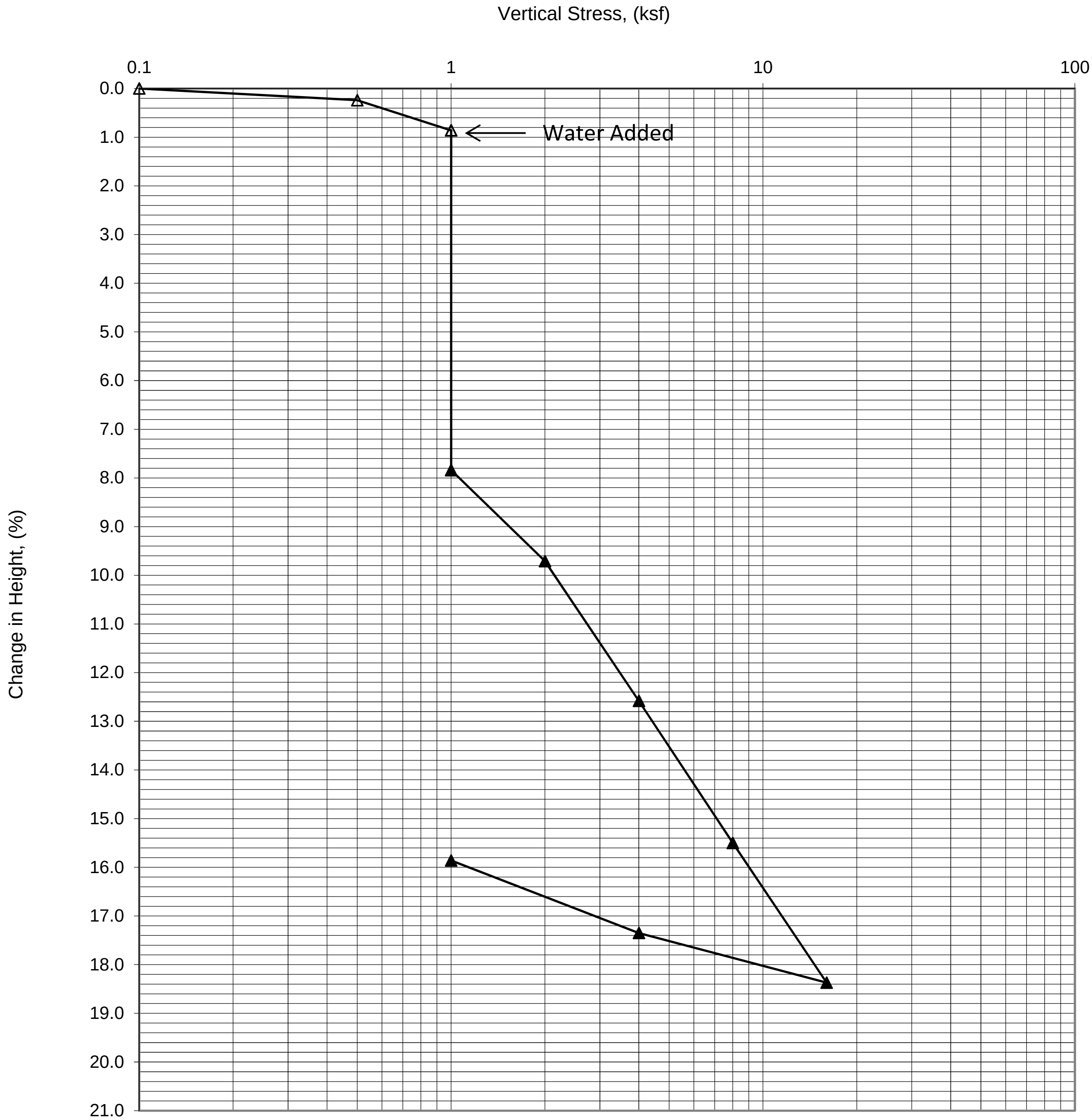
Specimen Height (inches)		Moisture Content (%)	Dry Density (pcf)	Saturation (%)	Void Ratio
Initial	1.0000	9.9	112.0	53.9	0.494
Final	0.9015	12.8	124.2	99.0	0.347

Consolidation Test

(ASTM D2435)

WILLDAN Geotechnical | *extending your reach*

Boyle Heights Sports Center | **Project No. 106965-2000** | **27-Jul-17**



Boring No. : B-1 Depth,(ft) : 5 Pre-Consolidation Pressure (P_c), ksf : 1.60

Sample Descriptions / Classification : Sandy Lean CLAY (CL) Overburden Pressure (P_o), ksf : 0.48

Sp. Gravity : 2.68 (Assumed) Compression Index, C_c : 0.095 Swell Index, C_s : 0.021

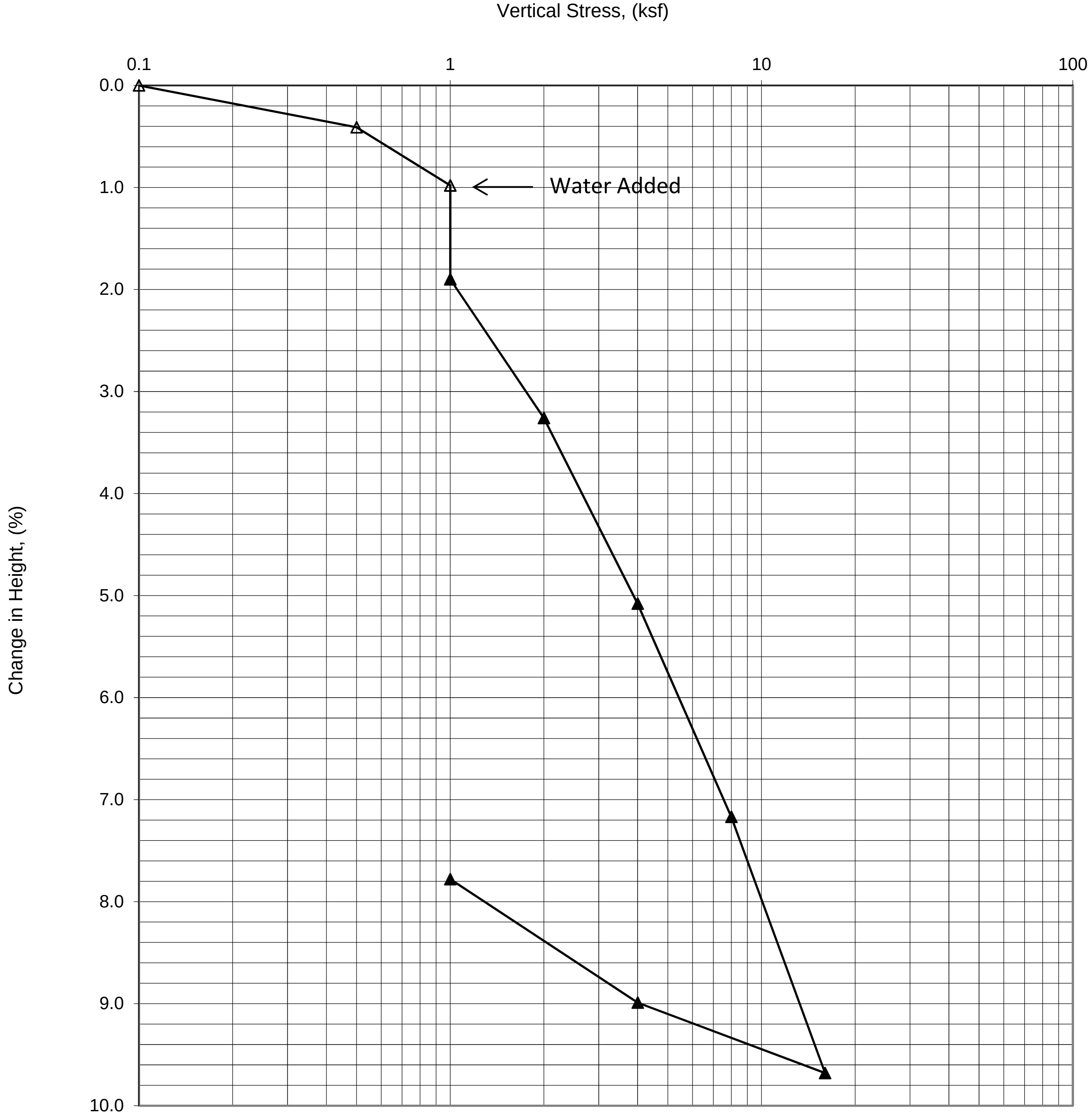
Specimen Height (inches)		Moisture Content (%)	Dry Density (pcf)	Saturation (%)	Void Ratio
Initial	1.0000	7.7	91.6	24.9	0.826
Final	0.8414	19.7	108.8	98.2	0.537

Consolidation Test

(ASTM D2435)

WILLDAN Geotechnical *extending your reach*

Boyle Heights Sports Center | **Project No. 106965-2000** | **27-Jul-17**



Boring No. : TW-1 Depth,(ft) : 3.5 Pre-Consolidation Pressure (P_c), ksf : 1.60

Sample Descriptions / Classification : Clayey SAND (SC) Overburden Pressure (P_o), ksf : 0.47

Sp. Gravity : 2.68 (Assumed) Compression Index, C_c : 0.083 Swell Index, C_s : 0.016

Specimen Height (inches)		Moisture Content (%)	Dry Density (pcf)	Saturation (%)	Void Ratio
Initial	1.0000	6.6	123.7	50.5	0.352
Final	0.9222	9.1	134.1	99.1	0.247

Consolidation Test
(ASTM D2435)

WILLDAN Geotechnical *extending your reach*

Boyle Heights Sports Center | **Project No. 106965-2000** | **27-Jul-17**

JOB NAME :	Boyle Heights Sports Center	JOB NUMBER:	106965-2000
SAMPLE NUMBER :		TESTED BY :	RMC
SAMPLE LOCATION :	B-1 @ 1' - 5'	DATE :	13-Jul-17
SAMPLE DESCRIPTIONS / CLASSIFICATION :	Sandy Lean CLAY (CL)		
TEST STANDARD:	ASTM D1557 - 12	METHOD:	A

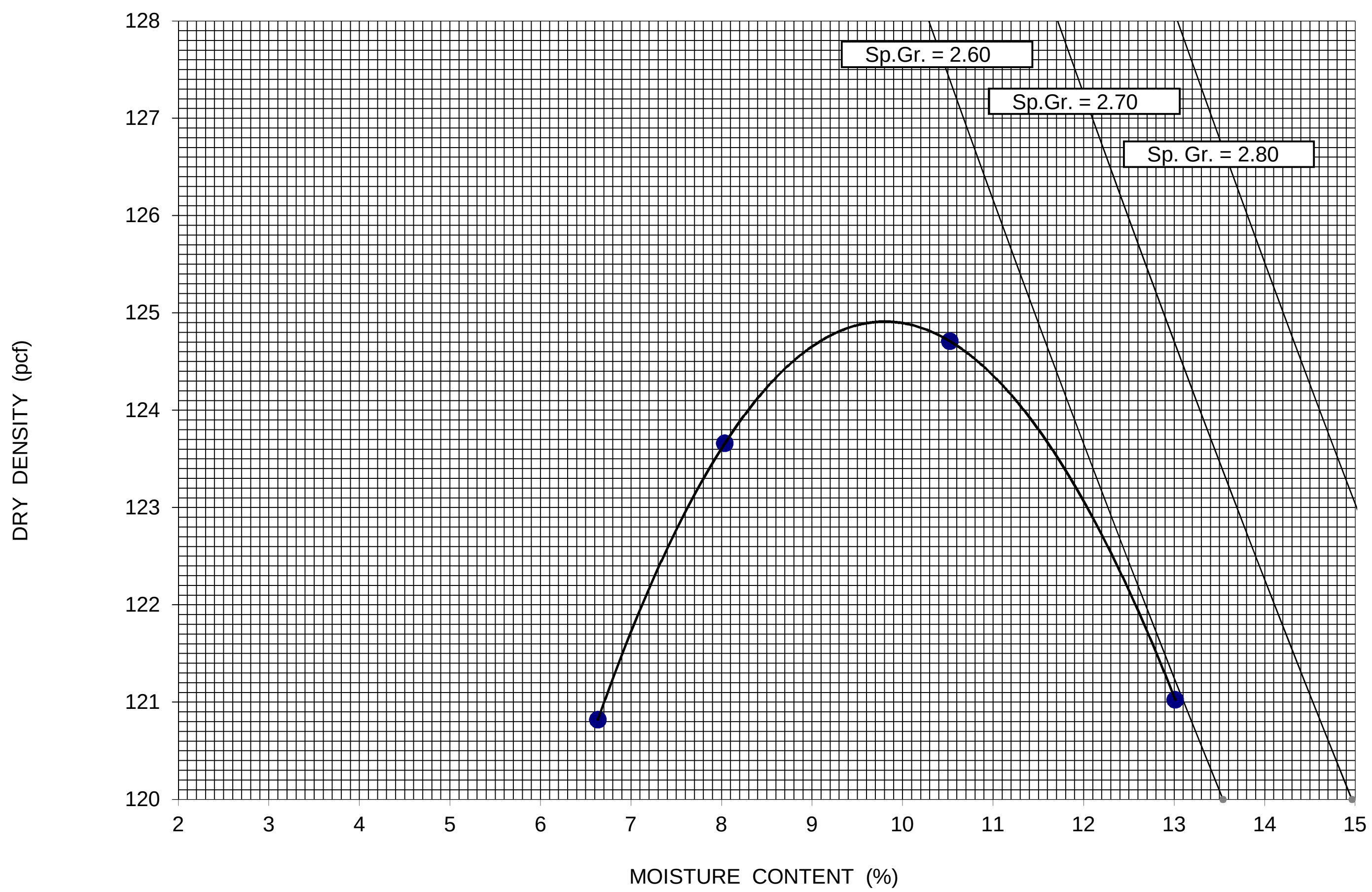
TRIAL NUMBER	1	2	3	4	5
WATER ADDED, (ml)	80	160	240	-	
WT. OF SOIL + MOLD, (g)	4070	4134	4118	3998	
WT.OF MOLD, (g)	2050	2050	2050	2050	
WT. OF WET SOIL, (g)	2020	2084	2068	1948	
WET DENSITY, (pcf)	133.6	137.8	136.8	128.8	
CAN NUMBER	B9	B6	B3	B4	
WET SOIL + TARE, (g)	640.90	673.50	652.60	615.60	
DRY SOIL + TARE, (g)	599.80	617.40	587.20	582.50	
TARE, (g)	88.40	84.20	84.60	83.60	
DRY SOIL, (g)	511.40	533.20	502.60	498.90	
WATER, (g)	41.10	56.10	65.40	33.10	
MOISTURE CONTENT, (%)	8.0	10.5	13.0	6.6	
DRY DENSITY, (pcf)	123.7	124.7	121.0	120.8	

DIAMETER OF MOLD :	4	in.
VOLUME OF MOLD :	0.0333	$ft.^3$
SCALPED ON SIEVE SIZE / NO. :	#4	
PERCENT RETAINED, (%) :	-	
MAXIMUM DRY DENSITY :	**124.9**	pcf.
OPT. MOISTURE CONTENT :	**9.9**	%

FOR OVERSIZE CORRECTION (ASTM D4718-07)

%,Finer Fraction,(P_f) =	-	% Moisture =	-
%,Oversize Fraction,(P_c) =	-	Assumed S.G. =	-
Corrected MDD of Total Materials, (pcf) =	-		
Corrected OMC of Total Materials, (%) =	-		

Remarks :



COMPACTION TEST

WILLDAN Geotechnical | extending your reach

JOB NAME :	Boyle Heights Sports Center	JOB NUMBER:	106965-2000
SAMPLE NUMBER :		TESTED BY :	RMC
SAMPLE LOCATION :	B-2 @ 1' - 5'	DATE :	17-Jul-17
SAMPLE DESCRIPTIONS / CLASSIFICATION :	Clayey SAND (SC)		

TEST STANDARD:	ASTM D1557 - 12	METHOD:	A

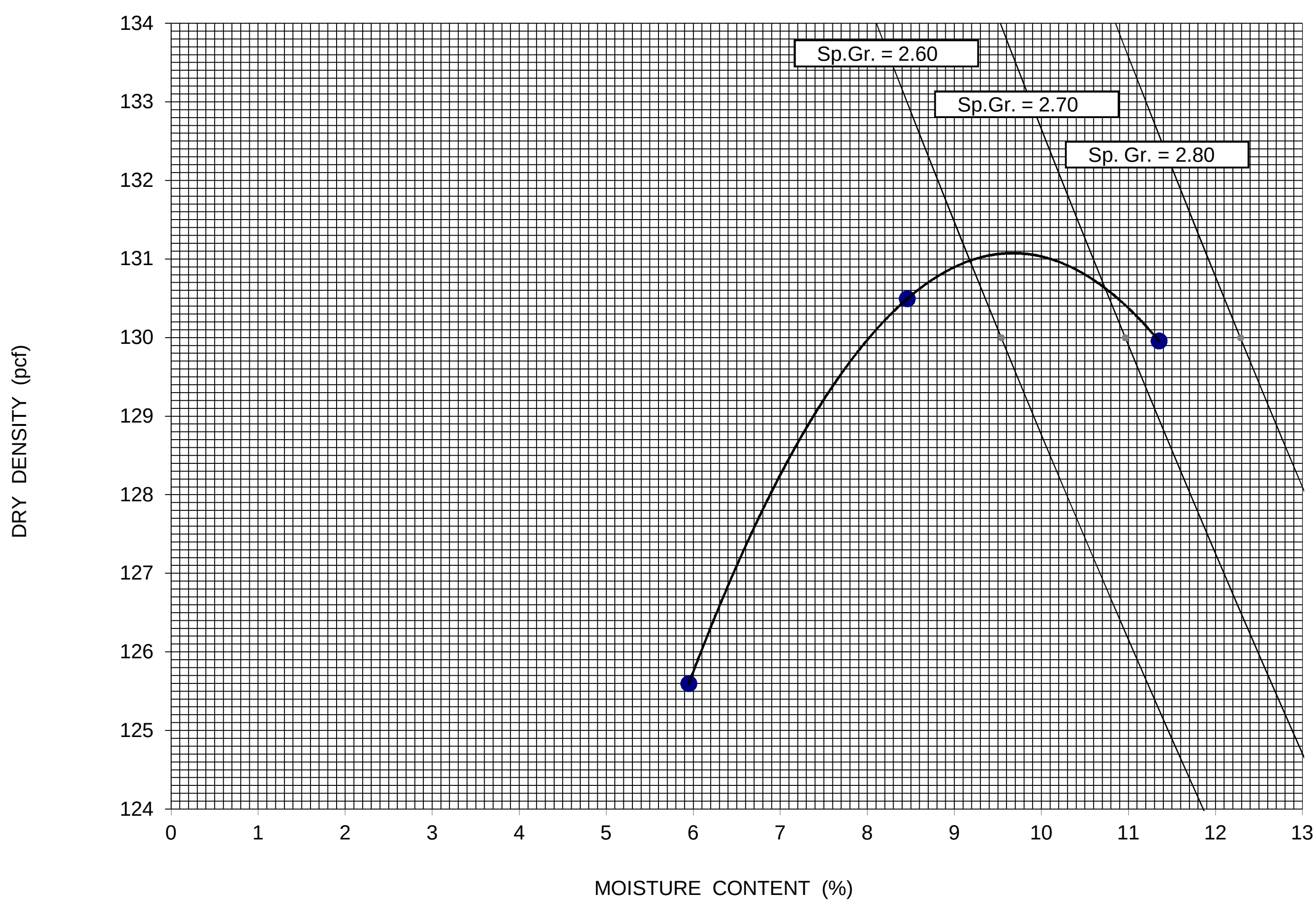
TRIAL NUMBER	1	2	3	4	5
WATER ADDED, (ml)	-	100	150		
WT. OF SOIL + MOLD, (g)	4062	4190	4238		
WT.OF MOLD, (g)	2050	2050	2050		
WT. OF WET SOIL, (g)	2012	2140	2188		
WET DENSITY, (pcf)	133.1	141.5	144.7		
CAN NUMBER	B5	B14	B2		
WET SOIL + TARE, (g)	543.22	542.10	565.45		
DRY SOIL + TARE, (g)	517.66	506.24	516.77		
TARE, (g)	88.02	82.37	87.95		
DRY SOIL, (g)	429.64	423.87	428.82		
WATER, (g)	25.56	35.86	48.68		
MOISTURE CONTENT, (%)	5.9	8.5	11.4		
DRY DENSITY, (pcf)	125.6	130.5	130.0		

DIAMETER OF MOLD :	4	in.
VOLUME OF MOLD :	0.0333	$ft.^3$
SCALPED ON SIEVE SIZE / NO. :	3/8"	
PERCENT RETAINED, (%) :	11.7	
MAXIMUM DRY DENSITY :	**131.1**	pcf.
OPT. MOISTURE CONTENT :	**9.7**	%

FOR OVERSIZE CORRECTION (ASTM D4718-07)

%,Finer Fraction,(P_f) =	88.3	% Moisture =	8.5
%,Oversize Fraction,(P_c) =	11.7	Assumed S.G. =	2.64
Corrected MDD of Total Materials, (pcf) =	134.3		
Corrected OMC of Total Materials, (%) =	8.6		

Remarks :



COMPACTION TEST

WILLDAN Geotechnical | extending your reach

JOB NAME :	Boyle Heights Sports Center	JOB NUMBER:	106965-2000
SAMPLE NUMBER :		TESTED BY :	RMC
SAMPLE LOCATION :	B-4 @ 1' - 5'	DATE :	17-Jul-17
SAMPLE DESCRIPTIONS / CLASSIFICATION :	Clayey SAND (SC)		
TEST STANDARD:	ASTM D1557 - 12	METHOD:	A

TRIAL NUMBER	1	2	3	4	5
WATER ADDED, (ml)	-	100	150		
WT. OF SOIL + MOLD, (g)	4111	4189	4208	4179	
WT.OF MOLD, (g)	2050	2050	2050	2050	
WT. OF WET SOIL, (g)	2061	2139	2158	2129	
WET DENSITY, (pcf)	136.3	141.5	142.7	140.8	
CAN NUMBER	B12	B7	B2	B8	
WET SOIL + TARE, (g)	566.45	513.32	581.41	535.35	
DRY SOIL + TARE, (g)	540.15	481.38	538.64	490.56	
TARE, (g)	84.54	62.92	83.14	82.92	
DRY SOIL, (g)	455.61	418.46	455.50	407.64	
WATER, (g)	26.30	31.94	42.77	44.79	
MOISTURE CONTENT, (%)	5.8	7.6	9.4	11.0	
DRY DENSITY, (pcf)	128.9	131.4	130.5	126.9	

DIAMETER OF MOLD : 4 in.

VOLUME OF MOLD : 0.0333 $ft.^3$

SCALPED ON SIEVE SIZE / NO. : 3/8"

PERCENT RETAINED, (%) : 3.2

MAXIMUM DRY DENSITY : 131.5 pcf.

OPT. MOISTURE CONTENT : 8.0 %

FOR OVERSIZE CORRECTION (ASTM D4718-07)

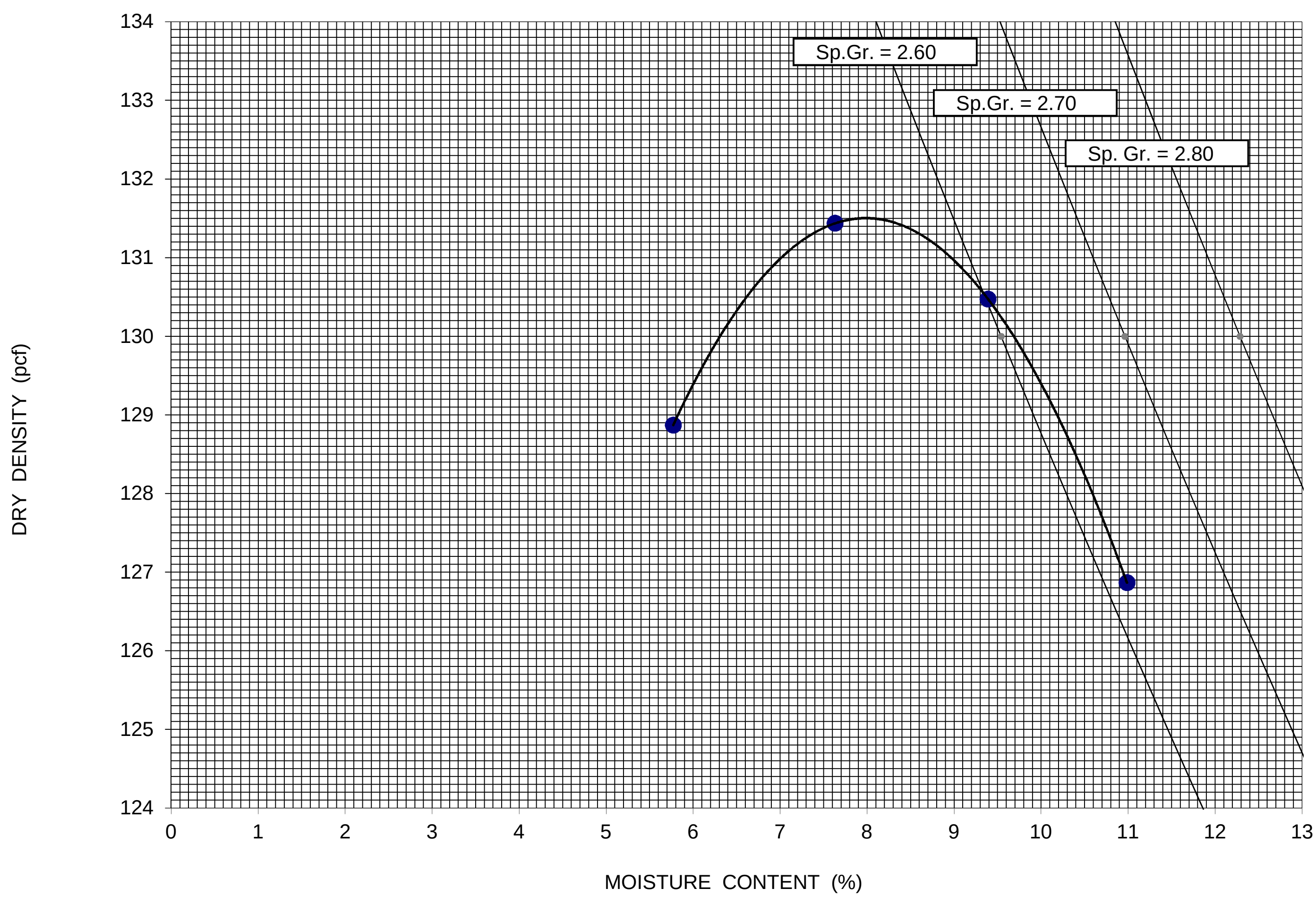
%,Finer Fraction,(P_f) = 96.8 % Moisture = 8.5

%,Oversize Fraction,(P_c) = 3.2 Assumed S.G. = 2.64

Corrected MDD of Total Materials, (pcf) = 132.3

Corrected OMC of Total Materials, (%) = 7.7

Remarks :



COMPACTION TEST

WILLDAN Geotechnical | extending your reach

Project Name :	Boyle Heights Sports Center	Project No.:	106965-2000
Sample Location / Source :	B-1	Tested by :	RMC	Date: 9/22/2017
Sample Depth / No. :	1.0' - 5.0'	Sampled by:		Date:
Sample Description / Classification :	Sandy Lean CLAY (CL)		

TRIAL NUMBER		1	2	3
WET WT. OF SOIL + RING	(g)	610.92		
WEIGHT OF RING	(g)	200.46		
WET WEIGHT OF SOIL	(g)	410.46		
FACTOR		0.303		
INITIAL WET UNIT WEIGHT	(pcf)	124.4		
DRY DENSITY	(pcf)	113.9		
% SATURATION (Assumed Sp.Gr. = 2.70)		51.8		
MOISTURE DETERMINATION				
WET WEIGHT OF SOIL	(g)	131.13		
DRY WEIGHT OF SOIL	(g)	120.09		
MOISTURE CONTENT	(%)	9.2		

RACK NO. : 1

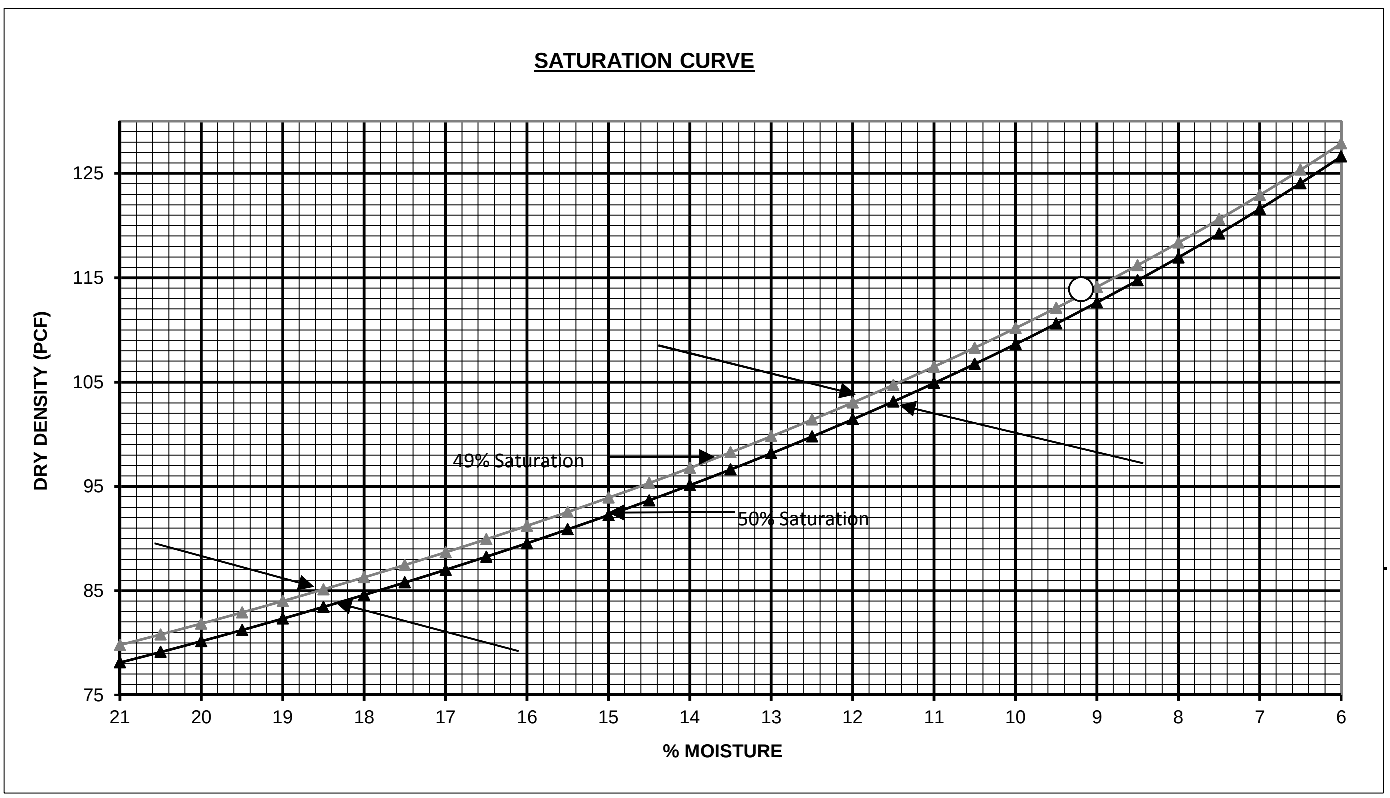
SURCHARGE : 144 psf

DATE	TIME	DIAL READINGS (In.)
22-Sep	10:50	0.642
	11:40	0.667
25-Sep	7:20	0.697
% RETAINED ON #4 SIEVE		< 5

EXPANSION INDEX : 55

REMARKS :

SOLUBLE SULFATE (SO_4) : - ppm



EXPANSION INDEX OF SOILS

(ASTM D4829)

WILLDAN Geotechnical

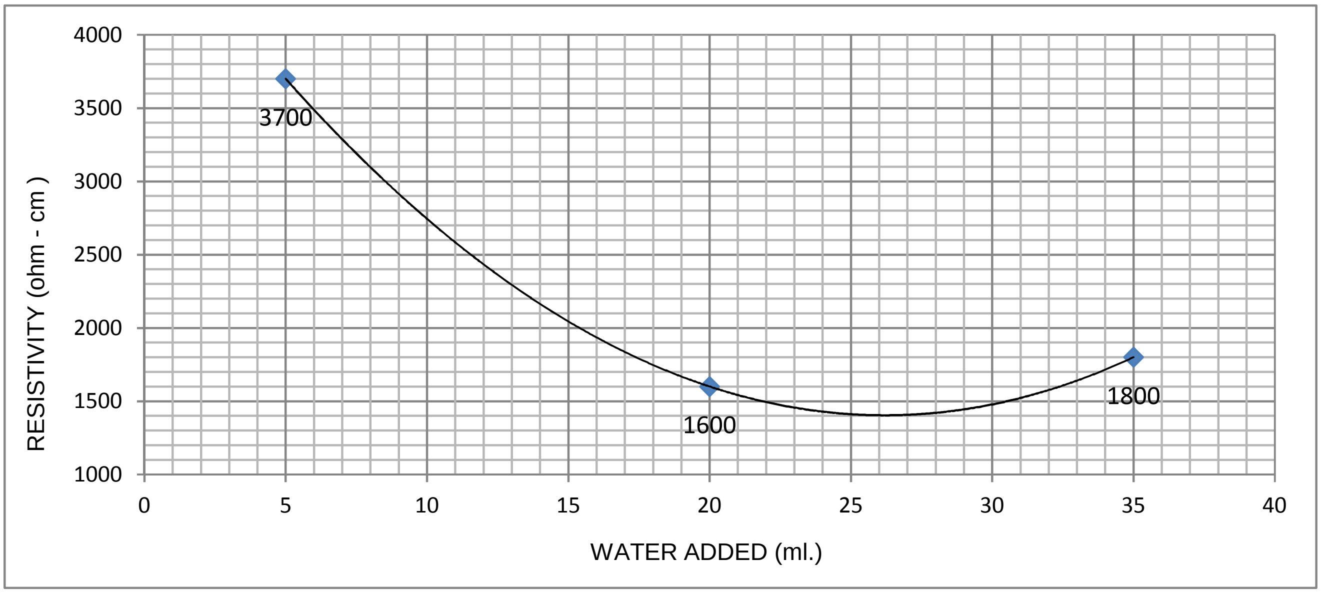
extending your reach

PROJECT NAME :	Boyle Heights Sports Center	PROJECT NUMBER :	106965-2000		
LOCATION :	B-1 @ 1' - 5'	TESTED BY :	RMC	DATE :	25-Jul-17
SAMPLE ID / DEPTH:		SAMPLED BY:		DATE :	

SOIL DESCRIPTIONS : Sandy Lean CLAY (CL)

A. MINIMUM RESISTIVITY CTM 643

WATER ADDED, (ml)	5	20	35	
RESISTIVITY MEASURED, (ohm-cm)	3700	1600	1800	
TEMPERATURE MEASURED, (^{0}C)	24.3			
MINIMUM RESISTIVITY (ohm-cm)	1400			
MIN. RESISTIVITY CORRECTED , $R_{min\ -15.5}$ (ohm-cm)	1776			



B. SULFATE CONTENT OF SOILS CTM 417

SOIL - WATER RATIO	100 : 300
SO_4 DILUTION (ALIQUOT : DISTILLED H2O)	5 : 20
FACTOR	15
SULFATE READING (ppm)	10
WATER SOLUBLE SULFATES, (ppm)	150

C. CHLORIDE CONTENT OF SOILS CTM 422 (SILVER NITRATE METHOD)

CHLORIDE DILUTION (ALIQUOT : DISTILLED H2O)	50 : 50
NUMBER OF DIGITS REQUIRED	60
WATER SOLUBLE CHLORIDES, (ppm)	180

D. pH OF SOILS CTM 643

pH VALUE	8.02

REMARKS :

CORROSION TESTS

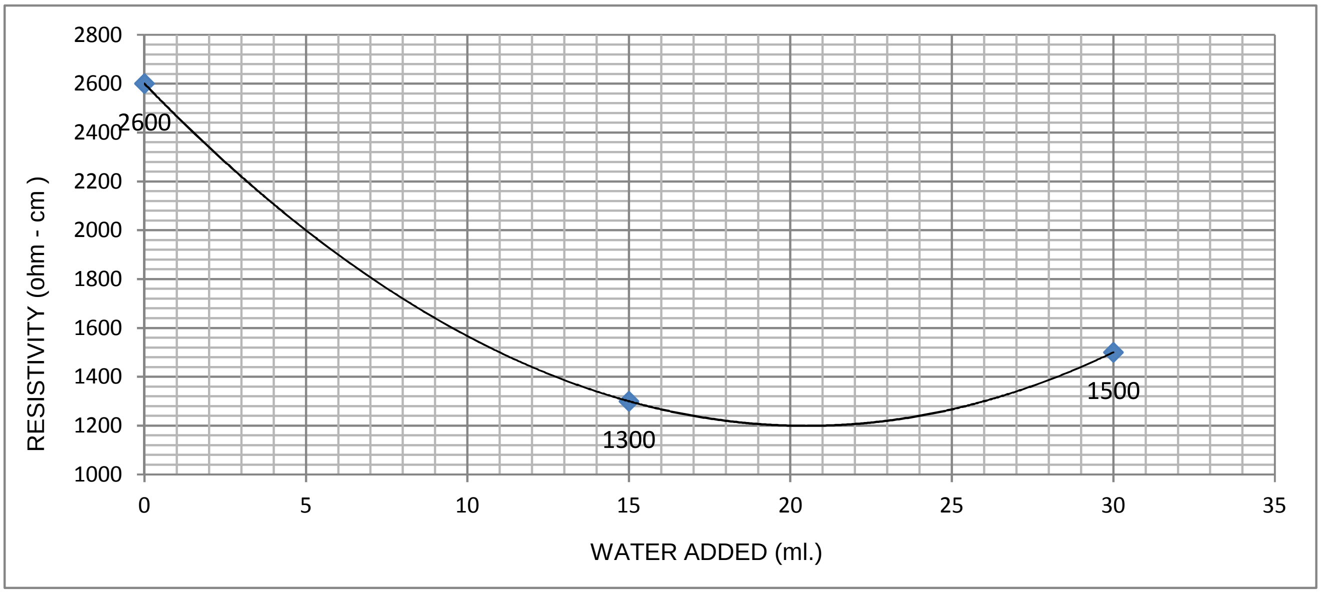
WILLDAN Geotechnical
extending your reach

PROJECT NAME : Boyle Heights Sports Center	PROJECT NUMBER : 106965-2000		
LOCATION : B-4 @ 1' - 5'	TESTED BY : RMC	DATE : 25-Jul-17	
SAMPLE ID / DEPTH:	SAMPLED BY:	DATE :	

SOIL DESCRIPTIONS : Clayey SAND (SC)

A. MINIMUM RESISTIVITY CTM 643

WATER ADDED, (ml)	0	15	30	
RESISTIVITY MEASURED, (ohm-cm)	2600	1300	1500	
TEMPERATURE MEASURED, (^{0}C)	24.3			
MINIMUM RESISTIVITY (ohm-cm)	1200			
MIN. RESISTIVITY CORRECTED , $R_{min\ -15.5}$ (ohm-cm)	1523			



B. SULFATE CONTENT OF SOILS CTM 417

SOIL - WATER RATIO	100 : 300
SO_4 DILUTION (ALIQUOT : DISTILLED H2O)	5 : 20
FACTOR	15
SULFATE READING (ppm)	22
WATER SOLUBLE SULFATES, (ppm)	330

C. CHLORIDE CONTENT OF SOILS CTM 422 (SILVER NITRATE METHOD)

CHLORIDE DILUTION (ALIQUOT : DISTILLED H2O)	50 : 50
NUMBER OF DIGITS REQUIRED	80
WATER SOLUBLE CHLORIDES, (ppm)	240

D. pH OF SOILS CTM 643

pH VALUE	7.39

REMARKS :

CORROSION TESTS

WILLDAN Geotechnical | extending your reach

Project Name: Boyle Heights Sports Center | Willdan Project No.: 106965-2000

'R' VALUE CA 301

Client: Willdan Geotechnical | Date: 9/25/17 | By: LD

Client's Job No.: 106965-2000 | Sample No.: B-2 @ 1' - 5'

GLA Reference: 2005-224 | Soil Type: Clayey SAND (SC)

TEST SPECIMEN		A	B	C	D
Compactor Air Pressure	psi	**350**	**130**	**250**	
Initial Moisture Content	%	**6.7**	**6.7**	**6.7**	
Water Added	ml	**50**	**70**	**60**	
Moisture at Compaction	%	11.1	12.9	12.0	
Sample & Mold Weight	gms	**3205**	**3214**	**3203**	
Mold Weight	gms	**2103**	**2098**	**2103**	
Net Sample Weight	gms	1102	1116	1100	
Sample Height	in.	**2.45**	**2.509**	**2.448**	
Dry Density	pcf	122.6	119.4	121.5	
Pressure	lbs	**7475**	**3310**	**4980**	
Exudation Pressure	psi	595	264	396	
Expansion Dial	x 0.0001	**50**	**10**	**26**	
Expansion Pressure	psf	217	43	113	
Ph at 1000lbs	psi	**21**	**28**	**24**	
Ph at 2000lbs	psi	**40**	**58**	**49**	
Displacement	turns	**3.57**	**4.45**	**4.08**	
R' Value		68	50	58	
Corrected 'R' Value		**68**	**50**	**58**	

FINAL 'R' VALUE	
By Exudation Pressure (@ 300 psi):	**52**
By Epansion Pressure :	**53**
TI = 5	

Geo-Logic ASSOCIATES

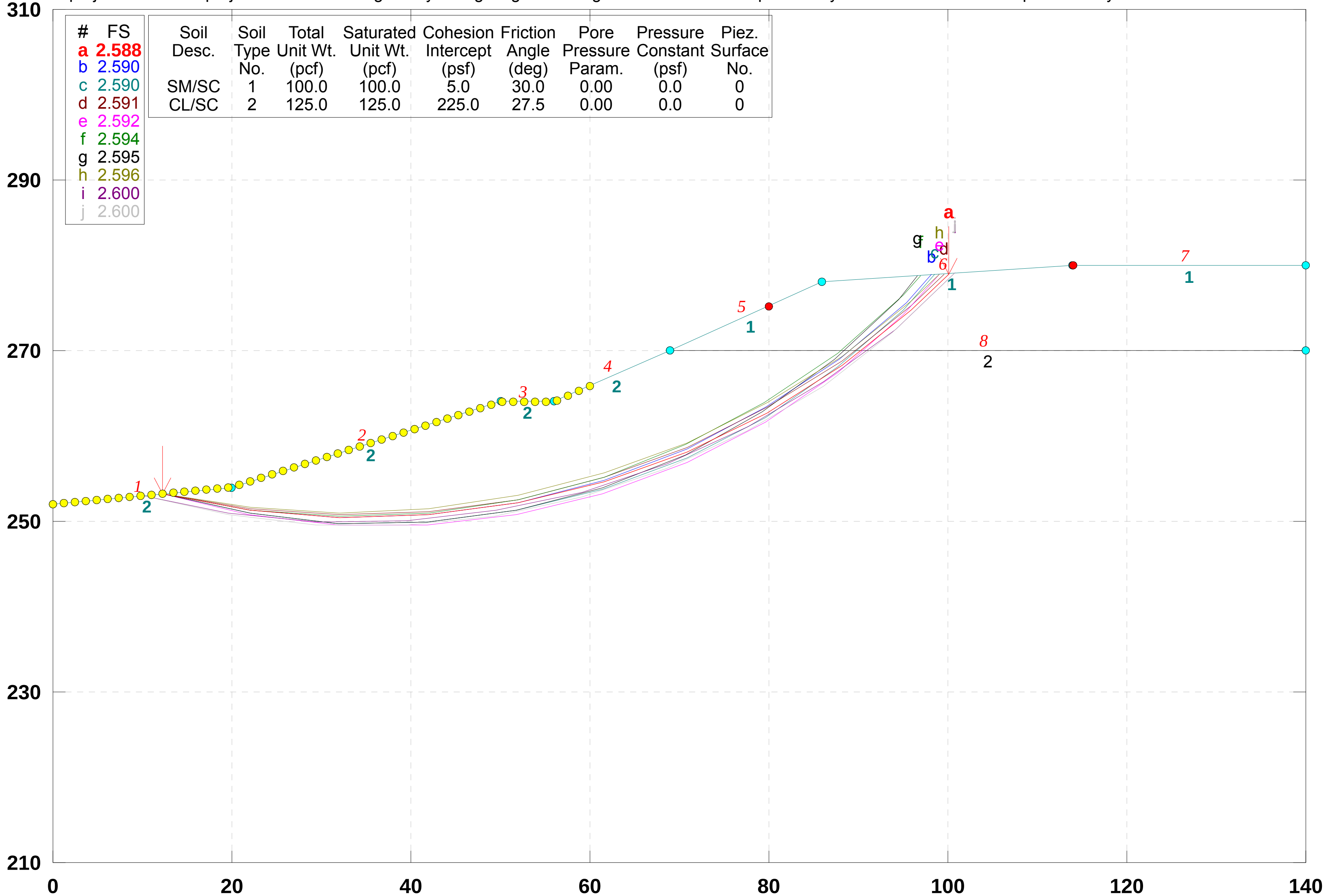
R-VALUE TEST
(CTM 301)

WILLDAN Geotechnical | extending your reach

APPENDIX C. SLOPE STABILTY ANALYSES

Cross Section A-A' Static Condition

q:\all projects\active projects\17 active projects\106965-2000 geo boyle heights geo investigation\calculations\slope stability\cross section a-a' static.pl2 Run By: Username 10/4/2017 12:20PM



#	FS
a	2.588
b	2.590
c	2.590
d	2.591
e	2.592
f	2.594
g	2.595
h	2.596
i	2.600
j	2.600

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
SM/SC	1	100.0	100.0	5.0	30.0	0.00	0.0	0
CL/SC	2	125.0	125.0	225.0	27.5	0.00	0.0	0

GSTABL7 v.2 FSmin=2.588

Safety Factors Are Calculated By The Modified Bishop Method

***** GSTABL7 *****

** GSTABL7 by Dr. Garry H. Gregory, Ph.D.,P.E.,D.GE **
** Original Version 1.0, January 1996; Current Ver. 2.005.3, Feb. 2013 **
(All Rights Reserved-Unauthorized Use Prohibited)
****
SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
****

Analysis Run Date: 10/4/2017
Time of Run: 12:20PM
Run By: Username
Input Data Filename: Q:\All Projects\Active Projects\17 Active Projects\106965-20
00 GEO Boyle Heights Geo Investigation\Calculations\Slope Stability\cross section a-a' static-modifi
Output Filename: Q:\All Projects\Active Projects\17 Active Projects\106965-20
00 GEO Boyle Heights Geo Investigation\Calculations\Slope Stability\cross section a-a' static-modifi
Unit System: English
Plotted Output Filename: Q:\All Projects\Active Projects\17 Active Projects\106965-20
00 GEO Boyle Heights Geo Investigation\Calculations\Slope Stability\cross section a-a' static-modifi
PROBLEM DESCRIPTION: Cross Section A-A'
Static Condition

BOUNDARY COORDINATES
7 Top Boundaries
8 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	252.00	20.00	254.00	2
2	20.00	254.00	50.00	264.00	2
3	50.00	264.00	56.00	264.00	2
4	56.00	264.00	69.00	270.00	2
5	69.00	270.00	86.00	278.00	1
6	86.00	278.00	114.00	280.00	1
7	114.00	280.00	140.00	280.00	1
8	69.00	270.00	140.00	270.00	2

User Specified Y-Origin = 210.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	100.0	100.0	5.0	30.0	0.00	0.0	0
2	125.0	125.0	225.0	27.5	0.00	0.0	0

A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.
25000 Trial Surfaces Have Been Generated.
500 Surface(s) Initiate(s) From Each Of 50 Points Equally Spaced
Along The Ground Surface Between X = 0.00(ft)
and X = 60.00(ft)
Each Surface Terminates Between X = 80.00(ft)
and X = 114.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 220.00(ft)
10.00(ft) Line Segments Define Each Trial Failure Surface.
Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By The Modified Bishop Method * *
Total Number of Trial Surfaces Attempted = 0
Number of Trial Surfaces With Valid FS = 0
Statistical Data On All Valid FS Values:
FS Max = 0.000 FS Min = 500.000 FS Ave = NaN
Standard Deviation = 0.000 Coefficient of Variation = NaN %
Failure Surface Specified By 11 Coordinate Points
Point X-Surf Y-Surf

No.	(ft)	(ft)
1	12.245	253.224
2	22.059	251.304
3	32.025	250.483
4	42.021	250.771
5	51.923	252.165
6	61.610	254.649
7	70.962	258.190
8	79.864	262.746
9	88.206	268.261
10	95.885	274.666
11	100.045	279.003

Circle Center At X = 34.429 ; Y = 340.542 ; and Radius = 90.092

Factor of Safety

*** 2.588 ***

Individual data on the 16 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	7.8	1111.5	0.0	0.0	0.	0.	0.0	0.0	0.0
2	2.1	730.3	0.0	0.0	0.	0.	0.0	0.0	0.0
3	10.0	6794.6	0.0	0.0	0.	0.	0.0	0.0	0.0
4	10.0	11304.7	0.0	0.0	0.	0.	0.0	0.0	0.0
5	8.0	11307.9	0.0	0.0	0.	0.	0.0	0.0	0.0
6	1.9	2877.4	0.0	0.0	0.	0.	0.0	0.0	0.0
7	4.1	5764.8	0.0	0.0	0.	0.	0.0	0.0	0.0
8	5.6	7969.5	0.0	0.0	0.	0.	0.0	0.0	0.0
9	7.4	11313.1	0.0	0.0	0.	0.	0.0	0.0	0.0
10	2.0	3077.6	0.0	0.0	0.	0.	0.0	0.0	0.0
11	8.9	13292.6	0.0	0.0	0.	0.	0.0	0.0	0.0
12	6.1	8031.4	0.0	0.0	0.	0.	0.0	0.0	0.0
13	2.2	2462.2	0.0	0.0	0.	0.	0.0	0.0	0.0
14	2.1	1943.2	0.0	0.0	0.	0.	0.0	0.0	0.0
15	5.6	3453.5	0.0	0.0	0.	0.	0.0	0.0	0.0
16	4.2	840.4	0.0	0.0	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	12.245	253.224
2	22.050	251.261
3	32.016	250.439
4	42.011	250.771
5	51.901	252.250
6	61.555	254.858
7	70.844	258.560
8	79.646	263.307
9	87.843	269.035
10	95.325	275.669
11	98.191	278.871

Circle Center At X = 34.147 ; Y = 337.138 ; and Radius = 86.725

Factor of Safety

*** 2.590 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	12.245	253.224
2	21.974	250.911
3	31.912	249.799
4	41.911	249.906
5	51.823	251.229
6	61.500	253.750
7	70.798	257.431
8	79.578	262.217
9	87.711	268.036
10	95.073	274.803
11	98.560	278.897

Circle Center At X = 36.038 ; Y = 331.652 ; and Radius = 81.957

Factor of Safety

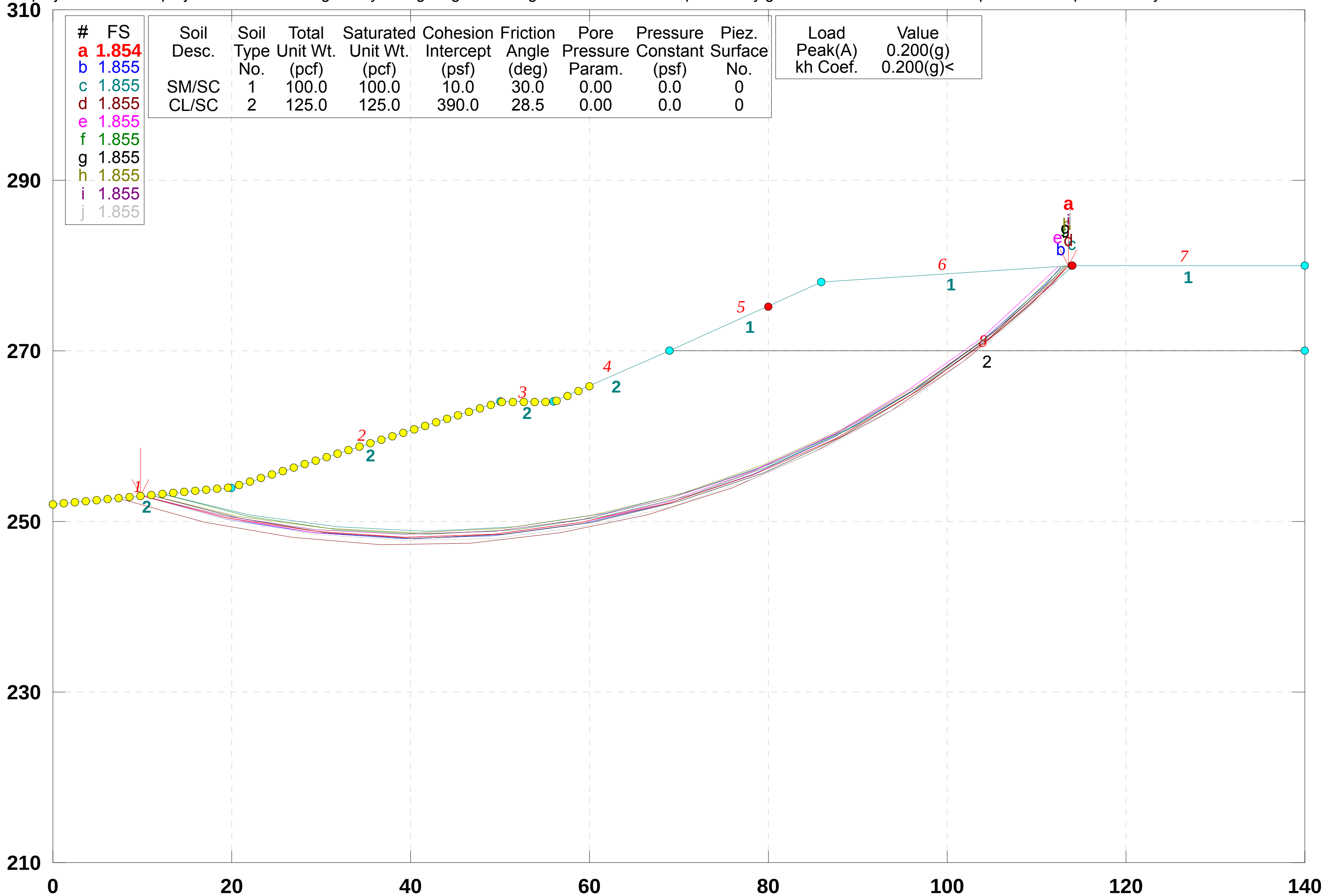
*** 2.590 ***

```
Failure Surface Specified By 11 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         12.245      253.224
    2         22.084      251.440
    3         32.060      250.737
    4         42.052      251.124
    5         51.943      252.597
    6         61.615      255.138
    7         70.953      258.716
    8         79.845      263.290
    9         88.187      268.805
   10         95.879      275.195
   11         99.525      278.966
Circle Center At X =    33.510 ; Y =   342.446 ; and Radius =    91.720
       Factor of Safety
      ***    2.591   ***
Failure Surface Specified By 11 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         12.245      253.224
    2         21.942      250.782
    3         31.867      249.556
    4         41.867      249.564
    5         51.789      250.806
    6         61.483      253.263
    7         70.799      256.898
    8         79.595      261.654
    9         87.736      267.461
   10         95.099      274.227
   11         99.096      278.935
Circle Center At X =    36.804 ; Y =   330.269 ; and Radius =    80.865
       Factor of Safety
      ***    2.592   ***
Failure Surface Specified By 11 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         12.245      253.224
    2         22.059      251.307
    3         32.030      250.545
    4         42.022      250.950
    5         51.899      252.515
    6         61.526      255.219
    7         70.773      259.025
    8         79.515      263.883
    9         87.631      269.725
   10         95.011      276.472
   11         97.015      278.787
Circle Center At X =    33.565 ; Y =   336.278 ; and Radius =    85.746
       Factor of Safety
      ***    2.594   ***
Failure Surface Specified By 11 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         12.245      253.224
    2         21.964      250.870
    3         31.902      249.762
    4         41.901      249.920
    5         51.800      251.341
    6         61.439      254.001
    7         70.665      257.858
    8         79.330      262.851
    9         87.294      268.899
   10         94.429      275.905
   11         96.684      278.763
Circle Center At X =    35.658 ; Y =   328.625 ; and Radius =    78.952
       Factor of Safety
      ***    2.595   ***
Failure Surface Specified By 11 Coordinate Points
  Point      X-Surf      Y-Surf
```

```
   No.        (ft)        (ft)
    1         12.245      253.224
    2         22.108      251.578
    3         32.091      250.994
    4         42.080      251.480
    5         51.959      253.030
    6         61.616      255.627
    7         70.940      259.241
    8         79.825      263.830
    9         88.169      269.341
   10         95.876      275.713
   11         99.013      278.930
Circle Center At X =    32.548 ; Y =   344.469 ; and Radius =    93.476
       Factor of Safety
      ***    2.596   ***
Failure Surface Specified By 11 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1          9.796      252.980
    2         19.583      250.925
    3         29.534      249.943
    4         39.534      250.045
    5         49.463      251.231
    6         59.206      253.485
    7         68.647      256.782
    8         77.674      261.083
    9         86.183      266.337
   10         94.072      272.482
   11        100.851      279.061
Circle Center At X =    33.593 ; Y =   341.988 ; and Radius =    92.135
       Factor of Safety
      ***    2.600   ***
Failure Surface Specified By 11 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1          9.796      252.980
    2         19.534      250.707
    3         29.467      249.549
    4         39.467      249.521
    5         49.406      250.623
    6         59.157      252.842
    7         68.595      256.148
    8         77.598      260.499
    9         86.052      265.840
   10         93.849      272.102
   11        100.737      279.053
Circle Center At X =    34.713 ; Y =   337.744 ; and Radius =    88.351
       Factor of Safety
      ***    2.600   ***
          **** END OF GSTABL7 OUTPUT ****
```

Cross Section A-A' Pseudo Static Condition

q:\all projects\active projects\17 active projects\106965-2000 geo boyle heights geo investigation\calculations\slope stability\gstabl files\cross section a-a' pseudostatic.pl2   Run By: Username   10/4/2017   01:



GSTABL7 v.2 FSmin=1.854

Safety Factors Are Calculated By The Modified Bishop Method

***** GSTABL7 *****

```
        ** GSTABL7 by Dr. Garry H. Gregory, Ph.D.,P.E.,D.GE **
   ** Original Version 1.0, January 1996; Current Ver. 2.005.3, Feb. 2013 **
            (All Rights Reserved-Unauthorized Use Prohibited)
 *********************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
        Modified Bishop, Simplified Janbu, or GLE Method of Slices.
        (Includes Spencer & Morgenstern-Price Type Analysis)
        Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
        Nonlinear Undrained Shear Strength, Curved Phi Envelope,
        Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
        Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
 *********************************************************************************
 Analysis Run Date:         10/4/2017
 Time of Run:               01:36PM
 Run By:                    Username
 Input Data Filename:       q:\All Projects\Active Projects\17 Active Projects\106965-20
00 GEO Boyle Heights Geo Investigation\Calculations\Slope Stability\GSTABL Files\cross section a-a'
 Output Filename:           q:\All Projects\Active Projects\17 Active Projects\106965-20
00 GEO Boyle Heights Geo Investigation\Calculations\Slope Stability\GSTABL Files\cross section a-a'
 Unit System:               English
 Plotted Output Filename:   q:\All Projects\Active Projects\17 Active Projects\106965-20
00 GEO Boyle Heights Geo Investigation\Calculations\Slope Stability\GSTABL Files\cross section a-a'
 PROBLEM DESCRIPTION:  Cross Section A-A'
                        Pseudo Static Condition
 BOUNDARY COORDINATES
     7 Top   Boundaries
     8 Total Boundaries
 Boundary     X-Left     Y-Left    X-Right    Y-Right    Soil Type
    No.        (ft)       (ft)       (ft)       (ft)     Below Bnd
     1          0.00     252.00      20.00     254.00        2
     2         20.00     254.00      50.00     264.00        2
     3         50.00     264.00      56.00     264.00        2
     4         56.00     264.00      69.00     270.00        2
     5         69.00     270.00      86.00     278.00        1
     6         86.00     278.00     114.00     280.00        1
     7        114.00     280.00     140.00     280.00        1
     8         69.00     270.00     140.00     270.00        2
 User Specified Y-Origin =       210.00(ft)
 Default X-Plus Value = 0.00(ft)
 Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  2 Type(s) of Soil
 Soil  Total  Saturated  Cohesion Friction   Pore   Pressure   Piez.
 Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
  No.  (pcf)    (pcf)     (psf)     (deg)    Param.   (psf)     No.
   1   100.0    100.0      10.0     30.0    0.00       0.0      0
   2   125.0    125.0     390.0     28.5    0.00       0.0      0
 Specified Peak Ground Acceleration Coefficient (A) =   0.200(g)
 Specified Horizontal Earthquake Coefficient (kh) =   0.200(g)
 Specified Vertical Earthquake Coefficient (kv) =   0.000(g)
 Specified Seismic Pore-Pressure Factor =   0.000
 A Critical Failure Surface Searching Method, Using A Random
 Technique For Generating Circular Surfaces, Has Been Specified.
25000 Trial Surfaces Have Been Generated.
  500 Surface(s) Initiate(s) From Each Of    50 Points Equally Spaced
 Along The Ground Surface Between  X =   0.00(ft)
                                and  X =  60.00(ft)
 Each Surface Terminates Between   X =  80.00(ft)
                               and   X = 114.00(ft)
 Unless Further Limitations Were Imposed, The Minimum Elevation
 At Which A Surface Extends Is  Y =    220.00(ft)
 10.00(ft) Line Segments Define Each Trial Failure Surface.
 Following Are Displayed The Ten Most Critical Of The Trial
       Failure Surfaces Evaluated. They Are
       Ordered - Most Critical First.
       * * Safety Factors Are Calculated By The Modified Bishop Method * *
       Total Number of Trial Surfaces Attempted =     0
       Number of Trial Surfaces With Valid FS =    0
       Statistical Data On All Valid FS Values:
```

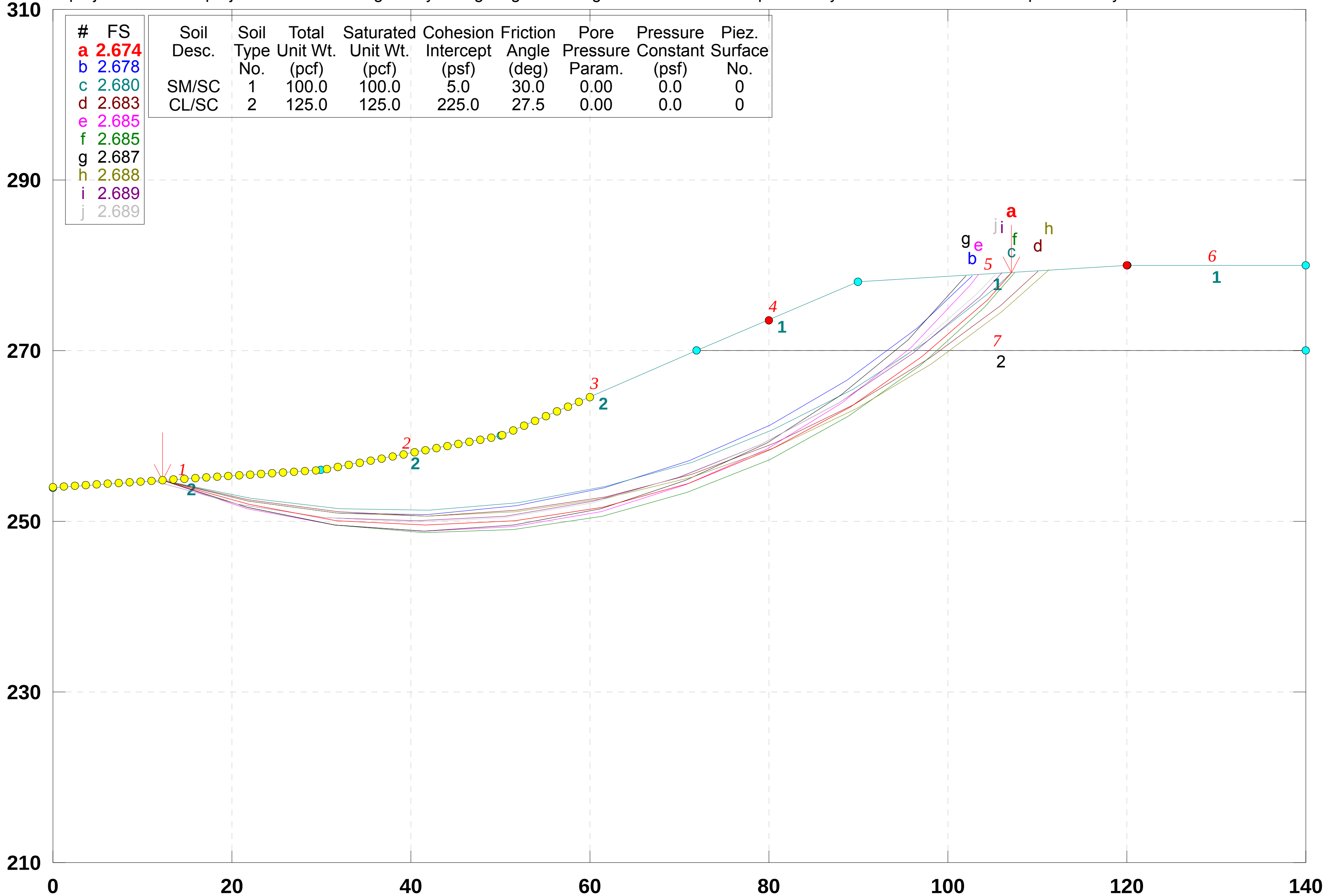
```
    FS Max =   0.000   FS Min = 500.000   FS Ave =  NaN
    Standard Deviation =    0.000   Coefficient of Variation =  NaN    %
 Failure Surface Specified By 13 Coordinate Points
   Point      X-Surf      Y-Surf
    No.        (ft)        (ft)
     1          9.796      252.980
     2         19.455      250.390
     3         29.324      248.775
     4         39.304      248.153
     5         49.297      248.527
     6         59.203      249.896
     7         68.923      252.245
     8         78.361      255.551
     9         87.422      259.781
    10         96.017      264.893
    11        104.059      270.836
    12        111.470      277.551
    13        113.661      279.976
 Circle Center At X =    40.549 ; Y =   348.370 ; and Radius =   100.225
        Factor of Safety
       ***    1.854   ***
      Individual data on the     0  slices
                  Water  Water     Tie      Tie     Earthquake
                  Force  Force    Force   Force       Force   Surcharge
Slice  Width   Weight    Top    Bot     Norm     Tan     Hor      Ver    Load
 No.    (ft)    (lbs)   (lbs)  (lbs)    (lbs)   (lbs)   (lbs)   (lbs)   (lbs)
 Failure Surface Specified By 13 Coordinate Points
   Point      X-Surf      Y-Surf
    No.        (ft)        (ft)
     1          9.796      252.980
     2         19.433      250.311
     3         29.293      248.642
     4         39.272      247.989
     5         49.265      248.361
     6         59.168      249.752
     7         68.876      252.149
     8         78.289      255.525
     9         87.307      259.847
    10         95.836      265.067
    11        103.786      271.133
    12        111.075      277.980
    13        112.749      279.911
 Circle Center At X =    40.645 ; Y =   345.652 ; and Radius =    97.672
        Factor of Safety
       ***    1.855   ***
 Failure Surface Specified By 13 Coordinate Points
   Point      X-Surf      Y-Surf
    No.        (ft)        (ft)
     1         12.245      253.224
     2         21.947      250.801
     3         31.842      249.356
     4         41.832      248.903
     5         51.817      249.448
     6         61.698      250.984
     7         71.377      253.497
     8         80.758      256.962
     9         89.747      261.343
    10         98.255      266.598
    11        106.197      272.674
    12        113.495      279.511
    13        113.916      279.994
 Circle Center At X =    41.369 ; Y =   349.184 ; and Radius =   100.281
        Factor of Safety
       ***    1.855   ***
 Failure Surface Specified By 13 Coordinate Points
   Point      X-Surf      Y-Surf
    No.        (ft)        (ft)
     1          7.347      252.735
     2         16.949      249.940
     3         26.782      248.122
```

```
    4         36.748      247.299
    5         46.746      247.479
    6         56.676      248.659
    7         66.438      250.829
    8         75.933      253.967
    9         85.066      258.040
   10         93.745      263.008
   11        101.882      268.821
   12        109.395      275.420
   13        113.636      279.974
Circle Center At X =    39.959 ; Y =   346.903 ; and Radius =    99.655
       Factor of Safety
      ***    1.855   ***
Failure Surface Specified By 13 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1          9.796      252.980
    2         19.445      250.353
    3         29.312      248.726
    4         39.293      248.115
    5         49.284      248.526
    6         59.182      249.955
    7         68.881      252.388
    8         78.282      255.798
    9         87.285      260.150
   10         95.797      265.399
   11        103.728      271.489
   12        110.996      278.358
   13        112.307      279.879
Circle Center At X =    40.273 ; Y =   345.914 ; and Radius =    97.804
       Factor of Safety
      ***    1.855   ***
Failure Surface Specified By 13 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         12.245      253.224
    2         21.905      250.638
    3         31.780      249.067
    4         41.766      248.528
    5         51.753      249.029
    6         61.635      250.562
    7         71.305      253.112
    8         80.657      256.651
    9         89.593      261.141
   10         98.014      266.533
   11        105.831      272.770
   12        112.959      279.784
   13        113.083      279.935
Circle Center At X =    41.949 ; Y =   344.820 ; and Radius =    96.292
       Factor of Safety
      ***    1.855   ***
Failure Surface Specified By 13 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         11.020      253.102
    2         20.646      250.391
    3         30.501      248.697
    4         40.480      248.038
    5         50.472      248.420
    6         60.371      249.841
    7         70.068      252.284
    8         79.458      255.723
    9         88.439      260.121
   10         96.914      265.430
   11        104.790      271.591
   12        111.982      278.539
   13        113.158      279.940
Circle Center At X =    41.808 ; Y =   343.982 ; and Radius =    95.953
       Factor of Safety
      ***    1.855   ***
```

```
Failure Surface Specified By 13 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1          11.020      253.102
    2          20.723      250.682
    3          30.617      249.230
    4          40.606      248.761
    5          50.593      249.279
    6          60.479      250.780
    7          70.170      253.248
    8          79.570      256.660
    9          88.588      260.982
   10          97.135      266.172
   11         105.129      272.180
   12         112.492      278.947
   13         113.394      279.957
Circle Center At X =    40.356 ; Y =   350.044 ; and Radius =   101.284
       Factor of Safety
      ***    1.855   ***
Failure Surface Specified By 13 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1           9.796      252.980
    2          19.488      250.516
    3          29.373      249.009
    4          39.359      248.474
    5          49.349      248.915
    6          59.249      250.328
    7          68.964      252.701
    8          78.400      256.009
    9          87.469      260.223
   10          96.084      265.300
   11         104.163      271.194
   12         111.628      277.848
   13         113.586      279.970
Circle Center At X =    39.842 ; Y =   350.874 ; and Radius =   102.402
       Factor of Safety
      ***    1.855   ***
Failure Surface Specified By 13 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1           9.796      252.980
    2          19.409      250.224
    3          29.253      248.467
    4          39.226      247.726
    5          49.222      248.010
    6          59.136      249.315
    7          68.865      251.629
    8          78.306      254.925
    9          87.360      259.171
   10          95.932      264.321
   11         103.931      270.322
   12         111.275      277.109
   13         113.810      279.986
Circle Center At X =    41.457 ; Y =   345.282 ; and Radius =    97.581
       Factor of Safety
      ***    1.855   ***
          **** END OF GSTABL7 OUTPUT ****
```

Cross Section B-B' Static Condition

q:\all projects\active projects\17 active projects\106965-2000 geo boyle heights geo investigation\calculations\slope stability\cross section b-b' static-.pl2 Run By: Username 10/4/2017 12:15PM



GSTABL7 v.2 FSmin=2.674

Safety Factors Are Calculated By The Modified Bishop Method

```
                          ***  GSTABL7  ***
         ** GSTABL7 by Dr. Garry H. Gregory, Ph.D.,P.E.,D.GE **
   ** Original Version 1.0, January 1996; Current Ver. 2.005.3, Feb. 2013 **
            (All Rights Reserved-Unauthorized Use Prohibited)
 *********************************************************************************
                   SLOPE STABILITY ANALYSIS SYSTEM
       Modified Bishop, Simplified Janbu, or GLE Method of Slices.
       (Includes Spencer & Morgenstern-Price Type Analysis)
       Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
       Nonlinear Undrained Shear Strength, Curved Phi Envelope,
       Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
       Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
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 Run By:                   Username
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00 GEO Boyle Heights Geo Investigation\Calculations\Slope Stability\cross section b-b' static-modifi
 Output Filename:          Q:\All Projects\Active Projects\17 Active Projects\106965-20
00 GEO Boyle Heights Geo Investigation\Calculations\Slope Stability\cross section b-b' static-modifi
 Unit System:              English
 Plotted Output Filename:  Q:\All Projects\Active Projects\17 Active Projects\106965-20
00 GEO Boyle Heights Geo Investigation\Calculations\Slope Stability\cross section b-b' static-modifi
 PROBLEM DESCRIPTION:  Cross Section B-B'
                       Static Condition
 BOUNDARY COORDINATES
     6 Top   Boundaries
     7 Total Boundaries
 Boundary    X-Left     Y-Left    X-Right    Y-Right    Soil Type
    No.       (ft)       (ft)       (ft)       (ft)     Below Bnd
     1         0.00     254.00      30.00     256.00        2
     2        30.00     256.00      50.00     260.00        2
     3        50.00     260.00      72.00     270.00        2
     4        72.00     270.00      90.00     278.00        1
     5        90.00     278.00     120.00     280.00        1
     6       120.00     280.00     140.00     280.00        1
     7        72.00     270.00     140.00     270.00        2
 User Specified Y-Origin =       210.00(ft)
 Default X-Plus Value = 0.00(ft)
 Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  2 Type(s) of Soil
 Soil  Total  Saturated  Cohesion Friction   Pore   Pressure   Piez.
 Type Unit Wt. Unit Wt. Intercept  Angle  Pressure Constant Surface
  No.  (pcf)    (pcf)     (psf)     (deg)    Param.   (psf)     No.
   1   100.0    100.0       5.0     30.0    0.00       0.0      0
   2   125.0    125.0     225.0     27.5    0.00       0.0      0
 EARTHQUAKE DATA HAS BEEN SUPPRESSED
 A Critical Failure Surface Searching Method, Using A Random
 Technique For Generating Circular Surfaces, Has Been Specified.
25000 Trial Surfaces Have Been Generated.
  500 Surface(s) Initiate(s) From Each Of    50 Points Equally Spaced
 Along The Ground Surface Between  X =   0.00(ft)
                              and  X =  60.00(ft)
 Each Surface Terminates Between   X =  80.00(ft)
                             and   X = 120.00(ft)
 Unless Further Limitations Were Imposed, The Minimum Elevation
 At Which A Surface Extends Is  Y =    220.00(ft)
 10.00(ft) Line Segments Define Each Trial Failure Surface.
 Following Are Displayed The Ten Most Critical Of The Trial
       Failure Surfaces Evaluated. They Are
       Ordered - Most Critical First.
       * * Safety Factors Are Calculated By The Modified Bishop Method * *
       Total Number of Trial Surfaces Attempted =     0
       Number of Trial Surfaces With Valid FS =    0
       Statistical Data On All Valid FS Values:
          FS Max =   0.000   FS Min = 500.000   FS Ave =  NaN
          Standard Deviation =    0.000   Coefficient of Variation =  NaN    %
       Failure Surface Specified By 12 Coordinate Points
         Point      X-Surf      Y-Surf
```

Point No.	X-Surf (ft)	Y-Surf (ft)
1	12.245	254.816
2	21.815	251.916
3	31.656	250.141
4	41.636	249.512
5	51.623	250.040
6	61.481	251.717
7	71.080	254.520
8	80.291	258.413
9	88.992	263.342
10	97.065	269.243
11	104.404	276.036
12	107.060	279.137

Circle Center At X = 42.047 ; Y = 335.613 ; and Radius = 86.117

Factor of Safety

*** 2.674 ***

Individual data on the 16 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	9.6	2116.1	0.0	0.0	0.	0.	0.0	0.0	0.0
2	8.2	4654.3	0.0	0.0	0.	0.	0.0	0.0	0.0
3	1.7	1216.4	0.0	0.0	0.	0.	0.0	0.0	0.0
4	10.0	9360.1	0.0	0.0	0.	0.	0.0	0.0	0.0
5	8.4	9858.8	0.0	0.0	0.	0.	0.0	0.0	0.0
6	1.6	2103.6	0.0	0.0	0.	0.	0.0	0.0	0.0
7	9.9	14910.4	0.0	0.0	0.	0.	0.0	0.0	0.0
8	9.6	17136.2	0.0	0.0	0.	0.	0.0	0.0	0.0
9	0.9	1733.8	0.0	0.0	0.	0.	0.0	0.0	0.0
10	8.3	15352.6	0.0	0.0	0.	0.	0.0	0.0	0.0
11	8.7	14809.7	0.0	0.0	0.	0.	0.0	0.0	0.0
12	1.0	1576.5	0.0	0.0	0.	0.	0.0	0.0	0.0
13	7.1	8767.3	0.0	0.0	0.	0.	0.0	0.0	0.0
14	0.8	733.5	0.0	0.0	0.	0.	0.0	0.0	0.0
15	6.5	3733.0	0.0	0.0	0.	0.	0.0	0.0	0.0
16	2.7	388.3	0.0	0.0	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	12.245	254.816
2	21.939	252.362
3	31.850	251.027
4	41.848	250.830
5	51.803	251.773
6	61.586	253.843
7	71.070	257.014
8	80.131	261.245
9	88.652	266.480
10	96.520	272.651
11	102.800	278.853

Circle Center At X = 38.574 ; Y = 338.433 ; and Radius = 87.664

Factor of Safety

*** 2.678 ***

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	12.245	254.816
2	22.008	252.652
3	31.940	251.487
4	41.939	251.332
5	51.902	252.189
6	61.727	254.051
7	71.313	256.896
8	80.563	260.697
9	89.380	265.415
10	97.675	271.000
11	105.363	277.396
12	107.071	279.138

Circle Center At X = 38.459 ; Y = 349.629 ; and Radius = 98.370

Factor of Safety
*** 2.680 ***
Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	12.245	254.816
2	21.958	252.438
3	31.862	251.053
4	41.854	250.676
5	51.834	251.310
6	61.699	252.949
7	71.348	255.576
8	80.682	259.165
9	89.605	263.678
10	98.028	269.069
11	105.862	275.284
12	110.025	279.335

Circle Center At X = 40.567 ; Y = 349.098 ; and Radius = 98.444
Factor of Safety
*** 2.683 ***
Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	12.245	254.816
2	21.693	251.540
3	31.485	249.511
4	41.457	248.763
5	51.442	249.308
6	61.273	251.138
7	70.786	254.221
8	79.821	258.507
9	88.227	263.923
10	95.864	270.379
11	102.604	277.766
12	103.391	278.893

Circle Center At X = 42.228 ; Y = 325.873 ; and Radius = 77.124
Factor of Safety
*** 2.685 ***
Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	12.245	254.816
2	21.717	251.611
3	31.507	249.571
4	41.471	248.725
5	51.465	249.087
6	61.342	250.651
7	70.958	253.394
8	80.174	257.275
9	88.855	262.240
10	96.874	268.214
11	104.114	275.111
12	107.451	279.163

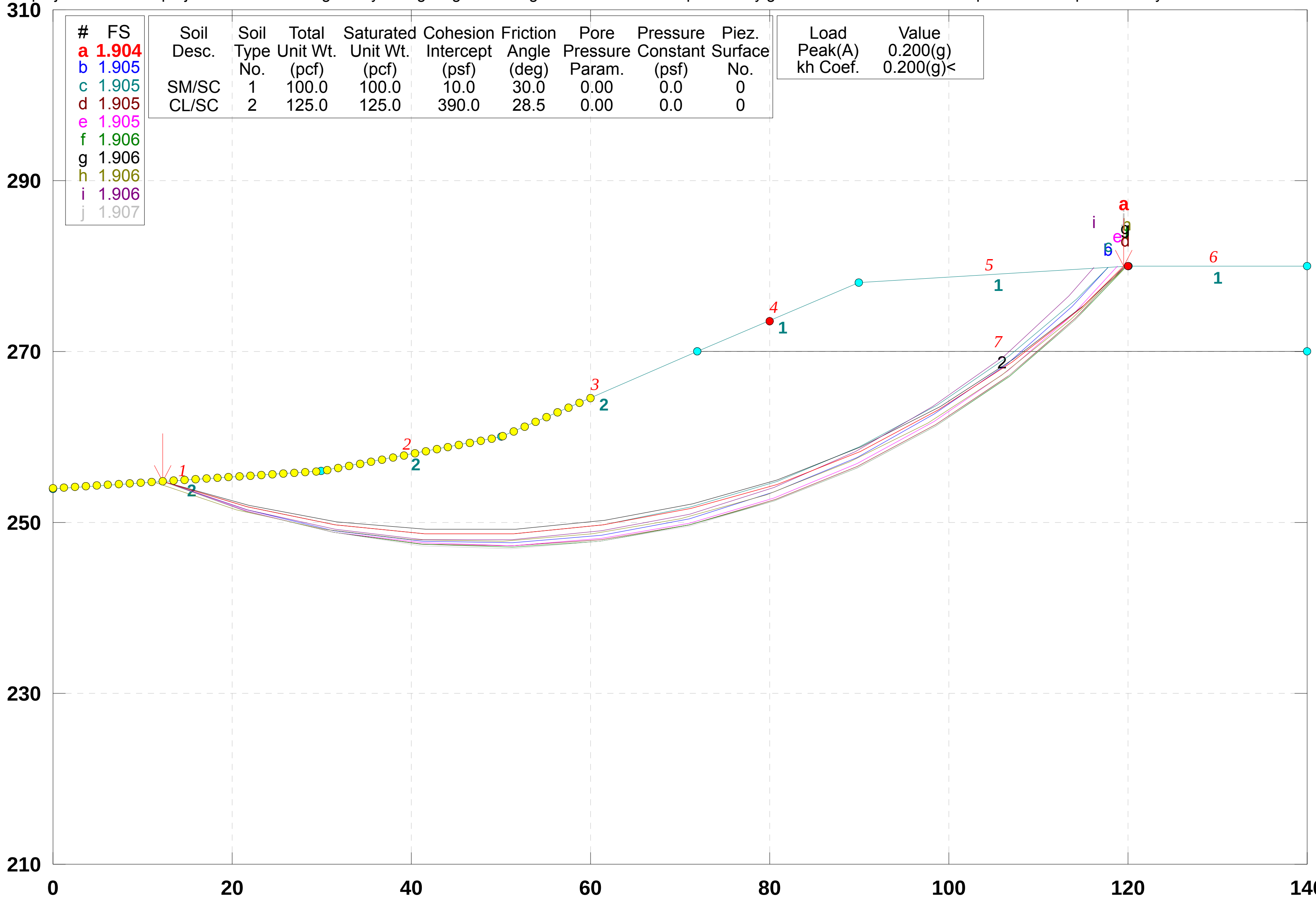
Circle Center At X = 43.445 ; Y = 331.132 ; and Radius = 82.447
Factor of Safety
*** 2.685 ***
Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	12.245	254.816
2	21.706	251.578
3	31.510	249.607
4	41.487	248.935
5	51.467	249.576
6	61.276	251.518
7	70.748	254.727
8	79.717	259.148
9	88.030	264.706
10	95.545	271.304
11	102.112	278.807

```
Circle Center At X =    41.604 ; Y =   325.158 ; and Radius =    76.223
       Factor of Safety
      ***    2.687   ***
Failure Surface Specified By 12 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         12.245      254.816
    2         21.947      252.395
    3         31.844      250.959
    4         41.834      250.521
    5         51.818      251.087
    6         61.695      252.651
    7         71.366      255.197
    8         80.732      258.699
    9         89.701      263.122
   10         98.181      268.422
   11        106.087      274.545
   12        111.216      279.414
Circle Center At X =    41.177 ; Y =   349.815 ; and Radius =    99.307
       Factor of Safety
      ***    2.688   ***
Failure Surface Specified By 12 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         11.020      254.735
    2         20.660      252.073
    3         30.534      250.494
    4         40.523      250.016
    5         50.503      250.645
    6         60.353      252.373
    7         69.951      255.180
    8         79.180      259.030
    9         87.927      263.876
   10         96.084      269.660
   11        103.553      276.311
   12        106.033      279.069
Circle Center At X =    39.822 ; Y =   339.936 ; and Radius =    89.938
       Factor of Safety
      ***    2.689   ***
Failure Surface Specified By 12 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         11.020      254.735
    2         20.638      251.994
    3         30.503      250.360
    4         40.490      249.854
    5         50.471      250.482
    6         60.315      252.237
    7         69.898      255.095
    8         79.096      259.020
    9         87.790      263.961
   10         95.868      269.855
   11        103.227      276.626
   12        105.300      279.020
Circle Center At X =    39.932 ; Y =   337.640 ; and Radius =    87.802
       Factor of Safety
      ***    2.689   ***
          **** END OF GSTABL7 OUTPUT ****
```

Cross Section B-B' Pseudo Static Condition

q:\all projects\active projects\17 active projects\106965-2000 geo boyle heights geo investigation\calculations\slope stability\gstabl files\cross section b-b' pseudo static.pl2 Run By: Username 10/4/2017 01:



GSTABL7 v.2 FSmin=1.904
Safety Factors Are Calculated By The Modified Bishop Method

***** GSTABL7 *****

** GSTABL7 by Dr. Garry H. Gregory, Ph.D.,P.E.,D.GE **
** Original Version 1.0, January 1996; Current Ver. 2.005.3, Feb. 2013 **
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****

SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

****

Analysis Run Date: 10/4/2017
Time of Run: 01:51PM
Run By: Username
Input Data Filename: q:\All Projects\Active Projects\17 Active Projects\106965-20
00 GEO Boyle Heights Geo Investigation\Calculations\Slope Stability\GSTABL Files\cross section b-b'
Output Filename: q:\All Projects\Active Projects\17 Active Projects\106965-20
00 GEO Boyle Heights Geo Investigation\Calculations\Slope Stability\GSTABL Files\cross section b-b'
Unit System: English
Plotted Output Filename: q:\All Projects\Active Projects\17 Active Projects\106965-20
00 GEO Boyle Heights Geo Investigation\Calculations\Slope Stability\GSTABL Files\cross section b-b'
PROBLEM DESCRIPTION: Cross Section B-B'
Pseudo Static Condition

BOUNDARY COORDINATES
6 Top Boundaries
7 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	254.00	30.00	256.00	2
2	30.00	256.00	50.00	260.00	2
3	50.00	260.00	72.00	270.00	2
4	72.00	270.00	90.00	278.00	1
5	90.00	278.00	120.00	280.00	1
6	120.00	280.00	140.00	280.00	1
7	72.00	270.00	140.00	270.00	2

User Specified Y-Origin = 210.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	100.0	100.0	10.0	30.0	0.00	0.0	0
2	125.0	125.0	390.0	28.5	0.00	0.0	0

Specified Peak Ground Acceleration Coefficient (A) = 0.200(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.200(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)
Specified Seismic Pore-Pressure Factor = 0.000
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.
25000 Trial Surfaces Have Been Generated.
500 Surface(s) Initiate(s) From Each Of 50 Points Equally Spaced
Along The Ground Surface Between X = 0.00(ft)
and X = 60.00(ft)
Each Surface Terminates Between X = 80.00(ft)
and X = 120.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 220.00(ft)
10.00(ft) Line Segments Define Each Trial Failure Surface.
Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By The Modified Bishop Method * *
Total Number of Trial Surfaces Attempted = 0
Number of Trial Surfaces With Valid FS = 0
Statistical Data On All Valid FS Values:
FS Max = 0.000 FS Min = 500.000 FS Ave = NaN

Standard Deviation = 0.000   Coefficient of Variation = NaN %

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	12.245	254.816
2	21.786	251.823
3	31.579	249.795
4	41.524	248.755
5	51.524	248.711
6	61.479	249.666
7	71.288	251.608
8	80.855	254.518
9	90.084	258.369
10	98.883	263.120
11	107.164	268.726
12	114.845	275.129
13	119.599	279.973

Circle Center At X = 46.959 ; Y = 348.751 ; and Radius = 100.144

Factor of Safety

*** 1.904 ***

Individual data on the 17 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	9.5	2164.6	0.0	0.0	0.	0.	432.9	0.0	0.0
2	8.2	4880.9	0.0	0.0	0.	0.	976.2	0.0	0.0
3	1.6	1223.3	0.0	0.0	0.	0.	244.7	0.0	0.0
4	9.9	9989.5	0.0	0.0	0.	0.	1997.9	0.0	0.0
5	8.5	11035.3	0.0	0.0	0.	0.	2207.1	0.0	0.0
6	1.5	2216.2	0.0	0.0	0.	0.	443.2	0.0	0.0
7	10.0	17129.9	0.0	0.0	0.	0.	3426.0	0.0	0.0
8	9.8	20613.0	0.0	0.0	0.	0.	4122.6	0.0	0.0
9	0.7	1612.4	0.0	0.0	0.	0.	322.5	0.0	0.0
10	8.9	20370.3	0.0	0.0	0.	0.	4074.1	0.0	0.0
11	9.1	20973.9	0.0	0.0	0.	0.	4194.8	0.0	0.0
12	0.1	190.0	0.0	0.0	0.	0.	38.0	0.0	0.0
13	8.8	17481.9	0.0	0.0	0.	0.	3496.4	0.0	0.0
14	8.3	11564.2	0.0	0.0	0.	0.	2312.8	0.0	0.0
15	1.5	1527.0	0.0	0.0	0.	0.	305.4	0.0	0.0
16	6.2	4237.0	0.0	0.0	0.	0.	847.4	0.0	0.0
17	4.8	1075.9	0.0	0.0	0.	0.	215.2	0.0	0.0

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	12.245	254.816
2	21.649	251.417
3	31.368	249.062
4	41.286	247.782
5	51.284	247.590
6	61.244	248.489
7	71.046	250.469
8	80.573	253.505
9	89.714	257.562
10	98.357	262.591
11	106.400	268.533
12	113.748	275.316
13	117.691	279.846

Circle Center At X = 48.040 ; Y = 339.126 ; and Radius = 91.594

Factor of Safety

*** 1.905 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	12.245	254.816
2	21.776	251.789
3	31.566	249.754
4	41.514	248.733
5	51.514	248.736
6	61.461	249.764

```
    7         71.250      251.806
    8         80.779      254.841
    9         89.946      258.836
   10         98.656      263.749
   11        106.817      269.528
   12        114.342      276.114
   13        117.822      279.855
Circle Center At X =    46.479 ; Y =   346.070 ; and Radius =    97.464
       Factor of Safety
      ***    1.905   ***
Failure Surface Specified By 13 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         12.245      254.816
    2         21.621      251.340
    3         31.318      248.895
    4         41.221      247.509
    5         51.216      247.199
    6         61.187      247.969
    7         71.016      249.808
    8         80.590      252.697
    9         89.796      256.601
   10         98.528      261.475
   11        106.684      267.261
   12        114.168      273.893
   13        119.700      279.980
Circle Center At X =    49.086 ; Y =   339.800 ; and Radius =    92.625
       Factor of Safety
      ***    1.905   ***
Failure Surface Specified By 13 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         12.245      254.816
    2         21.622      251.343
    3         31.322      248.910
    4         41.229      247.548
    5         51.225      247.272
    6         61.192      248.085
    7         71.011      249.978
    8         80.566      252.929
    9         89.742      256.902
   10         98.432      261.850
   11        106.532      267.714
   12        113.946      274.426
   13        118.825      279.922
Circle Center At X =    48.757 ; Y =   338.987 ; and Radius =    91.748
       Factor of Safety
      ***    1.905   ***
Failure Surface Specified By 13 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         12.245      254.816
    2         21.608      251.303
    3         31.295      248.824
    4         41.195      247.408
    5         51.189      247.072
    6         61.161      247.819
    7         70.994      249.642
    8         80.571      252.517
    9         89.781      256.412
   10         98.516      261.282
   11        106.672      267.068
   12        114.154      273.702
   13        119.860      279.991
Circle Center At X =    49.290 ; Y =   339.315 ; and Radius =    92.262
       Factor of Safety
      ***    1.906   ***
Failure Surface Specified By 13 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
```

1	12.245	254.816
2	21.833	251.977
3	31.652	250.080
4	41.608	249.142
5	51.608	249.174
6	61.557	250.174
7	71.364	252.134
8	80.934	255.034
9	90.178	258.847
10	99.010	263.538
11	107.345	269.063
12	115.106	275.369
13	119.778	279.985

Circle Center At X = 46.282 ; Y = 352.144 ; and Radius = 103.108

Factor of Safety

*** 1.906 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	11.020	254.735
2	20.487	251.513
3	30.233	249.273
4	40.157	248.038
5	50.154	247.820
6	60.122	248.623
7	69.956	250.437
8	79.554	253.245
9	88.816	257.015
10	97.645	261.710
11	105.950	267.281
12	113.644	273.669
13	119.844	279.990

Circle Center At X = 47.283 ; Y = 345.769 ; and Radius = 97.991

Factor of Safety

*** 1.906 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	12.245	254.816
2	21.676	251.492
3	31.415	249.221
4	41.344	248.033
5	51.344	247.940
6	61.293	248.945
7	71.072	251.035
8	80.563	254.184
9	89.651	258.356
10	98.227	263.499
11	106.188	269.552
12	113.436	276.441
13	116.227	279.748

Circle Center At X = 47.185 ; Y = 338.888 ; and Radius = 91.043

Factor of Safety

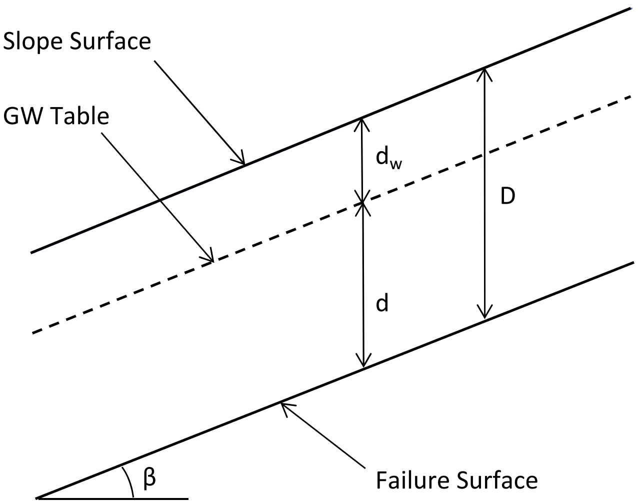
*** 1.906 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	12.245	254.816
2	21.595	251.270
3	31.276	248.764
4	41.173	247.331
5	51.167	246.986
6	61.139	247.733
7	70.969	249.565
8	80.542	252.458
9	89.741	256.379
10	98.457	261.281
11	106.587	267.104
12	114.032	273.780
13	119.579	279.972

```
Circle Center At X =    49.322 ; Y =   338.459 ; and Radius =    91.492
       Factor of Safety
      ***    1.907   ***
          **** END OF GSTABL7 OUTPUT ****
```

Calculation of Safety Factor for Surficial Slope Stability



FS = Factor of Safety

$$FS\ (\text{Effective Stress}) = \frac{C' + (\gamma D - \gamma_w d)\,(\cos^2\beta)\,(\tan\phi')}{\gamma D \sin\beta \times \cos\beta}$$

$$FS\ (\text{Total Stress}) = \frac{C + \gamma D\,(\cos^2\beta)\,(\tan\phi)}{\gamma D \sin\beta \times \cos\beta}$$

γ = (10 x 100 + 20 x 125) / 30 = 115 pcf

C = (10 x 5 + 20 x 225) / 30 = 150 psf

$\phi = \tan^{-1}$ [(10 x Tan 30 + 20 x Tan 27.5) / 30] = 28 degree

Slope Ratio, H:V =	2.2	
β =	24.4	degree
d_w =	0	ft
Unit Weight of Soil, γ =	115	pcf
Cohesion of Soil, Effective, c' =	150	psf
Cohesion of Soil, Total, c =	150	psf
Friction Angle of Soil, Effective, ϕ' =	28	degree
Friction Angle of Soil, Total, ϕ =	28	degree

D (ft)	d (ft)	FS	
		Effective Stress	Total Stress
0.50	0.50	7.47	8.11
1.00	1.00	4.00	4.64
1.50	1.50	2.85	3.48
2.00	2.00	2.27	2.91
2.50	2.50	1.92	2.56
3.00	3.00	1.69	2.33

Willdan Geotechnical	Project No. : 106965-2000	Date: 9/29/2017 By: MR

APPENDIX D. PERCOLATION TEST DATA SHEETS

Boring Percolation Testing (based on GS200.1, 12/31/14)

Project Name: Boyle Heights Sports Center Project, Los Angeles, California

Project No.: 106965-2000

Project Location:

Tested by: SM **Date Tested:** 6/28/2017

Boring/Test No.: TW-1/1

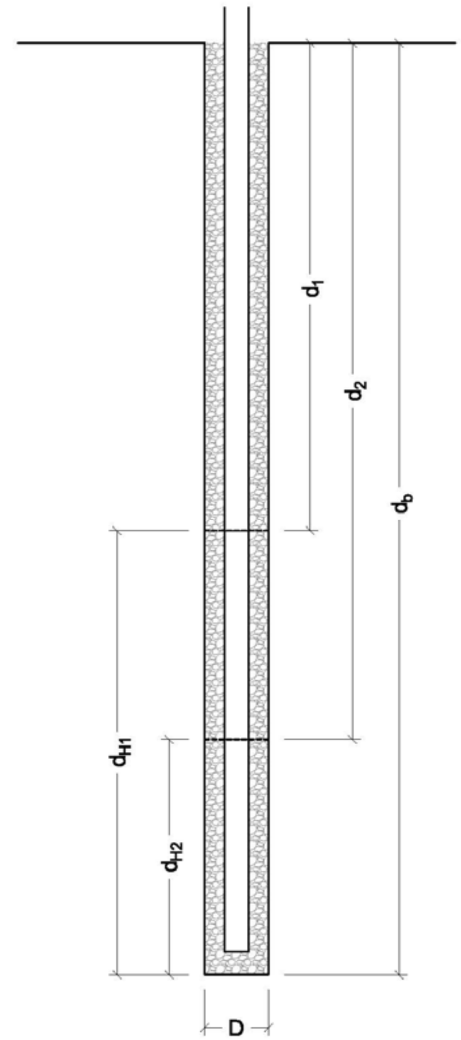
Depth of Boring, d_b (ft): 5.00

Diameter of Boring, D (in): 8

Water Table Depth (ft):

Initial Depth to Water, d_1 (ft): 0.35

Reading No.	Water Level Measurement			Water Level Calculations				Percolation Rate Calculations		
	Time Interval $\Delta T = T_2 - T_1$ (min)	Initial Depth to Water d_1 (ft)	Final Depth to Water d_2 (ft)	Initial Height of Water Colum $d_{H1} = d_b - d_1$ (in)	Final Height of Water Column $d_{H2} = d_b - d_2$ (in)	Drop in Height $\Delta d_H = d_{H1} - d_{H2}$ (in)	Average Height of Water Column $d_{avg} = (d_{H1}+d_{H2})/2$ (in)	Pre-adjusted Percolation Rate $K_i = \Delta d_H / \Delta T$ (in/hr)	Reduction Factor $R_f = ((2d_{H1} - \Delta d_H) / D) + 1$	Adjusted Percolation Rate $K = K_i / R_f$ (in/hr)
1	30	0.35	1.50	55.80	42.00	13.80	48.90	27.60	13.23	2.09
2	30	0.35	1.39	55.80	43.32	12.48	49.56	24.96	13.39	1.86
3	30	0.35	1.40	55.80	43.20	12.60	49.50	25.20	13.38	1.88
4	30	0.35	1.40	55.80	43.20	12.60	49.50	25.20	13.38	1.88
5	30	0.35	1.40	55.80	43.20	12.60	49.50	25.20	13.38	1.88



Adjusted Percolation Rate = 1.88 **in/hr**

Boring Percolation Testing (based on GS200.1, 12/31/14)

Project Name: Boyle Heights Sports Center Project, Los Angeles, California

Project No.: 106965-2000

Project Location:

Tested by: SM **Date Tested:** 6/28/2017

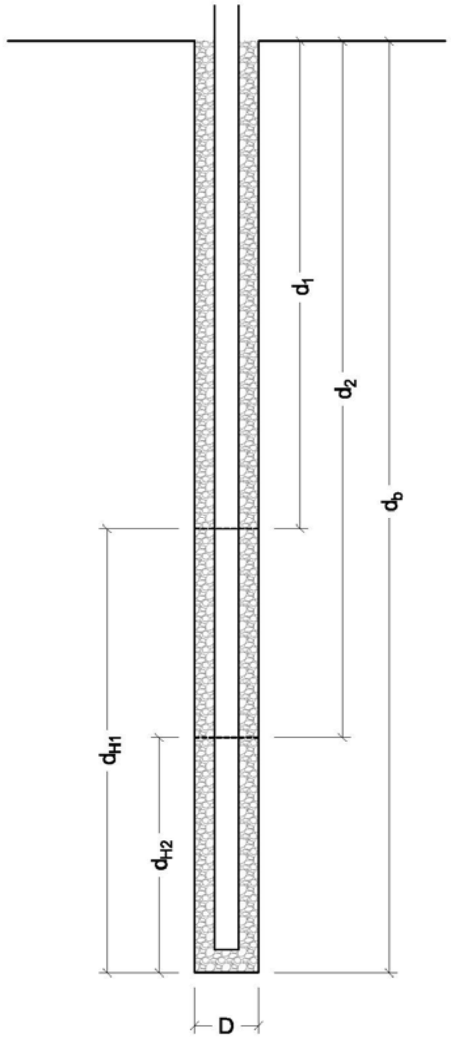
Boring/Test No.: TW-2/1

Depth of Boring, d_b (ft): 10.00

Diameter of Boring, D (in): 8

Water Table Depth (ft):

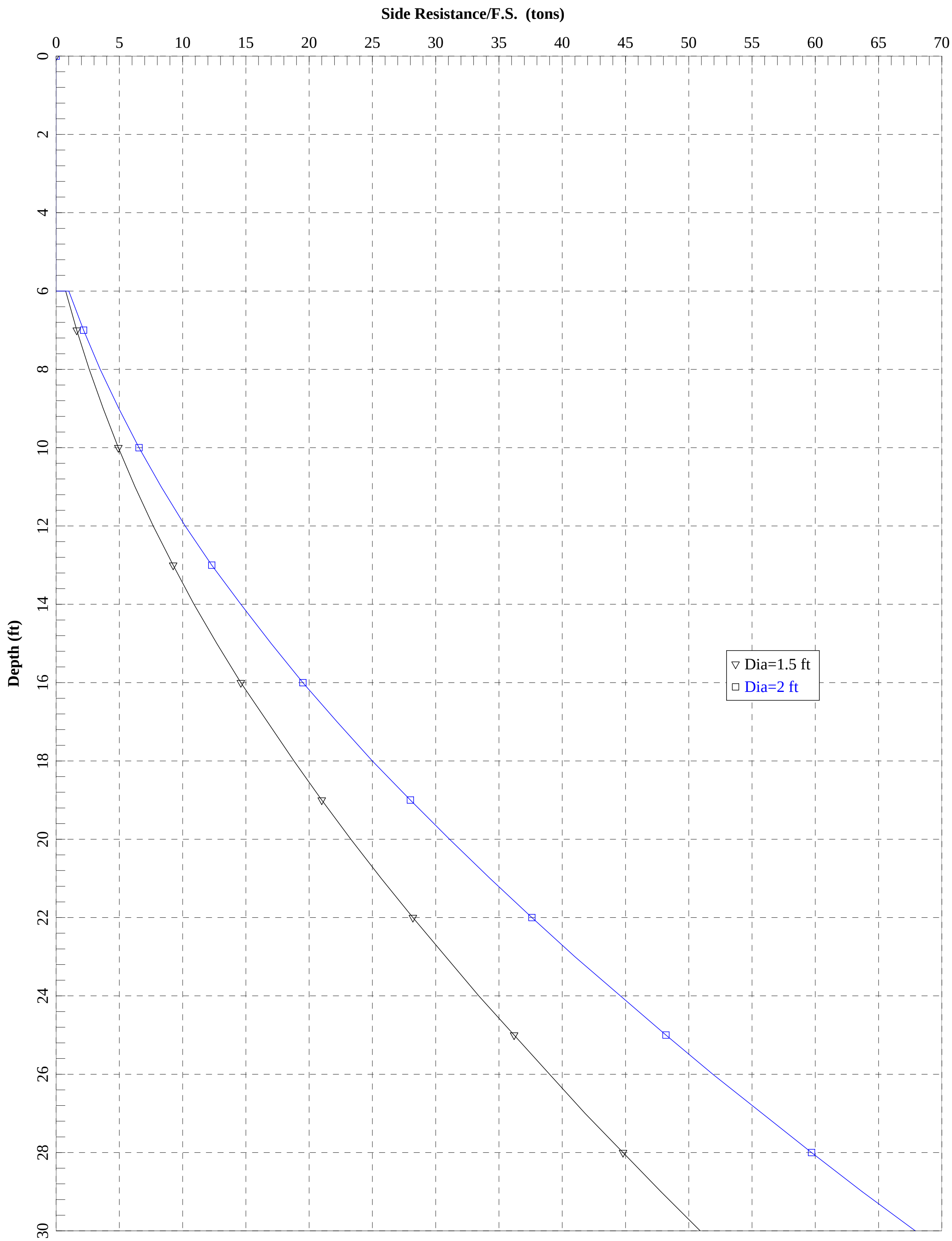
Initial Depth to Water, d_1 (ft): 0.35



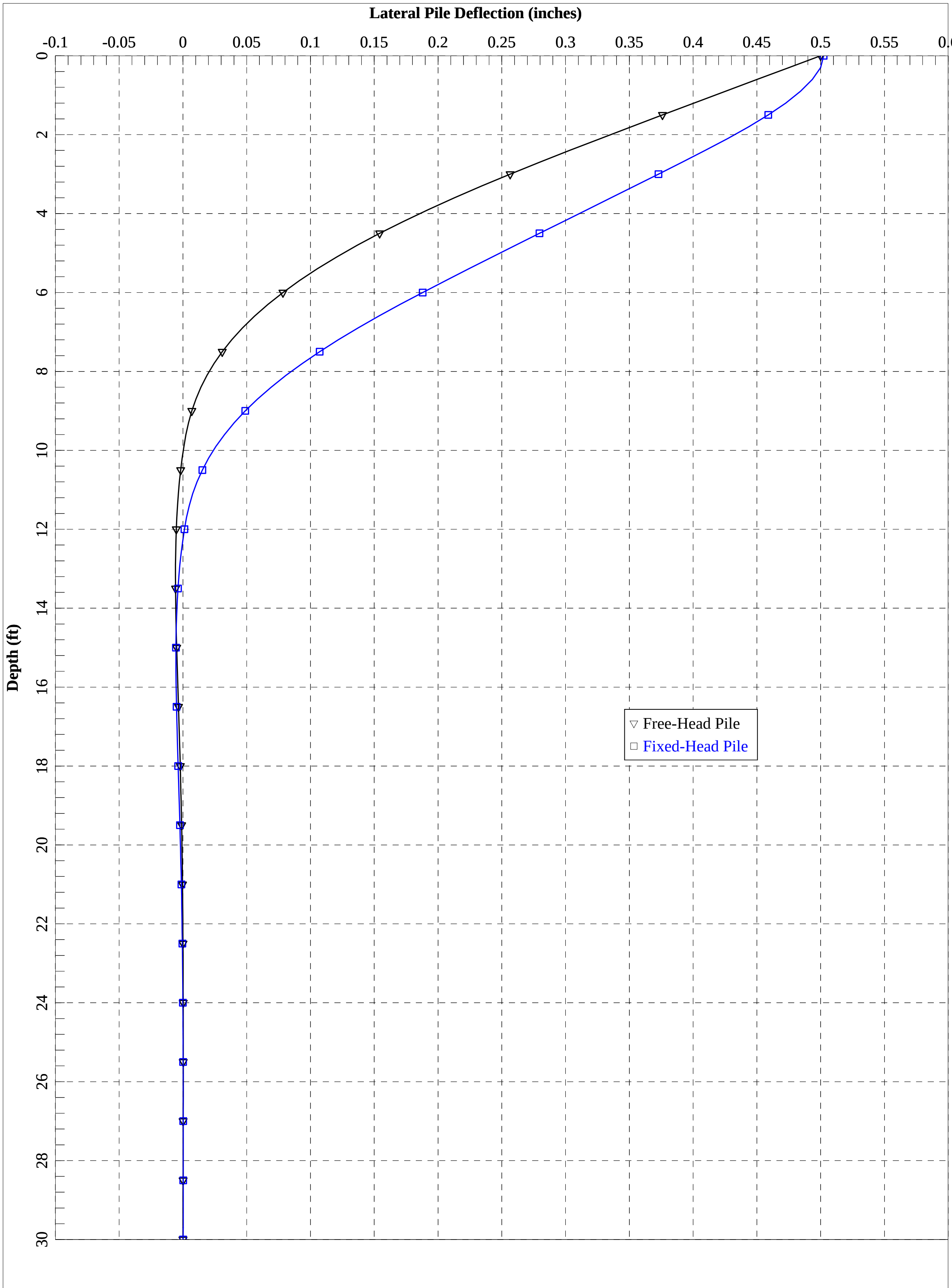
Reading No.	Water Level Measurement			Water Level Calculations				Percolation Rate Calculations		
	Time Interval $\Delta T = T_2 - T_1$ (min)	Initial Depth to Water d_1 (ft)	Final Depth to Water d_2 (ft)	Initial Height of Water Colum $d_{H1} = d_b - d_1$ (in)	Final Height of Water Column $d_{H2} = d_b - d_2$ (in)	Drop in Height $\Delta d_H = d_{H1} - d_{H2}$ (in)	Average Height of Water Column $d_{avg} = (d_{H1}+d_{H2})/2$ (in)	Pre-adjusted Percolation Rate $K_i = \Delta d_H / \Delta T$ (in/hr)	Reduction Factor $R_f = ((2d_{H1} - \Delta d_H) / D) + 1$	Adjusted Percolation Rate $K = K_i / R_f$ (in/hr)
1	30	0.35	3.65	115.80	76.20	39.60	96.00	79.20	25.00	3.17
2	30	0.35	3.45	115.80	78.60	37.20	97.20	74.40	25.30	2.94
3	30	0.35	3.50	115.80	78.00	37.80	96.90	75.60	25.23	3.00
4	30	0.35	3.50	115.80	78.00	37.80	96.90	75.60	25.23	3.00
5	30	0.35	3.50	115.80	78.00	37.80	96.90	75.60	25.23	3.00

Adjusted Percolation Rate = **3.00** **in/hr**

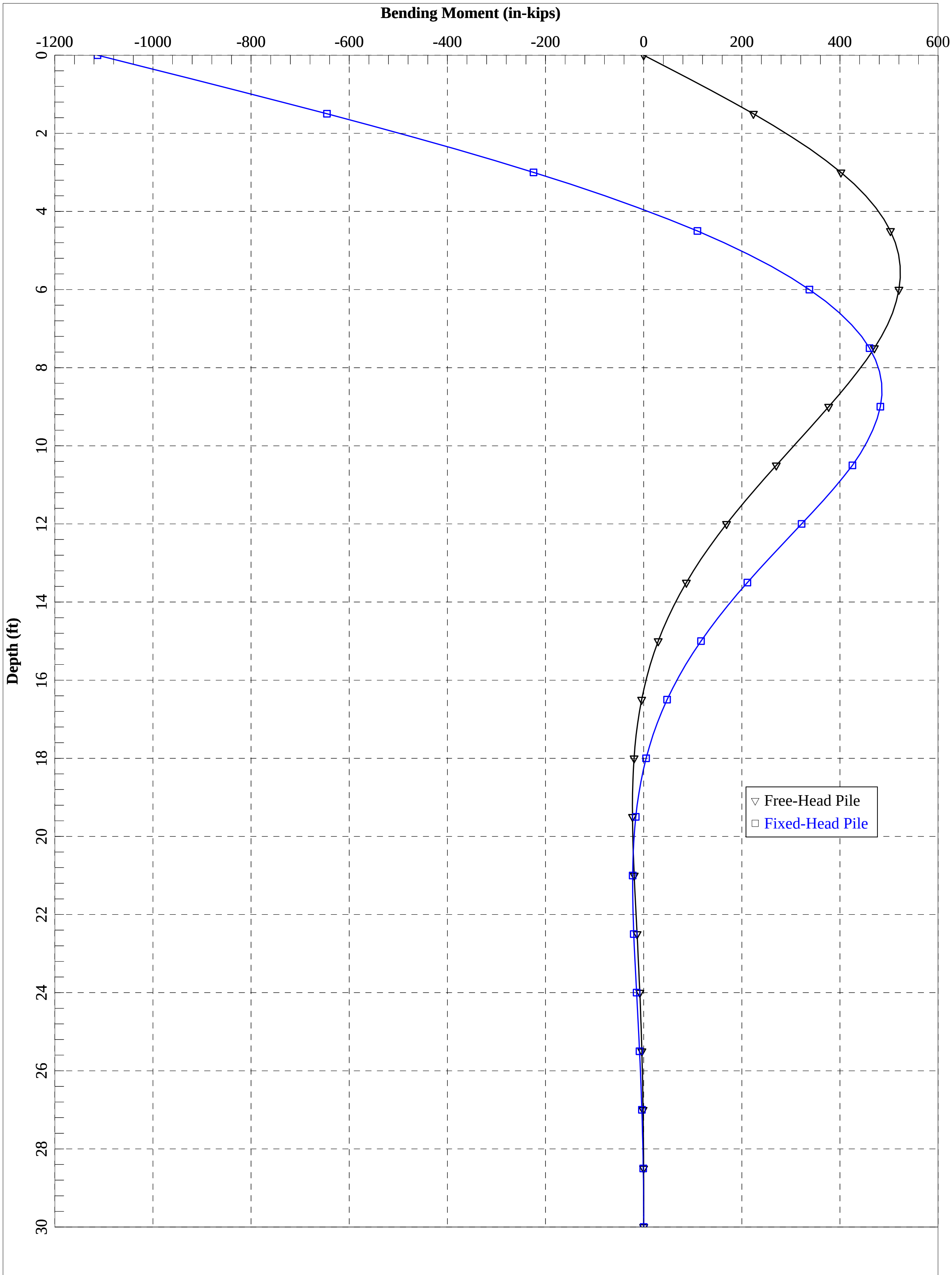
APPENDIX E. CIDH PILE CAPACITY GRAPHS



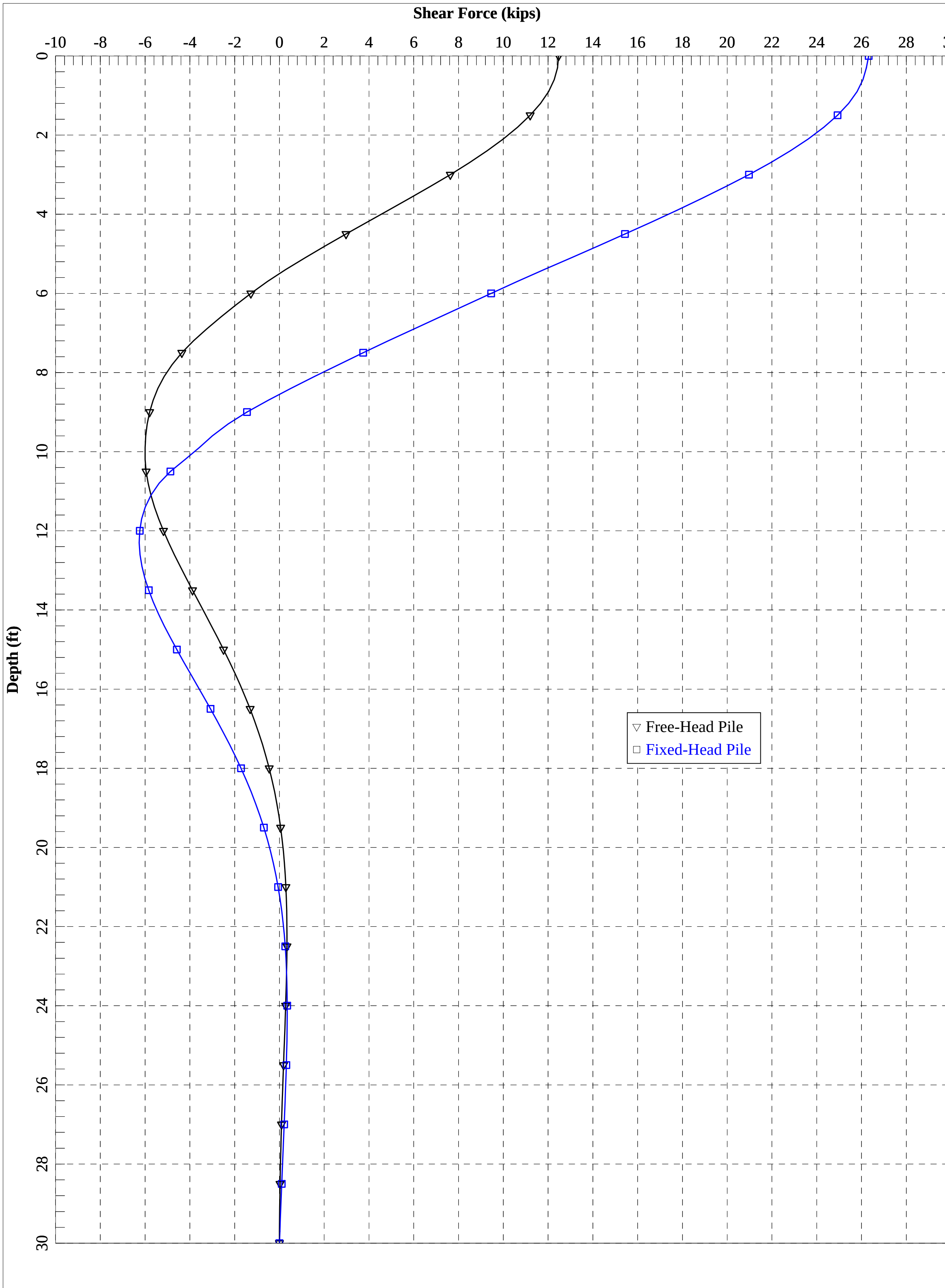
CIDH Pile Downward Axial Capacity



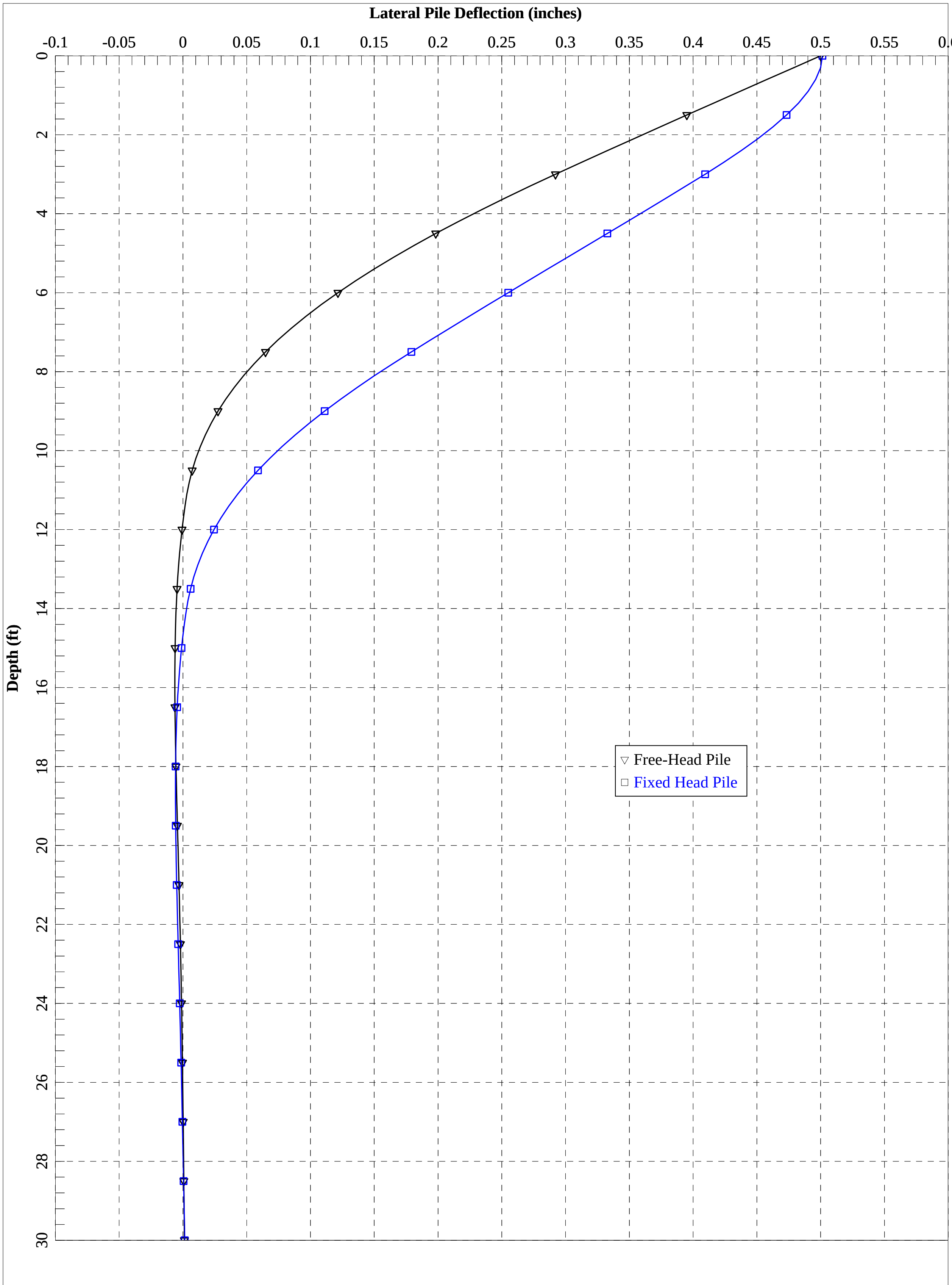
18-in Diameter CIDH Pile Lateral Capacity - Deflection



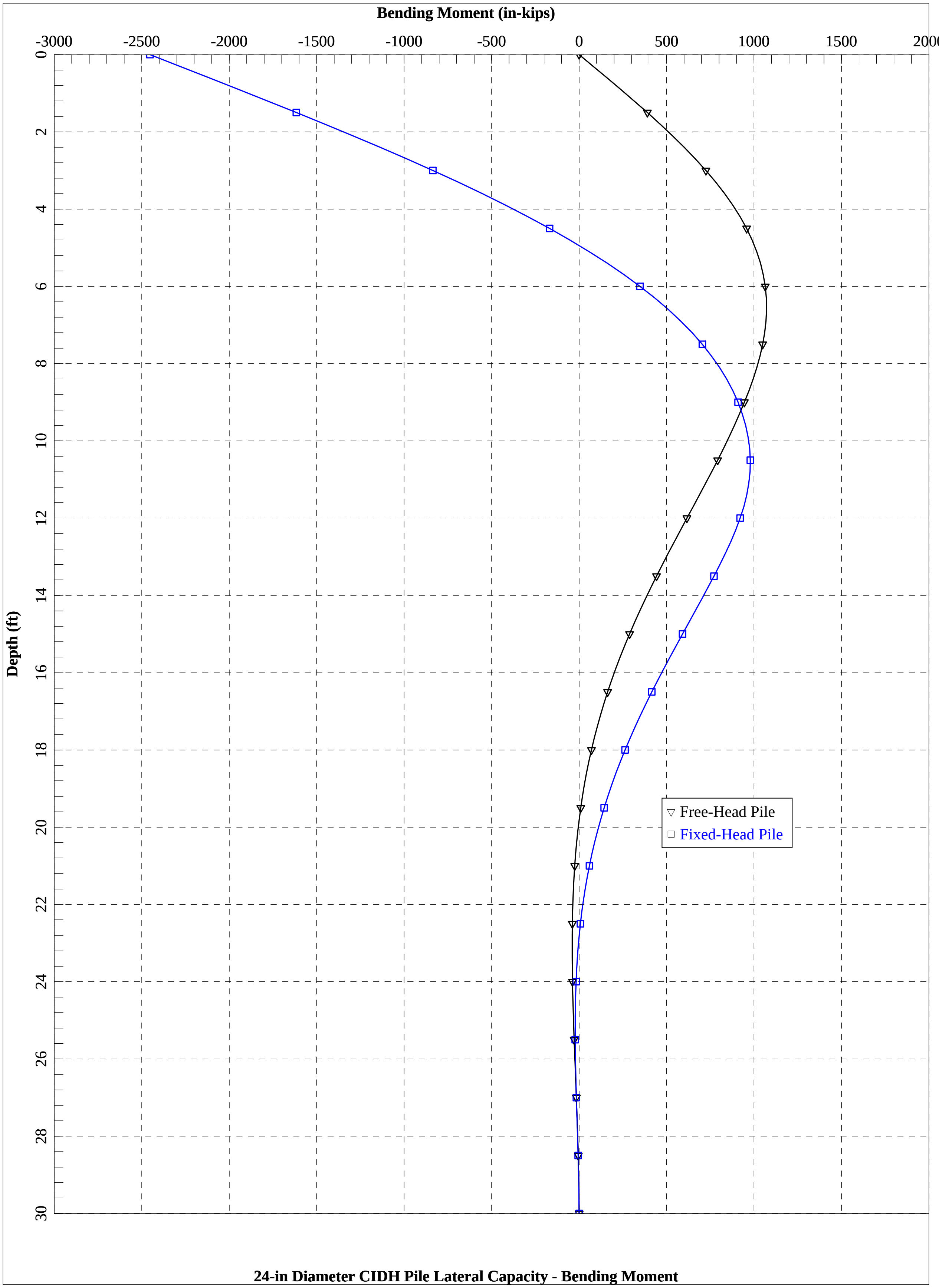
18-in Diameter CIDH Pile Lateral Capacity - Bending Moment



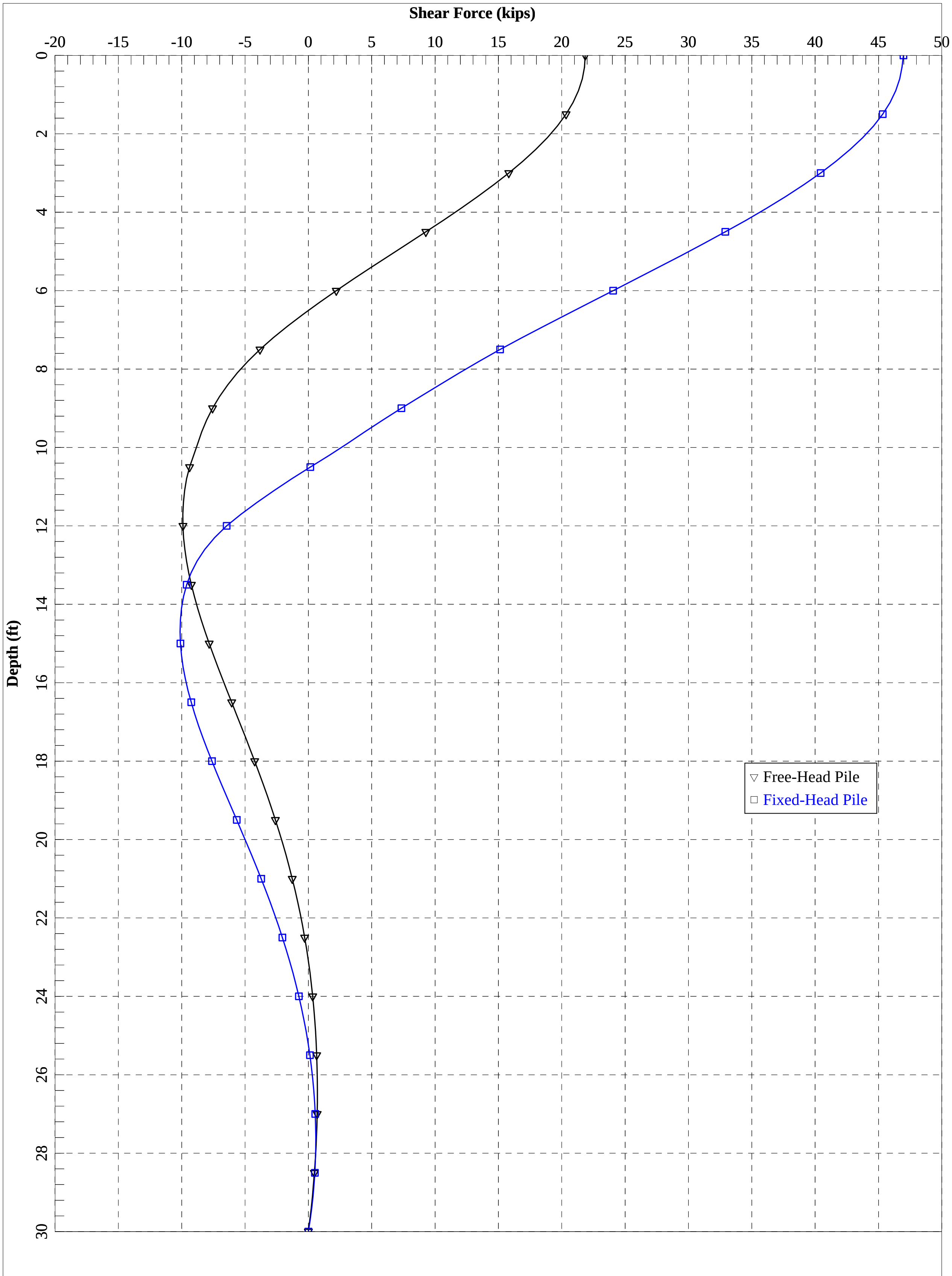
18-in Diameter CIDH Pile Lateral Capacity - Shear Force



24-in Diameter CIDH Pile Lateral Capacity - Deflection



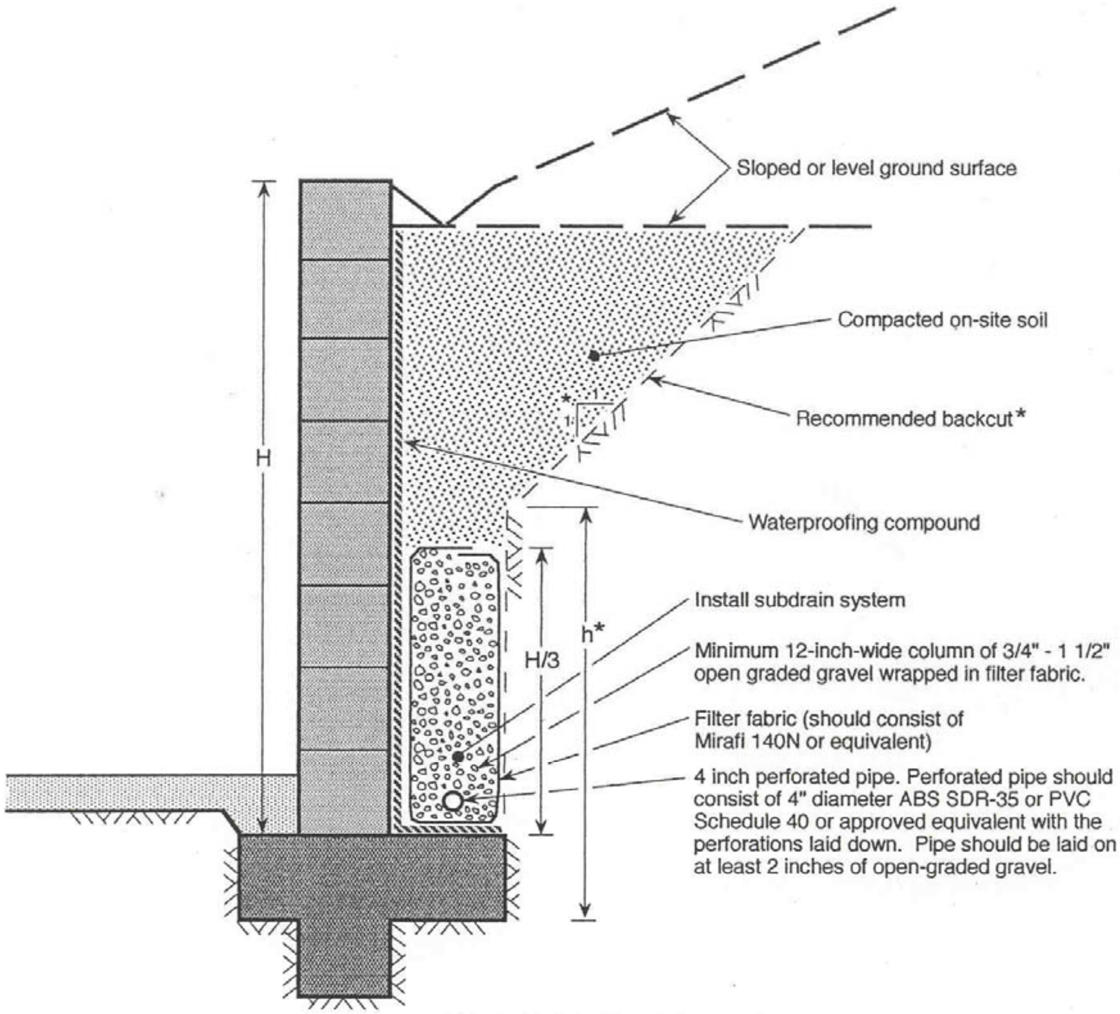
24-in Diameter CIDH Pile Lateral Capacity - Bending Moment



24-in Diameter CIDH Pile Lateral Capacity - Shear Force

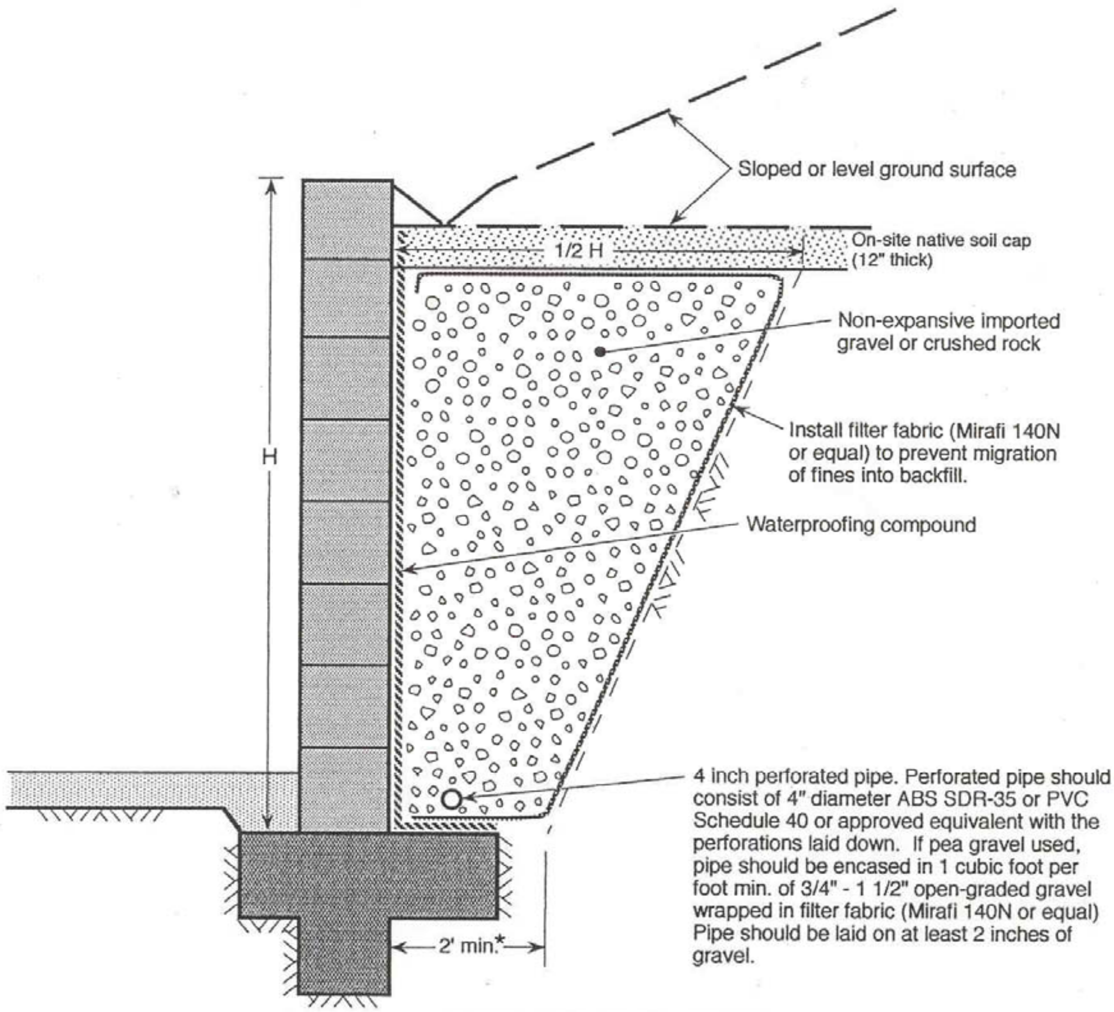
APPENDIX F. TYPICAL RETAINING WALL BACKFILL DETAILS

NATIVE SOIL BACKFILL



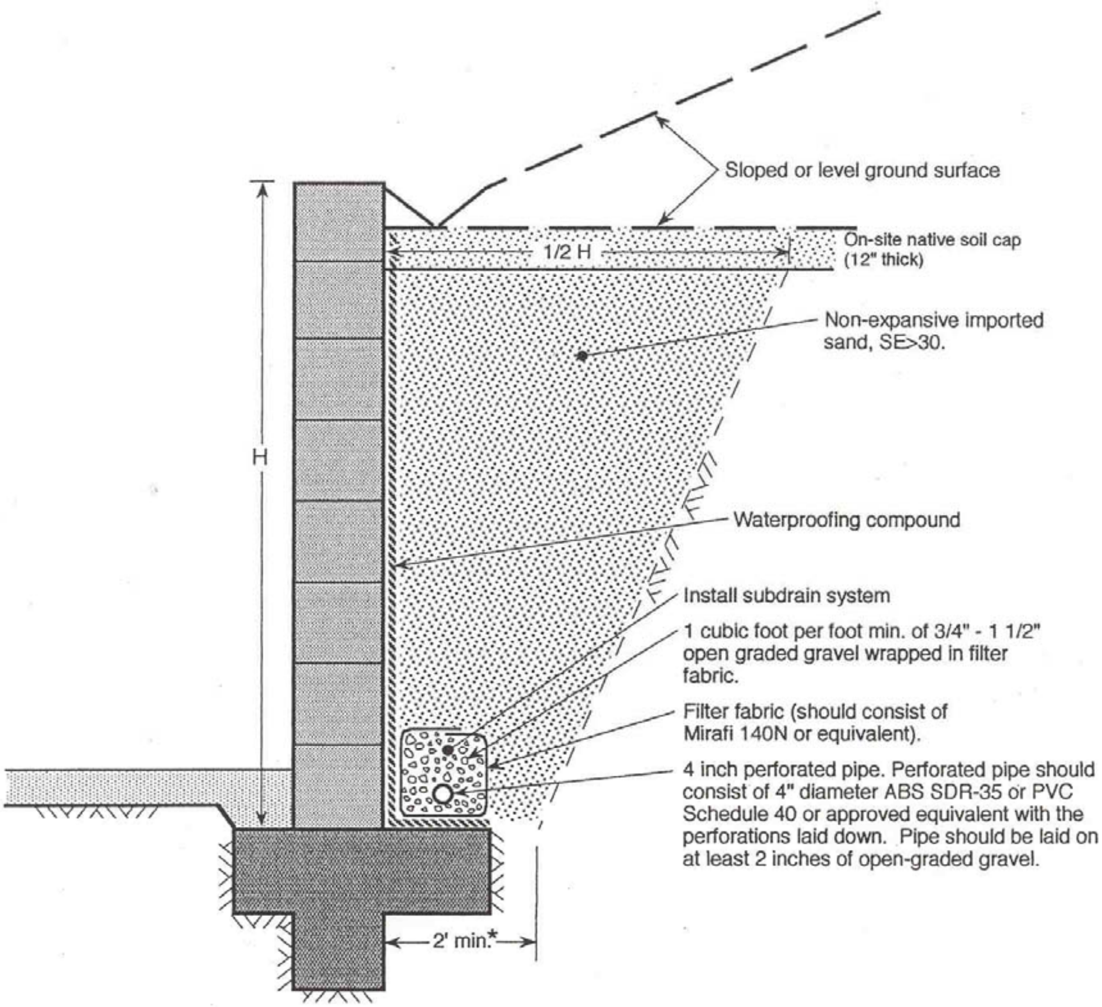
* Vertical height (h) and slope angle of backcut per soils report. Based on geologic conditions, configuration of backcut may require revisions (i.e. reduced vertical height, revised slope angle, etc.)

IMPORTED GRAVEL OR CRUSHED ROCK BACKFILL



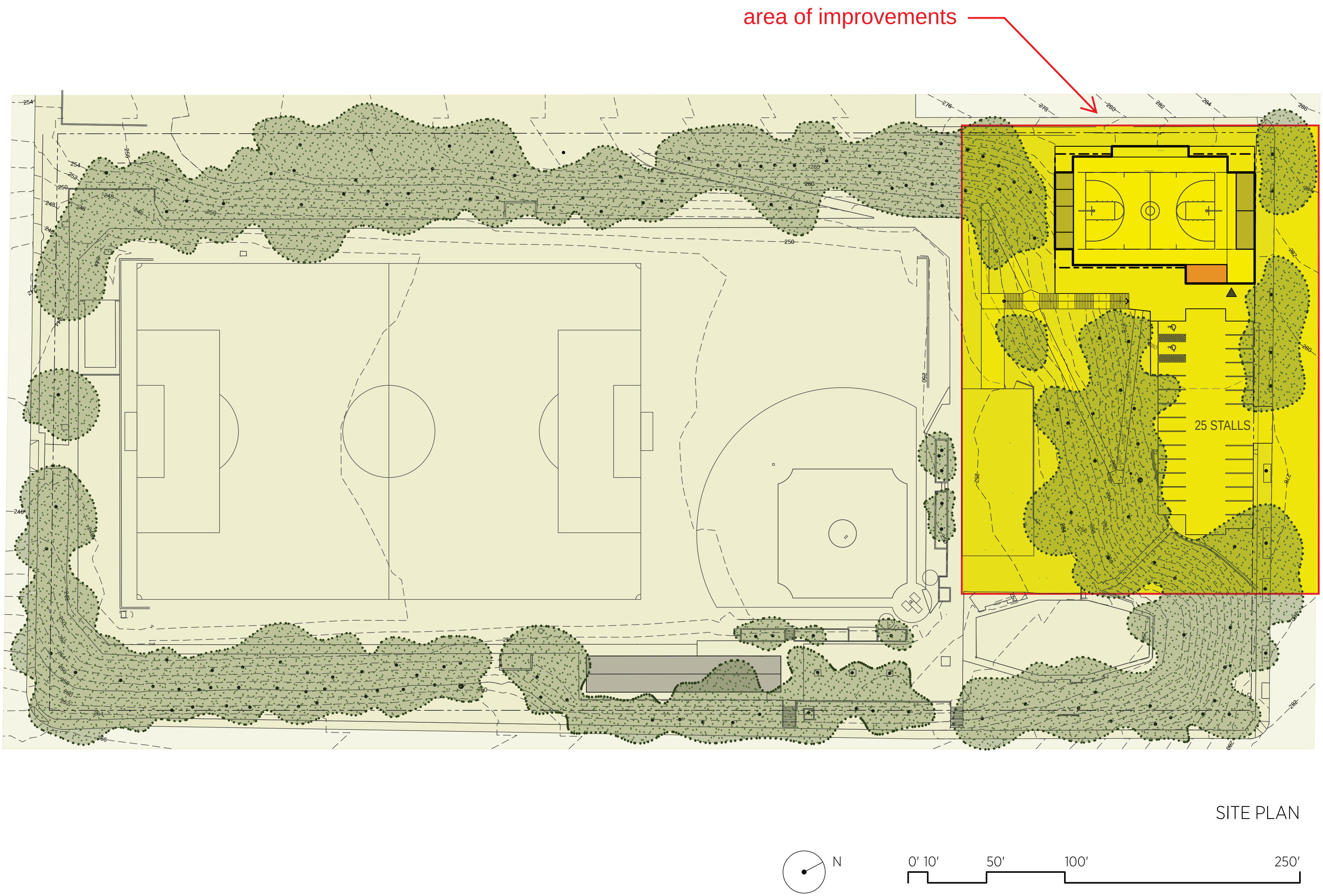
* At base of wall, the non-expansive backfill materials should extend to a min. distance of 2' or to a horizontal distance equal to the heel width of the footing, whichever is greater.

IMPORTED SAND BACKFILL



* At base of wall, the non-expansive backfill materials should extend to a min. distance of 2' or to a horizontal distance equal to the heel width of the footing, whichever is greater.

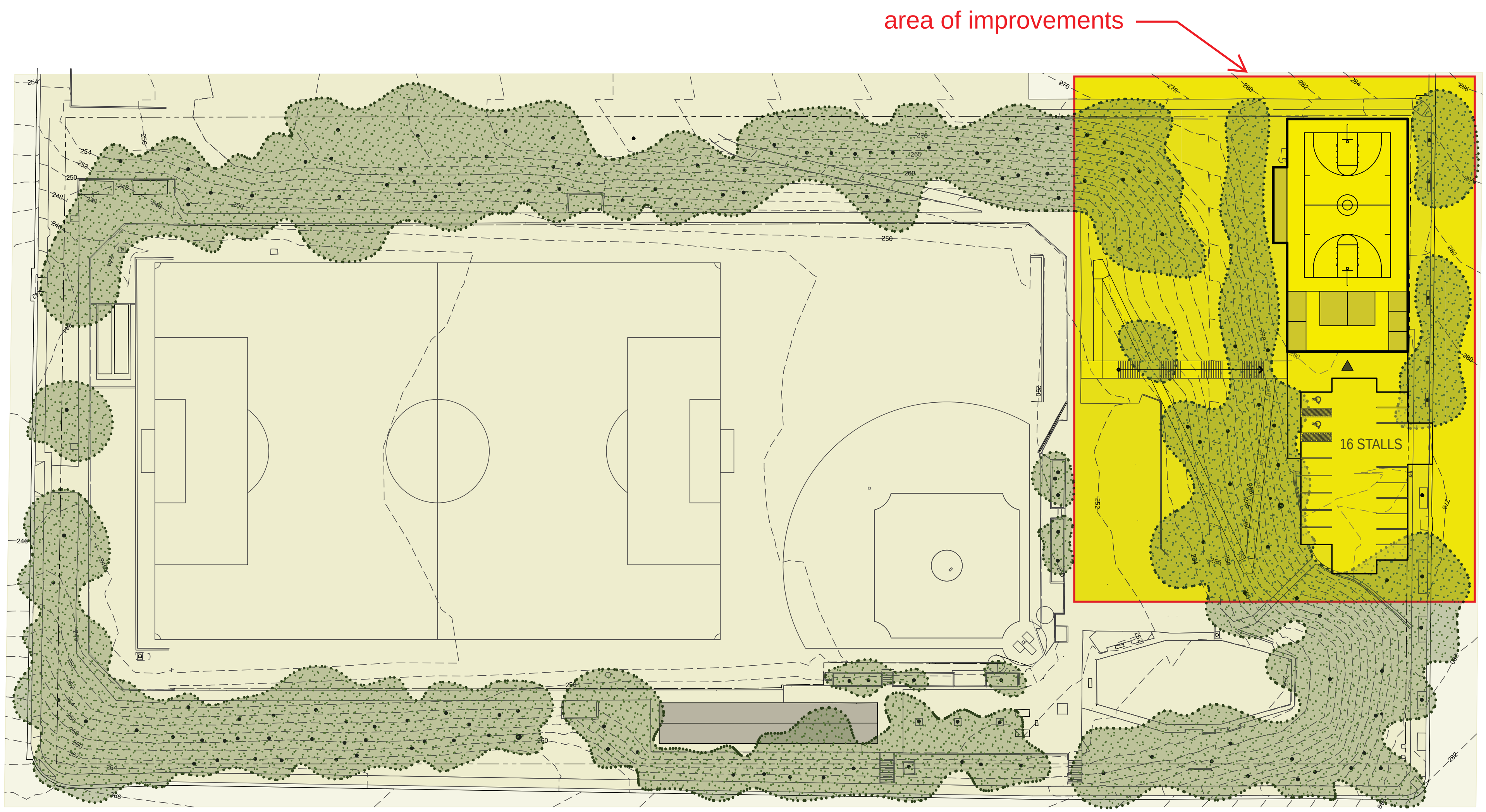
APPENDIX G. PRILIMINARY SITE PLANS



SITE PLAN

SITE PLAN - CORNER

A COMMUNITY BEACON



SITE PLAN - ALTERNATE LAYOUT

SITE PLAN - UPPER LEVEL OPTION

METHANE SOIL GAS INVESTIGATION REPORT
PROPOSED BOYLE HEIGHTS SPORTS CENTER PROJECT
2510 WHITTIER BOULEVARD
LOS ANGELES, CALIFORNIA

PREPARED FOR

CITY OF LOS ANGELES
GEOTECHNICAL ENGINEERING GROUP
1149 SOUTH BROADWAY, SUITE 120
LOS ANGELES, CALIFORNIA 90015-2213

PREPARED BY

WILLDAN GEOTECHNICAL
1515 SOUTH SUNKIST STREET, SUITE E
ANAHEIM, CALIFORNIA 92806
WILLDAN GEOTECHNICAL PROJECT NO. 106965-2010

OCTOBER 31, 2017

WILLDAN Geotechnical | extending your reach

October 31, 2017

Mr. Patrick J. Schmidt, PE, GE
City of Los Angeles
Geotechnical Engineering Group
1149 S. Broadway, Suite 120
Los Angeles, CA 90015-2213

Subject: Methane Soil Gas Investigation Report
Proposed Boyle Heights Sports Center Project, Los Angeles, California
Willdan Geotechnical Project No. 106965-2010

Dear Mr. Schmidt,

Willdan Geotechnical is pleased to submit this report for the proposed Boyle Heights Sports Center project located at 2510 Whittier Boulevard in the City of Los Angeles, California. This report presents the findings and conclusions with respect to methane soil gas investigation performed by our sub-consultant within the subject project site.

We appreciate the opportunity to assist you and look forward to future projects. If you have any questions, please contact us.

Respectfully submitted,
WILLDAN GEOTECHNICAL

REGISTERED PROFESSIONAL ENGINEER
MOHSEN RAHIMIAN
No. 3059
GEOTECHNICAL
STATE OF CALIFORNIA

Mohsen Rahimian

Mohsen Rahimian, PE, GE
Principal Engineer

Attachment: Methane Soil Gas Investigation Report, prepared by Sub-Consultant

Distribution: Addressee (4 unbound wet signed sets and one PDF copy via e-mail)

Engineering | Geotechnical | Environmental | Sustainability | Financial | Homeland Security
Phone: 714.634.3318 | Fax: 714.634.3372 | 1515 South Sunkist Street, Suite E, Anaheim, CA 92806 | www.willdan.com

REPORT OF METHANE SOIL GAS INVESTIGATION
PROPOSED GYMNASIUM BUILDING

2510 WHITTIER BOULEVARD
LOS ANGELES, CA

Prepared for:

WILLDAN GEOTECHNICAL

Anaheim, California

TERRA-PETRA ENVIRONMENTAL ENGINEERING
700 S Flower Street, Suite 2580
Los Angeles, California

October 30, 2017

October 30, 2017

Mohsen Rahimian, PE, GE
Supervising Engineer
Willdan Geotechnical
T. (657) 221-2714
C. (818) 577-3545
E. MRahimian@willdan.com

Subject: **Report of Methane Soil Gas Testing**
Proposed Gymnasium Building
2510 Whittier Blvd. Los Angeles, CA 90023
Tract: TR 5299, Block: None, Lot(s): 19-23

Terra-Petra is pleased to submit this report to summarize the methane soil gas investigation services conducted at the subject site referenced above. The purpose of this investigation was to determine the methane soil gas mitigation requirements, if any, in connection with the proposed gymnasium building. The project site has been determined to be located within a City of Los Angeles Designated Methane Buffer Zone (See **Exhibit 1, Site Location Map**).

Our professional services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable environmental consultants practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this report. This report has been prepared for Willdan Geotechnical and any pertinent consultants to be used solely for the design of the proposed project. This report has not been prepared for use by other parties, and may not contain sufficient information for purposes of other parties or other uses.

PROJECT INFORMATION

Terra-Petra was contacted to perform a soil gas investigation for the proposed development under our LADBS Testing License #10224. We understand that the new 10,000 sq ft gymnasium will consist of a high school standard full-sized basketball court, offices, storage rooms, rest rooms and a parking lot. The project will also include grading work at the sloped area for access to the lower portion of the basketball court and synthetic field. The purpose of this investigation was to detect the presence of any elevated levels of methane gas in the in-situ soils at the project site.

SOIL GAS PROBE INSTALLATION & TESTING

The methane soil gas testing at the site was performed based on the procedures conforming to the Los Angeles Department of Building and Safety (LADBS) Information Bulletin Ref. No. 91.71404.1, P/BC 2002-101. City guidelines require that one shallow-depth probe be installed for every 10,000 square feet of site area where the highest concentration of soil gas is most likely to be found, with a minimum of two shallow gas probes regardless of the total area of the site. A total of three (3) shallow probe locations were selected based on the site testing area of approximately 20,560 sq. ft. (See **Exhibit 2, Probe Locations Map**). Predicated on the soil gas testing results at the shallow probes, an additional two (2) deep gas probe locations were selected.

On 10/26/17, shallow and deep borings were drilled using a truck-mounted GeoProbe 7800 direct-push drill rig. Shallow borings were drilled to a depth of 4 feet, with gas probes installed at 4 ft bsg. Terra-Petra was obligated to install the deepest probe a minimum of 20 feet beneath the lowest level of the building. As such, deep boring 2 (DP-2) was drilled to a depth of 20 feet, with nested gas probes installed at depths of 5 ft, 10 ft and 20 ft. DP-1 was drilled to a depth of 19 ft, at which point the drill encountered refusal. Nested gas probes were installed at depths of 5 ft, 10 ft, and 19 ft within DP-1. Gas probes were constructed as shown in **Exhibit 3, Probe Construction Diagrams**. Groundwater was not encountered during the investigation and the historic groundwater level at the site is unknown.

The current investigation was performed in accordance with LADBS standards. Soil gas samples were collected during two rounds of monitoring on 10/26/17 and 10/27/17 from each of the probes. Each sampling period was separated by a time period of approximately 24 hours. As required by the LADBS, all probes were monitored for detectable combustible gas and soil gas pressures using a calibrated CES/Landtec GEM 5000 portable 4-gas detector with a lower limit for reporting methane levels of 1,000 ppmv (parts per million by volume).

TEST RESULTS

Methane soil gas was measured in non-detectable levels for the CES/Landtec GEM 5000 portable 4-gas detector in each of the shallow and deep gas probes during both days of monitoring. The results of the soil gas testing measurements were recorded in an approved format as presented in the attached **Exhibit 5, Form 01 – Certificate of Compliance for Methane Test Data**.

CONCLUSIONS

Methane gas is combustible with a lower explosive limit (LEL) of approximately 5%,v/v in air. In structures, regulatory agencies commonly consider methane concentrations above 25% of the LEL (1.25%,v/v) to be action levels above which gas concentrations must be mitigated. The City of Los Angeles Department of Building and Safety considers methane soil gas concentrations at 0.0%,v/v to be the action level at which soil gas concentrations must be mitigated for buildings to be constructed in a methane zone. For the methane buffer zone, this same action level applies only if the water column pressure is greater than 2 inches.

The calibration of the instrument used to detect combustible methane gas concentrations on site renders any readings of methane gas levels between 0 – 999 ppmv (0 – 0.009%, v/v) as non-detectable. Since all soil gas probes produced non-detectable readings of methane gas, it is possible that methane concentrations in the in-situ soils fall within the range of 101-1,000 ppmv for a Level II classification. Thus, based on the non-detect methane readings and negligible water column pressures encountered, along with LADBS action levels presented above, we recommend that the methane mitigation for the site adheres to design requirements for **Methane Buffer Zone – Level II, ≤ 2-in. water column pressure.** Therefore, as per said design level, no methane mitigation measures are required for this project.

I am a registered California civil engineer with experience in methane gas mitigation systems. Should you have any questions regarding this report, please contact Justin Conaway at 213-458-0494. We appreciate the opportunity to assist you with your project.

Sincerely,

John Conaway, PE
Terra-Petra
LADBS License #10224

<u>Attachments</u>
Exhibit 1: Site Location Map
Exhibit 2: Probe Locations Map
Exhibit 3: Probe Construction Diagrams
Exhibit 4: Field Data Sheets
Exhibit 5: Form 01 – Certificate of Compliance for Methane Test Data

Exhibit 1:
Site Location Map

NavigateLA Map

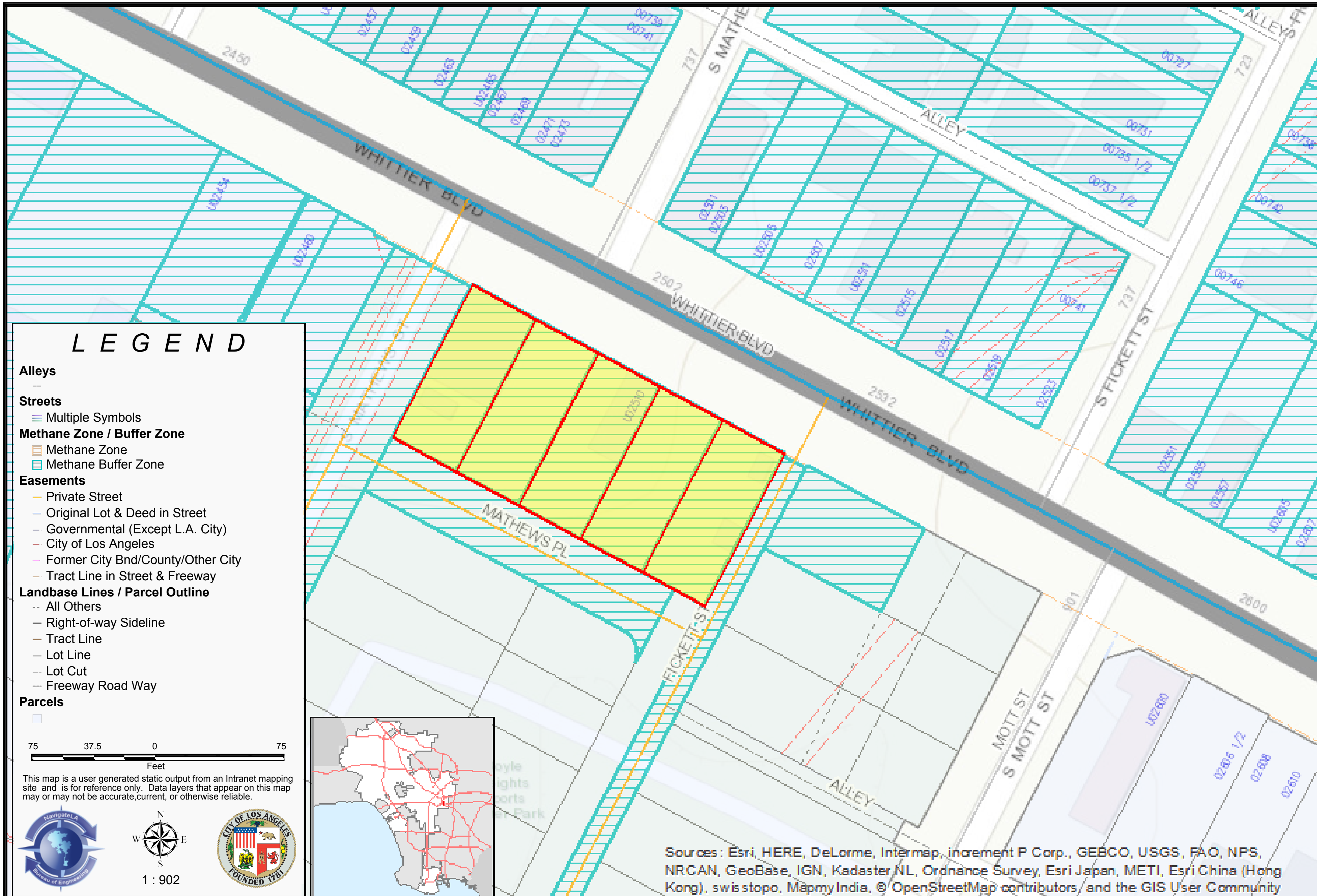


Exhibit 2:
Probe Locations Map

IMAGE SOURCE: GOOGLE EARTH

LEGEND

SP-2	SHALLOW PROBE
DP-2	DEEP PROBE
	APPROXIMATE PROPERTY LINE

OVERALL SITE

SCALE: 1" = 50'

0' 25' 50'

SCALE: 1" = 50'

EXHIBIT: 2

TITLE: SOIL GAS MONITORING PROBE LOCATIONS

DATE: OCTOBER 27, 2017

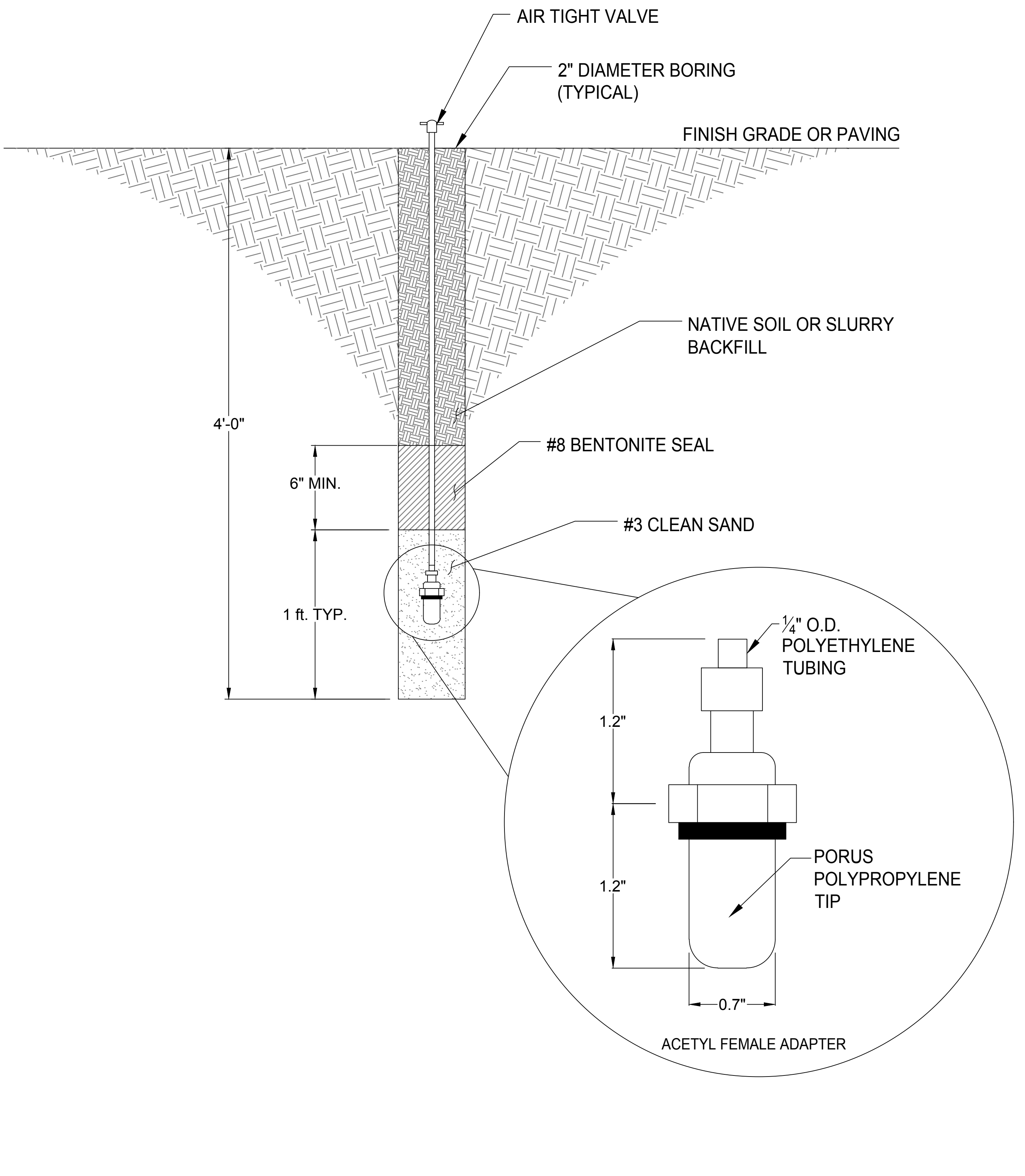
DRAWN BY: J.MORA

CHECKED BY:

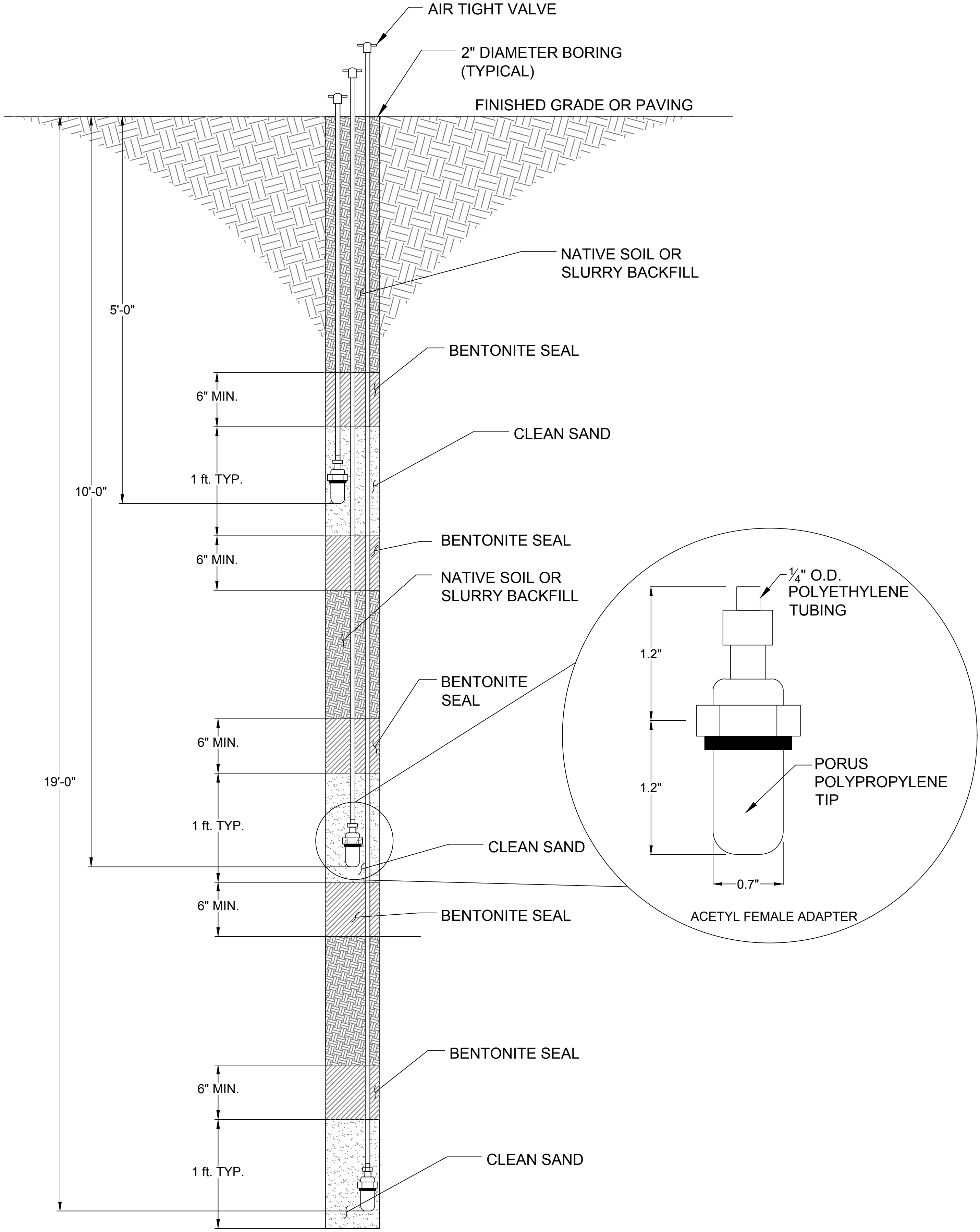
ADDRESS: 2510 WHITTIER BLVD.
LOS ANGELES, CA 90023

Terra-Petra
ENVIRONMENTAL ENGINEERING
700 S. Flower St., Ste. 2580, Los Angeles, CA 90017 213.458.0494
Los Angeles (HQ) . San Francisco . New York . Denver . Tacoma

Exhibit 3:
Probe Construction Diagrams



Terra-Petra ENVIRONMENTAL ENGINEERING 700 S. Flower St., Ste. 2580, Los Angeles, CA 90017 213.458.0494 Los Angeles (HQ) . San Francisco . New York . Denver . Tacoma	ADDRESS: 2510 WHITTIER BLVD. LOS ANGELES, CA 90023	TITLE: CONSTRUCTION DIAGRAM FOR SHALLOW PROBE AT 4 FT.	EXHIBIT: 3
		DRAWN BY: J.MORA	DATE: OCTOBER 27, 2017
		CHECKED BY:	



Terra-Petra
ENVIRONMENTAL ENGINEERING
700 S. Flower St., Ste. 2580, Los Angeles, CA 90017 213.458.0494
Los Angeles (HQ) . San Francisco . New York . Denver . Tacoma

ADDRESS: 2510 WHITTIER BLVD. LOS ANGELES, CA 90023

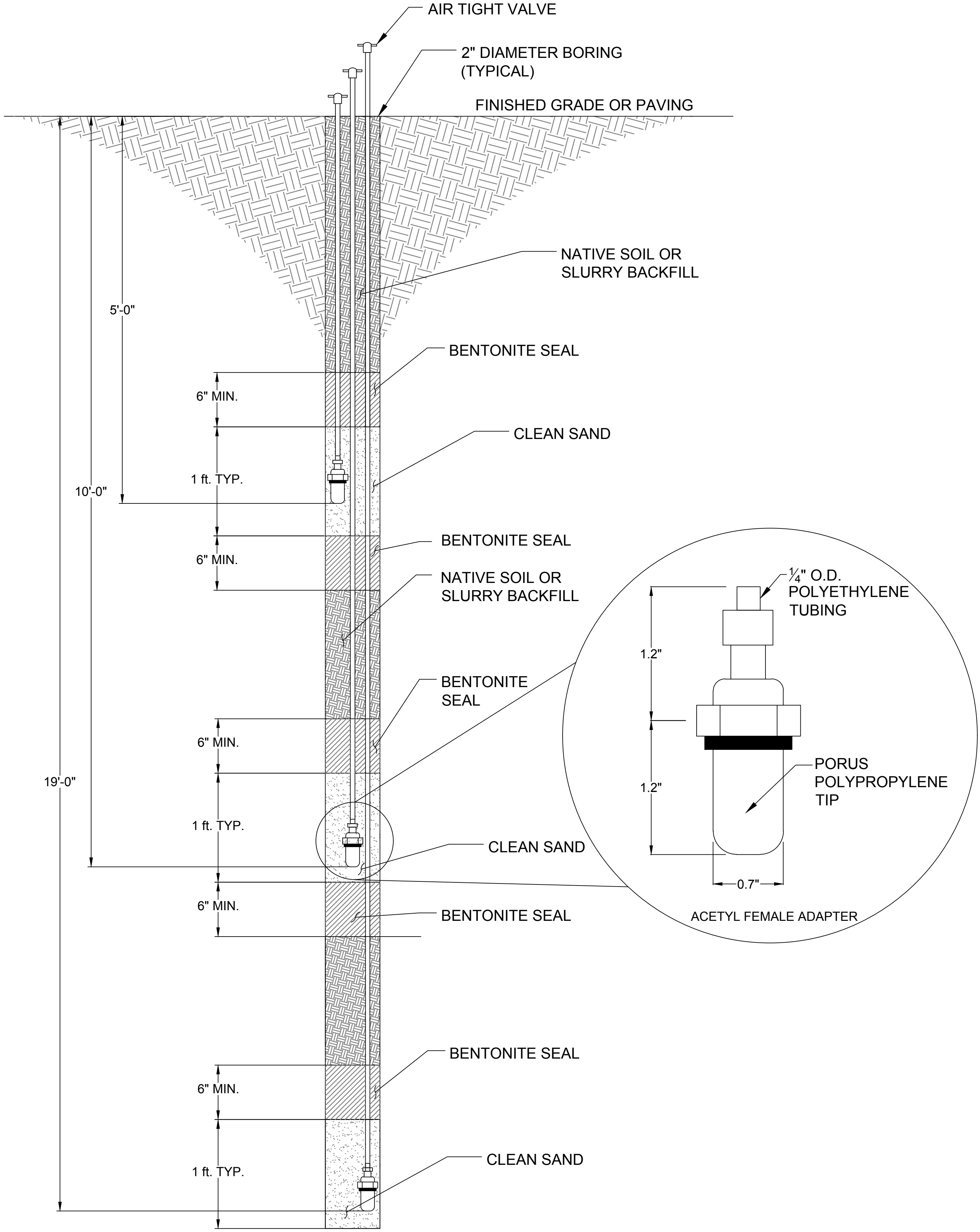
TITLE: CONSTRUCTION DIAGRAM FOR DEEP PROBE AT 19 FT.

DRAWN BY: J.MORA

DATE: OCTOBER 27, 2017

CHECKED BY:

EXHIBIT: 3



Terra-Petra
ENVIRONMENTAL ENGINEERING
700 S. Flower St., Ste. 2580, Los Angeles, CA 90017 213.458.0494
Los Angeles (HQ) . San Francisco . New York . Denver . Tacoma

ADDRESS: 2510 WHITTIER BLVD. LOS ANGELES, CA 90023	TITLE: CONSTRUCTION DIAGRAM FOR DEEP PROBE AT 20 FT.		EXHIBIT: 3
	DRAWN BY: J.MORA	DATE: OCTOBER 27, 2017	
	CHECKED BY:		

Exhibit 4:
Field Data Sheets

Soil Gas Investigation Spreadsheet

Site Location:		2510 Whittier Blvd., Los Angeles, CA(90023).						
Date:		10/26/17						
Time:		0700hr.						
Weather conditions:		Clear, warm, still, dry.						
Instrument:		Landtec GEM 5000 portable 4-gas detector (I/R for methane).						
Barometric Pressure:		29.52-in. Hg						
Drilling Method:		Truck-mounted GeoProbe 7800 direct-push drill rig.						
Probe No.	Depth	Probe Press. (in-H20)	Methane* (%v/v)		CO_2 (%v/v)	O_2 (%v/v)	N_2 (%v/v)	Comments:
SP-1	4.0	0.00	ND		0.6	18.4	Bal.	
SP-2	4.0	0.00	ND		1.2	17.8	Bal.	
SP-3	4.0	0.00	ND		0.3	19.4	Bal.	
DP-1	5.0	0.00	ND		0.5	18.7	Bal.	
	10.0	0.01	ND		1.1	18.0	Bal.	
	19.0	0.02	ND		1.9	16.2	Bal.	Refusal at 19 ft. bsg.
DP-2	5.0	0.01	ND		0.3	18.6	Bal.	
	10.0	0.04	ND		1.7	16.7	Bal.	
	20.0	0.02	ND		2.7	16.2	Bal.	
(Note: ND = Not Detected. All gas quality measurements taken with in-line carbon filter.)								

DL SCIENCE, INC.
532 W. Maple ave.
El Segundo. CA 90245
tel. (310) 416-1472
dllucero@sbcglobal.net

Soil Gas Investigation Spreadsheet

Site Location:		**2510 Whittier Blvd., Los Angeles, CA(90023).**						
Date:		**10/27/17**						
Time:		**0700hr.**						
Weather conditions:		**Clear, warm, still, dry.**						
Instrument:		**Landtec GEM 5000 portable 4-gas detector (I/R for methane).**						
Barometric Pressure:		**29.51-in. Hg**						
Drilling Method:		**Truck-mounted GeoProbe 7800 direct-push drill rig.**						
		Probe Press.	**Methane***		**CO_2**	**O_2**	**N_2**	
Probe No.	**Depth**	**(in-H20)**	**(%v/v)**		**(%v/v)**	**(%v/v)**	**(%v/v)**	**Comments:**
DP-1	**5.0**	**0.02**	**ND**		**1.1**	**18.8**	**Bal.**	
	10.0	**0.04**	**ND**		**3.7**	**17.5**	**Bal.**	
	19.0	**0.01**	**ND**		**3.9**	**16.2**	**Bal.**	**Refusal at 19 ft. bsg.**
DP-2	**5.0**	**0.04**	**ND**		**2.7**	**16.3**	**Bal.**	
	10.0	**0.01**	**ND**		**4.2**	**14.4**	**Bal.**	
	20.0	**0.02**	**ND**		**5.8**	**13.7**	**Bal.**	
(Note: ND = Not Detected. All gas quality measurements taken with in-line carbon filter.)								

DL SCIENCE, INC.
532 W. Maple ave.
El Segundo. CA 90245
tel. (310) 416-1472
dllucero@sbcglobal.net

Exhibit 5:
Form 01 Certificate of Compliance for Methane Test Data

FORM 1 - CERTIFICATE OF COMPLIANCE FOR METHANE TEST DATA

Part 1: Certification Sheet

Site Address: 2510 Whittier Blvd.

Legal Description: Tract: TR 5299 Lot: 19 - 23 Block: None

Building Use: Gymnasium

Name of Architect, Engineer, or Geologist: John Conaway

Mailing Address: 700 S. Flower St #2580, Los Angeles CA 90017

Telephone: 213 458 0494

Name of Testing Laboratory: Terra - Petra

City Test Lab License #: 10224

Telephone: 213 458 0494

Architect's, Engineer's or Geologist's Stamp:

REGISTERED PROFFESIONAL ENGINEER
JOHNNY RAY CONAWAY
NO. C 19,689
exp. 09-30-19
CIVIL
STATE OF CALIFORNIA

I hereby certify that I have tested the above site for the purpose of methane mitigation and that all procedures were conducted by a City of Los Angeles licensed testing agency in conformity with the requirements of the LADBS Information Bulletin P/BC 2014-101. Where the inspection and testing of all or part of the work above is delegated, full responsibility shall be assumed by the architect, engineer or geologist whose signature is affixed thereon.

Signed: Johnny R Conaway date 10/30/2017

Required Data:

- Project is in the ~~(Methane Zone)~~ or (Methane Buffer Zone).
- Depth of ground water observed during testing: N/A feet below the Impervious Membrane.
- Depth of Historical High Ground Water Table Elevation*: UNK feet below the Impervious Membrane.
- Design Methane Concentration**: 101 - 1,000 parts per million in volume (ppmv).
- Design Methane Pressure***: ≤ 2 inches of water column.
- Site Design Level: (Level I, Level II, Level III, Level IV, Level V) with ≤ 2 inches of water column.

De-watering:

- De-watering ~~(is)~~ (is not) required per Section 7104.3.7.
- Pump discharge rate N/A cubic feet per minute per reference geology or soil report: ____________ dated ____________.

Additional Investigation:

- Additional investigation ~~(was)~~ (was not) conducted.

Latest Grading on Site:

- Date of last grading on site (was) ~~(was not)~~ more than 30 days before Site Testing.
- See Attached explanation of the effect on soil gas survey results by grading operations.

Notes:

* Historical High Ground Water Table Elevation shall mean the highest recorded elevation of ground water table based on historical records and field investigations as determined by the engineer for the methane mitigation system.

** Design Methane Concentration shall mean the highest recorded measured methane concentration from either Shallow Soil Gas Test or any Gas Probe Set on the site.

*** Design Methane Pressure shall mean the highest total pressure measured from any Gas Probe Set on the site.

As a covered entity under Title II of the Americans with Disabilities Act, the City of Los Angeles does not discriminate on the basis of disability and, upon request, will provide reasonable accommodation to ensure equal access to its programs, services and activities. For efficient handling of information internally and in the internet, conversion to this new format of code related and administrative information bulletins including MGD and RGA that were previously issued will allow flexibility and timely distribution of information to the public.

FORM 1 (CONTINUED) - CERTIFICATE OF COMPLIANCE FOR METHANE TEST DATA

Part 2: Test Data - Shallow Soil Gas Test and Gas Probe Test

Site Address: 2510 Whittier Blvd, Los Angeles CA
Description of Gas Analysis Instrument(s): Infra Red
Instrument Name and Model: LAND TEC Gem 5000 Instrument Accuracy: ± 1,000 ppmv.
City of Los Angeles Testing License #: 10224

Date	Time	Probe Set #	Concentration (ppmv)	Pressure (inches water column)	Probe Depth (feet)	Description / Probe Location
						SEE SITE PLAN FOR PROBE LOCATIONS
10/26/2017	7:00	SP-1	ND*	0.00	4.0	
" "	" "	SP-2	ND*	0.00	4.0	
" "	" "	SP-3	ND*	0.00	4.0	
" "	" "	DP-1	ND*	0.00	5.0	
" "	" "	" "	ND*	0.01	10.0	
" "	" "	" "	ND*	0.02	19.0	
" "	" "	DP-2	ND*	0.01	5.0	
" "	" "	" "	ND*	0.04	10.0	
" "	" "	" "	ND*	0.02	20.0	
10/27/2017	7:00	DP-1	ND*	0.02	5.0	
" "	" "	" "	ND*	0.04	10.0	
" "	" "	" "	ND*	0.01	19.0	
" "	" "	DP-2	ND*	0.04	5.0	
" "	" "	" "	ND*	0.01	10.0	
" "	" "	" "	ND*	0.02	20.0	

*ND = NON DETECT

As a covered entity under Title II of the Americans with Disabilities Act, the City of Los Angeles does not discriminate on the basis of disability and, upon request, will provide reasonable accommodation to ensure equal access to its programs, services and activities. For efficient handling of information internally and in the internet, conversion to this new format of code related and administrative information bulletins including MGD and RGA that were previously issued will allow flexibility and timely distribution of information to the public.

Table 1B - MITIGATION REQUIREMENTS FOR METHANE BUFFER ZONE (See notes)

2510 WHITTIER BLVD.

		Site Design Level	Level I		Level II		Level III		Level IV		Level V
		Design Methane Concentration (ppmv)	0 - 100		101 - 1,000		1,001 - 5,000		5,001 - 12,500		> 12,500
		Design Methane Pressure (See note 1) **(inches of water column)**	≤ 2"	> 2"	≤ 2"	> 2"	≤ 2"	> 2"	≤ 2"	> 2"	All Pressure
PASSIVE SYSTEM		De-watering System		X		X		X	X	X	X
PASSIVE SYSTEM	Sub-Slab Vent System	Perforated Horizontal Pipes		X		X		X	X	X	X
PASSIVE SYSTEM	Sub-Slab Vent System	Gravel Blanket Thickness Under Impervious Membrane		2"		3"		3"	2"	4"	4"
PASSIVE SYSTEM	Sub-Slab Vent System	Gravel Thickness Surrounding Perforated Horizontal Pipes		2"		3"		3"	2"	4"	4"
PASSIVE SYSTEM	Sub-Slab Vent System	Vent Risers		X		X		X	X	X	X
PASSIVE SYSTEM		Impervious Membrane		X		X		X	X	X	X
ACTIVE SYSTEM	Sub-Slab System	Mechanical Extraction System (See note 2)								X	X
ACTIVE SYSTEM	Lowest Occupied Space System	Gas Detection System (See note 3)		X		X		X	X	X	X
ACTIVE SYSTEM	Lowest Occupied Space System	Mechanical Ventilation (See Notes 3, 4, 5)		X		X		X	X	X	X
ACTIVE SYSTEM	Lowest Occupied Space System	Alarm System		X		X		X	X	X	X
ACTIVE SYSTEM		Control Panel		X		X		X	X	X	X
MISC. SYSTEM		Trench Dam		X		X		X	X	X	X
MISC. SYSTEM		Conduit or Cable Seal Fitting		X		X		X	X	X	X
MISC. SYSTEM		Additional Vent Risers (See note 5)									X

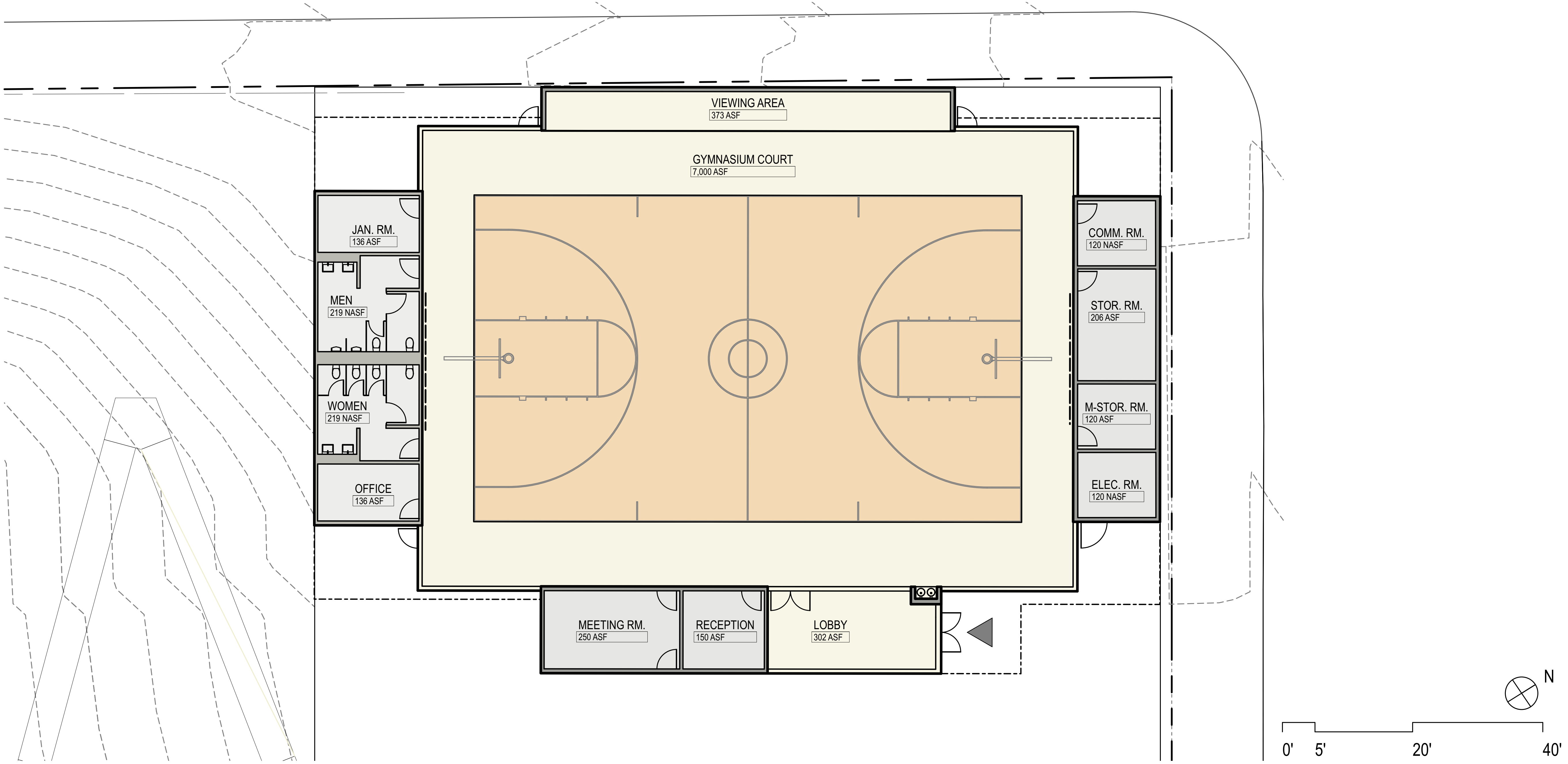
NOTES FOR TABLES 1A AND 1B:

"x" = Indicates a required mitigation component

1. De-watering is not required when the maximum Historical High Ground Water Table Elevation, or projecterd post-construction ground water level, is more than 12 inches below the bottom of the Perforated Horizontal Pipes.
2. The Mechanical Extraction System shall be capabale of providing an equivalent of a complete change of air 20 minutes of the total volume of the Gravel Blanket.
3. The mechanical ventilation system shall be capable of providing an equivalent of one complete change of the lowest occupied space every 15 minutes.
4. Vent openings to comply with Item IV.B.4 on sheet 1 may be used in lieu of mechanical ventilation.
5. The total quantity of the installed Vent Risers shall be increased to twice the rate for the Passive System.

FLOOR PLAN - OPTION A

PLAN DE EDIFICIO - OPCIÓN A



VIEW FROM WHITTIER BOULEVARD

VISTA DESDE EL BULEVAR WHITTIER

CORNER VIEW FROM WHITTIER BOULEVARD

VISTA DESDE LA ESQUINA DEL BULEVAR WHITTIER

PARK VIEW FROM RAMP

VISTA DESDE LA RAMPA EN EL PARQUE

PARK VIEW FROM LOWER LEVEL

VISTA DESDE EL NIVEL INFERIOR DEL PARQUE

BUILDING HEIGHT STUDY

ESTUDIO DE ALTURA DEL EDIFICIO