REVISED REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Proposed Arts District Center
1101 East 5th Street
Los Angeles, California

For
Art District Development, LLC
August 15, 2017

GeoDesign Project: ADD-1-01

GeoDesign Inc

August 15, 2017

Art District Development, LLC
1129 East 5th Street
Los Angeles, CA 90013

Attention: Kevin Chen

Revised Report of Geotechnical Engineering Services
Proposed Arts District Center
1101 East 5th Street
Los Angeles, California
GeoDesign Project: ADD-1-01

GeoDesign, Inc. is pleased to submit this geotechnical engineering report for the proposed mixed-use development to be constructed at 1101 East 5th Street in Los Angeles, California. Our services were performed in accordance with our revised proposal dated May 27, 2014.

We appreciate the opportunity to be of service to you. Please contact us if you have questions regarding this report.

Sincerely,

GeoDesign, Inc.

Christopher J. Zadoorian, G.E.
Principal Engineer

cc: Tom Hsieh, Togawa Smith Martin, Inc. (via email only)
Tom Greer, Togawa Smith Martin, Inc. (via email only)

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed Arts District Center to be constructed at 1101 East 5th Street in Los Angeles, California. The site location is shown on Figure 1.

The site is located at the northeast corner of Seaton Street and East 5th Street and is currently developed with a two-story, masonry brick commercial/warehouse building. The site is bound by Seaton Street on the west, Colyton Street on the east, and East 5th Street on the south. Existing one-story and three-story masonry buildings border the site on the north.

You furnished us with plans dated November 22, 2016 prepared by Togawa Smith Martin depicting the proposed development. Based on our review of the plans and our discussions with you, the proposed development will include the construction of a 12-level, above-grade, mixed-use building over 5 subterranean levels that extends 47 feet BGS.

Structural loading information was not available at the time this report was prepared; however, based on our experience with similar projects, we anticipate typical interior dead-plus-live column loading will be on the order of 1,500 to 2,500 kips.

The site is also located within a LADBS-designated methane zone and appropriate methane mitigation provisions are required in accordance with the LADBS Ordinance No. 175790.

Our investigation is summarized below followed by our conclusions and recommendations for the proposed development.

2.0 PURPOSE AND SCOPE

The purpose of our investigation was to collect subsurface information at the site to develop geotechnical design recommendations for the proposed development. Our specific scope of services included the following primary tasks:

- Reviewed historical site use information, including Sanborn maps, topographic maps, and aerial photographs
- Drilled four borings at the site to depths ranging between 67.0 and 77.0 feet BGS
- Performed field percolation testing in one boring
- Converted each boring to a soil gas monitoring well
- Performed soil gas measurements
- Evaluated geologic and seismic hazards at the site, including liquefaction potential and surface fault rupture potential
- Determined the design methane mitigation level (transmitted in a separate memorandum)
- Developed recommendations for on-site infiltration
- Developed foundation recommendations for the proposed development
- Developed seismic design parameters in accordance with the 2017 LABC
- Developed recommendations for the design of below-grade building walls

- Developed recommendations for temporary shoring, including considerations for surcharge loading from the adjacent buildings
- Developed recommendations for the design and construction of concrete floor slabs, including an estimate of the soil subgrade modulus
- Developed recommendations for site flatwork
- Prepared this report summarizing our investigation and presenting our design recommendations

3.0 SITE AND SUBSURFACE CONDITIONS

3.1 SURFACE CONDITIONS

The ground surface level at the site slopes gently to the south and ranges from approximately Elevation 258 to Elevation 256. The site is currently occupied with an at-grade, two-story masonry building.

3.2 HISTORICAL SITE DEVELOPMENT

We reviewed Sanborn maps, topographic maps, and aerial photographs dating back to 1894 to provide general background information regarding the historical site development.

Based on our review of this information, the site was developed with residences circa 1894. The current masonry buildings were constructed in phases between approximately 1906 and 1920.

3.3 SUBSURFACE CONDITIONS

We drilled four borings (B-1 through B-4) within the existing structure to depths ranging between 67.0 to 77.0 feet BGS at the locations shown on Figure 2. The borings were drilled using limited-access, hollow-stem auger drilling equipment.

Fill material up to 12.0 feet thick were encountered in the borings. The fill material generally consists of loose to medium dense, silty sand and contains construction debris, including brick fragments. Based on the age of the current development, we anticipate that deeper fill similar in nature is present at other locations on site.

In boring B-1 a 6.5-foot-thick layer of loose sand was encountered below the fill material to a depth of approximately 13 feet BGS that is underlain primarily by very dense sand, silty sand and very dense gravel with intermittent layers of medium dense sand with silt and/or stiff silt to the depths explored.

Gravel-sized particles are present within a majority of the sand layers. In several cases, sheared cobbles were recovered in the sampling tubes, indicating the presence of cobbles. Although not directly observable in the small-diameter borings drilled as part of this investigation, boulders are also likely present within the sand and gravel layers.

The larger-sized particles are common in this part of Los Angeles due to the proximity of the site to the Los Angeles River and the corresponding higher-energy geologic depositional environment.

Figures 3 and 4 present typical geologic cross sections depicting the generalized subsurface conditions. Logs of the borings are presented in Appendix A.

3.4 GROUNDWATER AND GROUNDWATER SEEPAGE

Neither groundwater nor groundwater seepage were encountered in the borings at the time they were drilled.

It should be noted that the borings were drilled in 2014 after several years of low levels of rainfall in Southern California and that it is common for localized perched water to be present within intermittent fine-grained (silt and clay) layers and typically present within the overall granular subsurface matrix. A layer of silty sand was encountered at a depth of approximately 25 feet BGS in boring B-2. Layers of silty sand and/or silt was encountered at depths of approximately 60 to 73 (or 70) feet BGS in borings B-1, B-3, B-4.

Based on our review of the *Seismic Hazard Zone Report for the Los Angeles 7.5-Minute Quadrangle* (CGS, formerly CDMG, 1998), the historical high groundwater level is at a depth of approximately 90 feet BGS.

3.5 FIELD PERCOLATION TESTING

We performed field percolation testing upon the completion of drilling in boring B-4 to estimate the on-site infiltration capacity of the on-site soils in accordance with *Administrative Manual, County of Los Angeles Department of Public Works, Geotechnical and Material Engineering Division, Guidelines for Design, Investigation, and Reporting Low Impact Development Stormwater Infiltration (LA County Stormwater Infiltration Guidelines), GS200.1, June 30, 2014*.

To perform the testing, we installed a 2-inch-diameter PVC pipe within the hollow-stem auger simultaneously as the auger was withdrawn from the hole. The lower 5 feet of the PVC pipe was screened and an end cap was installed at the bottom of the pipe. To prevent caving of the boring side wall, filter pack gravel was placed around the PVC pipe as the hollow-stem auger was withdrawn. The bottom of the PVC pipe was established at a depth of approximately 77 feet BGS.

The testing consisted of introducing water to the subsurface soils through the PVC pipe and measuring the rate of percolation. Prior to the start of the test, the soils at a depth of 77 feet were presoaked. During the presoak, water level was measured using a water-level meter and the rate of percolation recorded. After each water level measurement, water was added to refill the approximate initial water depth, and the procedure was repeated. The water level dropped more than 12 inches within 30 minutes or less during the pre-soak period; therefore, the presoak was considered completed.

Field percolation testing was initiated following the completion of the pre-soak process. Water was refilled to a depth of 73 feet BGS, and each water drop was recorded for a ten-minute interval. A stabilized rate of drop was obtained at the fifth reading, where the highest and lowest readings from three consecutive readings were within 10 percent of each other.

In accordance with *Administrative Manual, County of Los Angeles Department of Public Works, Geotechnical and Material Engineering Division, Guidelines for Design, Investigation, and Reporting Low Impact Development Stormwater Infiltration, GS200.1, June 30, 2014*, high flowrate percolation test procedures were used due to water draining faster than an infiltration rate of 14 inches per hour during the pre-soak procedures.

In accordance with the Los Angeles County Stormwater Infiltration Guidelines, the results of the field testing indicate an infiltration rate for the on-site soil at a depth of 77 feet BGS of approximately 16 inches per hour. The results of percolation testing are presented in Appendix B.

After the completion of the percolation test, the PVC pipe was removed from the boring, the boring was backfilled with bentonite chips, and the boring was converted into a soil gas monitoring well. Excess soil cuttings were placed in 55-gallons drums for disposal. The disposal of drums was completed on July 10, 2014.

3.6 SOIL GAS TESTING

Upon completion of borings B-1 through B-4, the borings were converted into methane monitoring wells to comply with the City of Los Angeles (City) LADBS requirements for soil gas testing in accordance with LADBS Ordinance No. 175790.

The results of the methane testing were summarized in a memorandum dated June 16, 2014 that concluded the site is classified as Level I per LADBS' standards. Level I mitigation includes the installation of an impermeable barrier below the building floor slab and behind the below-grade building walls in conjunction with a passive ventilation system.

Construction details for the methane monitoring wells are shown on the exploration logs presented in Appendix A.

3.7 GEOTECHNICAL LABORATORY TESTING

Geotechnical laboratory testing was performed on selected samples from the borings. The following tests were performed:

- In-place moisture and density
- Direct shear

Results of the geotechnical testing are presented in Appendix A.

Please note that consolidation testing was attempted on samples of the very dense sand with gravel at the foundation level and below. However, the samples were generally too disturbed to yield meaningful results.

4.0 GEOLOGIC AND SEISMIC HAZARDS

Primary geologic and seismic hazards that may impact the development project include surface fault rupture, liquefaction potential, and static and seismic slope stability issues. Each is addressed briefly in the following sections.

4.1 SURFACE FAULT RUPTURE

Faults in Southern California are considered active, potentially active, and inactive based on criteria developed by CGS for the Alquist-Priolo Earthquake Fault Zoning Program (Hart, 1999). By definition, an active fault is one that has had surface displacement within Holocene time (approximately the last 11,000 years). A potentially active fault is one that has demonstrated surface displacement of Quaternary age deposits (last 1.6 million years). Inactive faults have not moved in the last 1.6 million years.

The primary purpose of the Alquist-Priolo Earthquake Fault Zoning Program is to identify sites that have a potential for surface rupture due to active faults that are in close proximity to the site. In such cases, a building setback zone is established to mitigate the potential for surface rupture.

The site is not located within an Alquist-Priolo Special Study zone. Based on our review of the *Fault Activity Map of California* (CGS, 2010), the site is not located within an active fault zone. Therefore, the potential for surface fault rupture at the site is considered to be very low.

4.2 LIQUEFACTION POTENTIAL AND SEISMIC (DRY) SETTLEMENT

Liquefaction generally occurs in saturated, loose to medium dense, granular soil and in saturated, soft to moderately firm silt as a result of strong ground shaking. As the density and/or particle size of the soil increases and as the confinement (overburden pressure) increases, the potential for liquefaction decreases. Typically, saturated soils within the upper 50 feet of the ground surface or lowest adjacent grade are considered subject to liquefaction.

Based on the CGS *Seismic Hazard Evaluation of the Hollywood 7.5-Minute Quadrangle, Los Angeles County, California*, Open File Report 98-17, the site is not in an area subject to liquefaction.

The native soils at the planned foundation levels consist of very dense sand and gravel with sand and cobbles. In addition, the depth to groundwater is greater than 90 feet BGS at the site.

Based on these conditions, the potential for liquefaction at the site is considered to be very low.

Seismic (dry) settlement can occur in loose to medium dense, granular soil as a result of strong ground shaking. As stated above, the soil at the planned foundation levels is very dense sand and gravel with sand and cobbles, and is not considered subject to seismically induced (dry) settlement.

4.3 SLOPE INSTABILITY
Slope instability may occur in hillside areas and at sites with slopes where adverse geologic conditions are present and/or as a result of soil liquefaction (generally referred to as lateral spreading).

The ground surface level at the site is generally flat with only a very modest gradient to the south, and the potential for the liquefaction is very low.

The site is not located within a static or seismic slope stability hazard zone. Therefore, the potential for static or seismic slope instability at the site is considered to be negligible.

4.4 STRONG GROUND MOTION
The site is subject to strong ground shaking that would result from an earthquake occurring on a nearby or distant fault source; however, this hazard is common in Los Angeles and can be mitigated by following LABC seismic design requirements as discussed in Section 6.6 herein.

5.0 CONCLUSIONS

5.1 GENERAL
The site is free from geologic or seismic hazards that would preclude the proposed development, and the proposed development is considered feasible from a geotechnical perspective.

The site is subject to strong ground shaking that would result from an earthquake occurring on a nearby or distant fault source; however, this hazard is common in Los Angeles and can be mitigated by following LABC seismic design requirements.

The site is also located within a LADBS-designated methane zone and appropriate methane mitigation provisions are required in accordance with the LADBS Ordinance No. 175790.

The subsurface materials encountered at the site consist of high-energy (gravel, cobbles, and possibly boulders) deposits that will present challenges to excavation and shoring.

It would prudent, when access is available on site, to perform supplemental borings using bucket-auger (large diameter) drilling equipment to define the particle sizes and the depth intervals for cobbles and potential boulders at some time during the final bidding or pre-construction phase of the project.

5.2 FOUNDATION AND FLOOR SLAB SUPPORT
The soils anticipated at the foundation and floor slab levels are generally very dense sand with gravel and cobbles or very dense gravel with sand and cobbles. These soils are suitable for support of the proposed building on spread and continuous footings and the proposed building floor slab on grade.

The presence of gravel, cobbles, and potentially boulders will likely result in uneven and/or non-uniform excavation bottoms and result in additional bottom preparation effort and possibly materials to establish an even, working bottom.

5.3 SHORING, EXCAVATIONS, AND PERMANENT BELOW-GRADE WALLS

Temporary shoring will be required to provide support for the mass excavation. Drilled solider piles used for temporary shoring will encounter gravel, cobbles, and potentially boulders, and the presence of these materials will present challenges for conventional solider pile drilling equipment.

It may be necessary to use specialized equipment, including, but not limited to, core barrels and potentially hydraulic rock breakers to advance soldier pile shaft and/or tieback excavations.

The presence of gravel, cobbles, and potentially boulders will likely result in uneven and/or non-uniform excavation sides and result in additional preparation effort and materials during lagging installation.

The drilling and excavation could encounter localized zones of perched water where silt layers are present.

Considerations for temporary and permanent support for the existing masonry warehouse building foundations will also be required. Preliminary design recommendations are presented herein; however, additional information outlined in Section 6.2.2 regarding the adjacent structures will be required prior the final design of temporary shoring and below-grade building walls.

5.4 ON-SITE MATERIALS

On-site granular soils are suitable for re-use in required fills; however, the on-site excavations will generate a significant percentage of relatively large-sized particles, defined herein as particles greater than 3 inches in largest dimension. Larger-size particles are not suitable for re-use in required fill and will require processing to meet the specifications presented herein for fill materials.

Remnants of the prior development, including construction debris, are not suitable for re-use in the required fills.

5.5 METHANE GAS MITIGATION

Based on the results of the soil gas testing, the site is classified as Design Level I. As a minimum, Design Level I requires a passive sub-slab venting system and the installation of a soil gas barrier as discussed in Section 3.6.

6.0 RECOMMENDATIONS

6.1 FOUNDATIONS

6.1.1 Allowable Bearing Pressure

The proposed building may be supported on spread footings established in the very dense sand and gravel encountered at the planned foundation levels. Spread footings established at least 2 feet below the lowest adjacent grade or top of floor slab may be designed using an allowable bearing pressure of 12,000 psf and increased by 500 psf for each additional foot of embedment depth to a maximum value of 15,000 psf. The additional 500 psf increase is assumed to begin below 2 feet from the lowest adjacent grade or floor level.

The recommended bearing pressures are a net value and apply to the total of dead and long-term live loads and may be increased up to one-third when considering earthquake or wind loads. The weight of the footing and overlying backfill can be neglected when calculating footing loads.

Core shear walls, if planned, may be supported on mat foundations established in the very dense sand and gravel encountered at the planned foundation levels. A modulus of subgrade reaction equal to 225 pci may be used for design of mat foundations. The recommended modulus value includes a reduction for the anticipated size of the mat foundations.

6.1.2 Settlement

Based on the assumed column loading, we estimate total foundation settlement on the order of 1 inch or less and differential settlement between footings on the order of ¼ inch or less for spread and mat foundations designed and constructed as recommended herein.

We should be provided with structural loading information during the design development phase to evaluate our preliminary settlement settlements.

6.1.3 Lateral Resistance

For spread footings, lateral loading may be resisted by foundations using a passive pressure of 400 psf for footings where the concrete is placed directly against the undisturbed, very dense sand and gravel. A coefficient of friction equal to 0.4 may be used when calculating resistance to sliding for foundations bearing on undisturbed, stiff or dense alluvial soils. A factor of safety of 1.5 was used to compute the recommended allowable passive and frictional resistance design values.

6.1.4 On-Site Stormwater Infiltration

It is our understanding that on-site groundwater infiltration may be implemented as part of Standard Urban Storm Water Mitigation Plan mitigation measures. Considering that the proposed development is a zero-lot limit project, any infiltration would necessarily need to be located within the building footprint via the use of drywell elements.

Preliminary recommendations are presented in Section 6.9 for the design of deep drywells.

6.2 PERMANENT BELOW-GRADE WALLS

6.2.1 Design Lateral Earth Pressures

For static conditions, drained below-grade building walls should be designed to resist a trapezoidal-shaped at-rest lateral earth pressure distribution equal to 32H psf as shown on Figure 5.

For seismic loading conditions, drained below-grade building walls that retain 12 feet or more of soil should be designed to resist a triangular-shaped active lateral earth pressure distribution equal to 33H psf in conjunction and a triangular-shaped seismic lateral earth pressure distribution equal to 16H psf as shown on Figure 6.

The upper 10 feet of the below-grade building walls should also be designed to resist a uniform lateral pressure of 100 psf to account for normal traffic loading as shown on Figures 5 and 6.

The load combination (active and seismic earth pressure) and the shape of the seismic pressure distribution are each based on *Seismic Earth Pressures on Cantilevered Retaining Structures,* Journal of Geotechnical and Geoenvironmental Earthquake Engineering, Volume No. 136, October 2010 (Sitar and Atik, 2010) and *Seismic Earth Pressures: Fact or Fiction,* Earth Retention Conference, 2010, Seismic Evaluation of Retention Systems, pp 656 – 673 (Sitar et al., 2010).

Though not currently planned, if the surface at the top of the wall is sloped, the recommended lateral earth pressures should be increased as indicated in Table 1.

Table 1. Permanent Below-Grade Walls – Lateral Earth Pressures

Slope Inclination at Top of Wall (H:V)	Increase in Lateral Earth Pressure (percent)
1:1	200
1.5:1	165
2:1	150

6.2.2 Surcharge Loading from Adjacent Buildings

Foundations for existing buildings that remain adjacent to the north of the proposed building will impose lateral surcharge loading on the proposed below-grade building walls.

Specific foundation information, including estimated applied bearing pressure and depth, width, and horizontal distance from the back of the permanent below-grade wall, will be required to provide final surcharge loading design recommendations.

For preliminary design and programming purposes, we assumed the existing foundations are in close proximity to the existing ground surface level and computed lateral surcharge pressures assuming that the existing buildings have continuous footings 3 feet wide and an applied bearing pressure of approximately 2,000 psf or less.

Based on these assumptions, preliminary surcharge pressures are presented on Figures 5 and 6.

6.2.3 Wall Back-Drainage

Permanent retaining walls should be constructed with adequate back-drainage to prevent the buildup of hydrostatic pressure behind the walls. Typically, a pre-fabricated geo-composite drainage board is fixed to the shoring wall, and the below-grade building wall is constructed by the placement of shotcrete directly against the drainage board.

In addition to drainage boards, the City requires the installation of rock pockets consisting of 1 cubic foot of crushed rock spaced at 8-foot centers around the perimeter of the below-grade building walls to promote drainage as noted in Section 91.7013.11 of LABC. The City also requires each rock pocket to be drained at each 8-foot center back into the building.

The impact of this requirement on the design and construction of the proposed development is complicated by another City requirement related to methane mitigation as outlined in LADBS Bulletin / Public Building Code 2002-101. The methane mitigation requirement is to include vent risers at each penetration through the below-grade building wall for the purpose of mitigating the potential for methane gas to enter the building through the penetrations. The combined result of the two requirements is that an alternative method to provide back-drainage in a manner that meets each of the City's requirements may be desirable.

One alternative method includes perimeter drainage element at the base of the below-grade building walls. Per City requirements, the subject pipe would only need to be drained into the building at one or two locations and conveyed to the building sump system. Therefore, the number of vent risers required as a function of the wall back-drainage system could be significantly reduced.

6.3 TEMPORARY EXCAVATIONS AND VERTICAL CUTS

If necessary, temporary, unsurcharged slopes should not exceed a 1H:1V gradient when constructed in existing fill and/or native materials. Such temporary slopes should not exceed 15 feet in height.

Temporary vertical cuts that will be beneficial for foundation construction may be made into the dense native materials, but should not exceed 4 feet in height.

Temporary cut slopes should be protected from erosion by directing surface water away by placing sand bags at the top of the slopes and during wet weather, covering the slopes with plastic sheeting.

6.4 TEMPORARY SHORING

6.4.1 Temporary Shoring Design Lateral Earth Pressures

Typically, cantilevered shoring is feasible for retained heights of approximately 15 feet or less, and braced shoring typically becomes economical for retained heights exceeding 15 feet.

Cantilevered shoring should be designed to resist a triangular lateral earth pressure distribution where the maximum value is 30H psf.

Internally braced shoring should be designed to resist a trapezoidal earth pressure distribution where the maximum value is equal to 22H psf.

For cantilevered and braced shoring design, where the surface at the top of the shoring is sloped, the recommended lateral earth pressures should be increased as indicated in Table 1.

The north shoring walls should also be designed to resist the surcharge pressure shown on Figures 5 and 6, as applicable.

The upper 10 feet of the below-grade building walls should be designed to resist a uniform lateral pressure of 100 psf to account for normal traffic loading.

In addition, when developing design drawings for temporary shoring, the drawings should include consideration for the location of construction cranes and other potentially heavy equipment or loads that may act against the shoring system.

6.4.2 Soldier Piles

For the design of soldier piles spaced at least 2 diameters on-center, the allowable lateral bearing value (passive value) of the native soil below the level of excavation may be assumed to be 400 psf per foot of depth, up to a maximum of 4,000 psf of depth. To develop the full lateral value, the contractor shall ensure firm contact between the soldier piles and the undisturbed soil.

If the embedded portion of the soldier pile shaft is filled with lean-mix concrete with a minimum compressive strength of 2,000 psi, then the effective width of the soldier pile shaft for use in developing passive resistance may be assumed to be twice the diameter of the shaft. If the embedded portion of the soldier pile shaft is filled with other material (such as low-strength sand-cement slurry), the effective width of the soldier pile should be limited to the diagonal dimension of the soldier pile beam.

The frictional resistance between the soldier piles and the retained earth may be used in resisting the downward component of the tieback anchor loads. For design, the coefficient of friction between the soldier piles and the retained earth is 0.4. This value is based on the assumption that uniform full bearing will be developed between the steel soldier beam and the shaft backfill material and the retained earth.

In addition, provided the portion of the soldier piles below the excavated level is backfilled with structural concrete, the soldier piles below the excavated level may be used to resist downward loads.

We do not anticipate that vibration of solider piles will be an effective method of installation given the presence of gravel, cobbles, and potentially boulders. However, if planned by the shoring contractor, vibration for solider piles should not be used within 80 feet of existing structures and the peak particle velocity should not exceed 0.5 inch per second. If the peak particle velocity is exceeded, the vibration installation operation should be terminated and a mitigation plan should be submitted by the contractor for review and approval prior to resuming vibration.

Where vibratory methods are used, the diagonal of the solider pile beam may be used for the width when computing allowable passive resistance.

Pre-drilling, if used in conjunction with vibratory methods, should not extend below the bottom of the planned excavation and the diameter of the pre-drilling auger should be less than the beam diagonal.

For resisting downward loads, the frictional resistance between the concrete soldier piles and the soil below the excavated level may be taken equal to 400 psf for drilled solider piles. For soldier piles that are vibrated into the supporting soil, the frictional resistance between the soldier piles and the soil below the excavated level may be taken as 800 psf.

6.4.3 Timber Lagging

Continuous lagging will be required between the soldier piles. The soldier piles and anchors should be designed for the full anticipated lateral pressure; however, the pressure on the lagging will be less due to arching in the soils. For clear spans of up to 6 feet, we recommend that the lagging be designed for a triangular distribution of earth pressure where the maximum pressure is 400 psf at the mid-line between soldier piles and 0 psf at the soldier piles.

The presence of cobbles and potentially boulders will result in uneven and non-uniform excavation sidewalls. Caution should be used when removing cobbles and boulders, where present, to minimize disturbance to overlying soils. Voids created due to removal should be filled with grout as soon as possible.

6.4.4 Tiebacks

Tieback friction anchors may be used to resist lateral loads. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn at 35 degrees with the vertical through the bottom of the excavation. The anchors should extend at least 20 feet beyond the potential active wedge and to a greater length as necessary to develop the desired capacities.

The capacities of anchors should be determined by testing the initial anchors as outlined below. We anticipate that gravity-filled anchors will be capable of achieving an allowable bond strength of 1 to 3 kips per lineal foot of anchor, depending on the method of construction. A variety of methods is available for construction of anchors. If post-grouted anchors are used, we estimate that the anchors will develop resistance on the order of three times the estimated value.

We recommend that the shoring designer and contractor be responsible for selecting the appropriate bond length and installation methods to achieve the required capacity.

Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. If the anchors are spaced at least 6 feet on-centers, reduction in the capacity of the anchors does not need to be considered due to group action.

The anchors may be installed at angles of 15 to 40 degrees below the horizontal. Caving of the anchor holes should be anticipated and provisions made to minimize such caving. The anchors should be filled with concrete placed by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. To minimize chances of caving, we suggest that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flushed with the face of the excavation. The sand backfill may contain a small amount of cement to allow the sand to be placed by pumping. For post-grouted anchors of 8-inch diameters or less, the anchor may be filled with concrete to the surface of the shoring.

Our representative should select a representative number, at least 10 percent of total number of anchors, for 24-hour, 200 percent tests and 200 percent quick tests. The purpose of the 200 percent test is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

For post-grouted anchors where concrete is used to backfill the anchor along its entire length, the test load should be computed as required to develop the appropriate friction along the entire bonded length of the anchor.

We estimate that the influence of post-grouting and the adjacent soils within the bonded length of the anchors will be less than 5 feet from the anchor.

The total deflection during the 24-hour, 200 percent tests should not exceed 12 inches during loading. The anchor deflection should not exceed 0.75 inch during the 24-hour period, measured after the 200 percent test load is applied. If anchor movement after the 200 percent load has been applied for six hours is less than 0.5 inch and the movement over the previous four hours has been less than 0.1 inch, the test may be terminated.

For the quick 200 percent tests, the 200 percent test load should be maintained for 30 minutes. The total deflection of the anchor during the quick 200 percent tests should not exceed 12 inches. Deflection after the 200 percent test load has been applied should not exceed 0.75 inch during the 30-minute period. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

All of the production anchors should be pre-tested to at least 150 percent of the design load. Total deflection during the tests should not exceed 12 inches. The rate of creep under the 150 percent test should not exceed 0.1 inch over a 15-minute period for the anchor to be approved for the design loading.

After a satisfactory test, each production anchor should be locked off at the design load. The locked-off load should be verified by rechecking the load in the anchor. If the locked-off load varies by more than 10 percent from the design load, the load should be reset until the anchor is locked off within 10 percent of the design load. The installation of the anchors and the testing of the completed anchors should be observed by a representative of our firm.

6.4.5 Raker Bracing

As an alternative to tiebacks, raker bracing may be used to internally brace the soldier piles. If used, raker bracing could be supported laterally by temporary concrete footings (aka deadmen) or by the permanent interior footings. For design of such temporary footings poured with the bearing surface normal to the rakers inclined at 45 to 60 degrees with the vertical, a bearing value of 6,000 psf may be used for footings on the dense or stiff native soils provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade. To reduce the movement of the shoring, the rakers should be tightly wedged against the footings and/or shoring system.

6.4.6 Monitoring

Monitoring the performance of the shoring system is recommended. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all the soldier piles.

It is difficult to accurately predict the amount of deflection of a shoring system. It should be realized, however, that some deflection will occur. We estimate that this deflection could be on the order of 1 inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of the utilities in the adjacent streets. If it is desired to reduce the deflection of the shoring, a greater active pressure could be used in the shoring design.

A pre-construction survey of the adjacent masonry warehouse buildings should be performed to establish baseline conditions.

Given the relative age of the infrastructure in the site vicinity, we recommend a diligent mapping of existing utilities be performed and provided to us, the shoring designer, and the shoring contractor to evaluate potential impact of the proposed shoring and excavation on existing utilities.

6.4.7 Shoring Construction Considerations

Due to the presence of localized fill materials, cobbles, and potentially boulders; granular soils that may be subject to caving; and potential groundwater seepage perched on fine-grained layers at depth, difficult drilling is expected for soldier pile and tieback installations.

It may be necessary to use specialized equipment, including, but not limited to, core barrels and potentially hydraulic rock breakers to advance soldier pile shaft and/or tieback excavations.

Provisions to mitigate caving of shaft sidewalls, including, but not limited to, the use steel casing, may also be required. The shoring contractor should carefully evaluate the native soil conditions to determine what means and methods are required to install the components of shoring system.

6.5 FLOOR SLABS

The proposed building floor slab may be established on the very dense granular soils present at the planned lowest finish floor level

Satisfactory subgrade support for floor slabs supporting up to an estimated 400 psf areal loading on the dense native alluvial soils can be obtained provided the building areas are prepared as described previously.

Typically, it is common to include a capillary break beneath building floor slabs that will have moisture-sensitive flooring. Typically, a capillary break section consists of 6 inches of gravel underlying a 15-mil HDPE membrane. However, since the native materials at the planned finish floor level are generally similar to gravel, those materials may be used as the 6-inch gravel section.

6.6 SEISMIC DESIGN

The current LABC methodology for determining the seismic design parameters follows the procedure outlined in the ASCE 7-10 document. The procedure outlined in ASCE 7-10 uses the MCE as the basis of the design. The MCE is defined as an earthquake that results in ground motions that have a 2 percent chance of being exceeded in 50 years (a 2,475-year recurrence interval).

Based on the data available at this time, the soil profile type for the site is S_D. However, we anticipate that a more favorable classification of S_C could be obtained by performing site-specific shear wave velocity measurements during the design development phase of the project.

Based on a soil profile type $S_{D,}$ Table 2 summarizes the seismic design parameters in accordance with ASCE 7-10, Section 21.4 for use in the seismic design of the proposed development.

Table 2. Seismic Design Parameters

Parameter	**Short Period (T_s = 0.2 second)**	**1-Second Period (T_1 = 1.0 second)**
MCE Spectral Acceleration, S	2.362	0.827
Site Class	D	
Site Coefficient	F_a = 1.0	F_v = 1.5
Adjusted Spectral Acceleration	S_{MS} = 2.362 g	S_{M1} = 1.241 g
Design Spectral Response Acceleration Parameters	S_{DS} = 1.575 g	S_{D1} = 0.827 g

6.7 SITE PREPARATION

Site preparation for this project will primarily include exposing the bottom of foundations and floor slabs and preparing soils at the bottom of trenches, behind below-grade walls, and behind free-standing site retaining walls to receive backfill. For foundation and floor slab support, the exposed bottoms do not require special preparation, except when disturbed by construction activities.

In this case, loose or otherwise disturbed soils should be removed and either replaced with structural concrete for footing bottoms or re-compacted prior to the placement of concrete for floor slabs.

The presence of cobbles and potentially boulders will result in uneven and non-uniform excavation bottoms. Removal of cobbles and boulders will require backfill. Due to the anticipated irregular nature of this process, we recommend the use of ¾-inch-minus crushed rock and/or two-sack sand-cement slurry as local backfill where isolated cobbles and/or boulders are removed. Soil backfill may be used in larger areas where it is feasible to place and compact soil.

Provided the exposed bottom of the area to receive fill consists of soil, the upper 6 inches should be scarified and re-compacted to the degree of relative compaction recommended in Section 6.8.3.

If gravel or cobbles are exposed at the bottom of a required excavation, voids due to removal of oversized particles should be filled as recommended above and the surface should be densified with plate vibratory compaction equipment to ensure uniform support for new fill.

6.8 GRADING CONSIDERATIONS

6.8.1 General

If not carefully executed, site preparation can result in the presence of disturbed and/or excessively soft soil conditions. This may require additional effort to mitigate or in more extreme cases, if not detected, could result in significant costs to repair damage to flatwork or structures.

Earthwork should be planned and executed to minimize subgrade disturbance. Soil that has been disturbed during site preparation activities and/or soft or loose zones identified during probing should be removed beneath floor slabs.

6.8.2 Materials for Fill

Fill materials should be free of organic matter and other deleterious materials and, in general, should consist of particles no larger than 3 inches in largest dimension.

The on-site native granular soils are suitable for use in the required fills provided particles larger than 3 inches in largest dimension are removed. Where larger-sized materials are used, the percentage of these materials in a representative section of the fill should be limited to 5 percent.

The on-site native silty soils are not considered suitable for use in structural fills or within 2 feet of floor slabs or other flatwork, but may be used as secondary fill in landscaping areas.

Imported fill materials should have a sand equivalent of at least 35 and should be approved by our firm prior to import to the site.

6.8.3 Compaction

All granular fill materials should be compacted to at least 95 percent of the maximum dry density at or near the optimum moisture content, as determined by ASTM D 1557. Cohesive fills, though not anticipated for this project, should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D 1557, and moisture conditioned 2 to 4 percent over the optimum moisture content.

Fill materials should be placed in loose lifts not exceeding 8 inches in thickness, properly moisture conditioned, and mechanically compacted to the minimum required density. For granular fills, compaction may be achieved using heavy equipment and vibration.

6.8.4 Site Drainage

Adequate site drainage should be maintained at all times. Site drainage should be collected and routed to suitable discharge locations.

6.9 STORMWATER INFILTRATION

For preliminary design purposes, we recommend that the drywell elements extend to a depth of at least 65 feet BGS. A design infiltration rate of 16 inches per may be used for drywells discharging into the native granular deposits below a depth of 70 feet BGS.

Our preliminary recommendations should be evaluated once a stormwater infiltration concept is developed by the civil engineer.

7.0 CONSTRUCTION OBSERVATION

Geotechnical testing and observation during construction is considered to be a continuing part of the geotechnical consultation. To confirm that the recommendations presented herein remain applicable, our representative should be present at the site to provide appropriate observation and testing.

8.0 LIMITATIONS

We have prepared this report for use by Art District Development, LLC and members of the design and construction teams for the proposed development. The data and report can be used for estimating purposes, but our report, conclusions, and interpretations should not be construed as a warranty of the subsurface conditions and are not applicable to other sites.

Soil borings indicate soil conditions only at specific locations and only to the depths penetrated. They do not necessarily reflect soil strata or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during the course of excavation and construction, re-evaluation will be necessary.

The recommendations presented in this report are based on the current site development plan and structural information provided to us by the project team. If design changes are made, we should be retained to review our conclusions and recommendations and to provide a written evaluation or modification.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences or procedures, except as specifically described in our report for consideration in design.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with that degree of skill and care ordinarily exercised by reputable geotechnical consultants practicing in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

◆ ◆ ◆

We appreciate the opportunity to be of continued service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,

GeoDesign, Inc.

Christopher J. Zadoorian, G.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
CHRISTOPHER JOHN ZADOORIAN
No. 2493
GEOTECHNICAL
STATE OF CALIFORNIA

Signed 08/15/2017

REFERENCES

Administrative Manual, County of Los Angeles Department of Public Works, Geotechnical and Material Engineering Division, Guidelines for Design, Investigation, and Reporting Low Impact Development Stormwater Infiltration, GS200.1, June 30, 2014.

California Division of Mines and Geology, 1998, “State of California Seismic Hazard Zone Map, Los Angeles Quadrangle, Official Map,” released March 25, 1999.

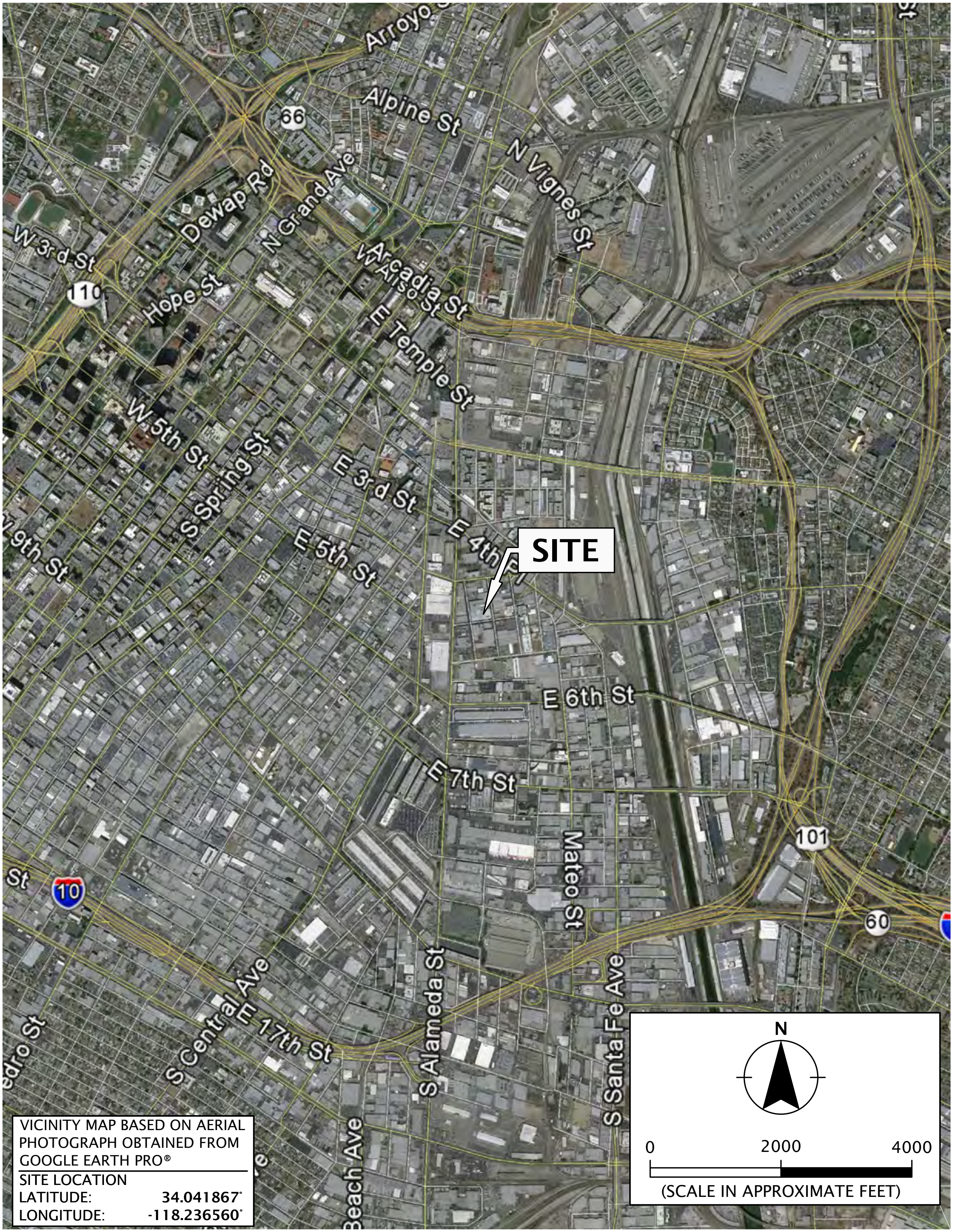
California Geological Survey, 2010, “Fault Activity Map of California.”

Hart, E. W., 1973, revised 1999, "Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps," California Division of Mines and Geology Special Publication 42.

Sitar and Atik, 2010, *Seismic Earth Pressures on Cantilevered Retaining Structures,* Journal of Geotechnical and Geoenvironmental Earthquake Engineering, Volume No. 136, October 2010.

Sitar et al., 2010, *Seismic Earth Pressures: Fact or Fiction,* Earth Retention Conference, 2010, Seismic Evaluation of Retention Systems, pp 656 – 673.

FIGURES



VICINITY MAP BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®

SITE LOCATION	
LATITUDE:	34.041867°
LONGITUDE:	-118.236560°

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VICINITY MAP

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PROPOSED ARTS DISTRICT CENTER
LOS ANGELES, CA

FIGURE 1

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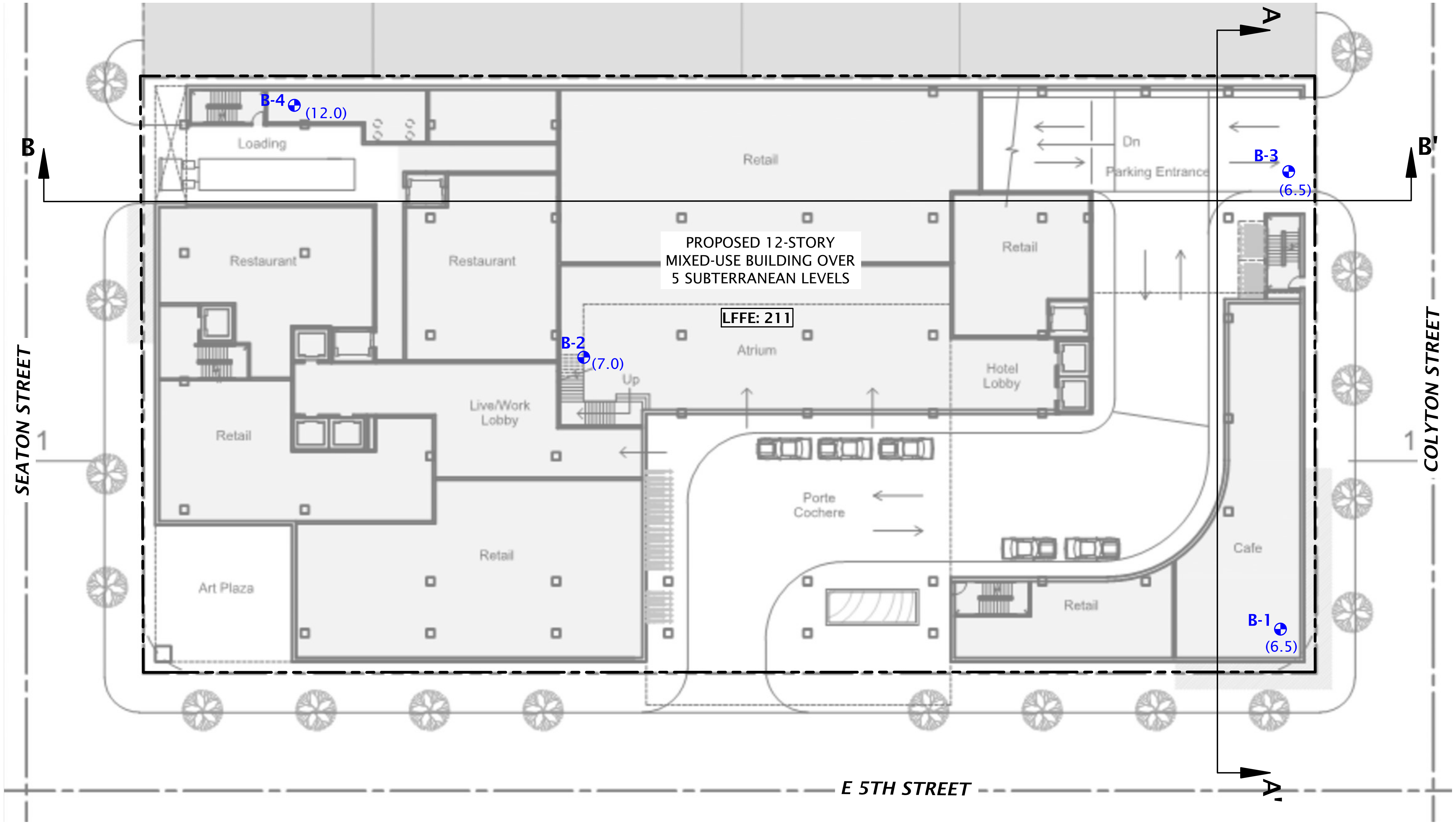
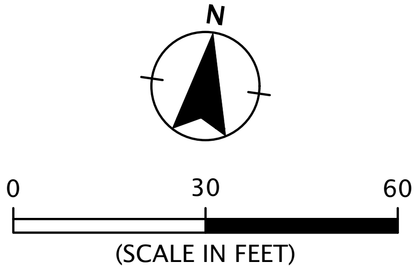


LEGEND:

B-1 ● (6.5)	BORING (FILL THICKNESS IN FEET)
— - — - —	APPROXIMATE SITE BOUNDARY
LFFE: 211	LOWEST FINISHED FLOOR ELEVATION (FEET)
A A'	CROSS SECTION



SITE PLAN BASED ON IMAGE OF SHEET 2
SITE PLAN DATED NOVEMBER 22, 2016
PREPARED BY TOGAWA SMITH MARTIN.

FIGURE 2

SITE PLAN

PROPOSED ARTS DISTRICT CENTER
LOS ANGELES, CA

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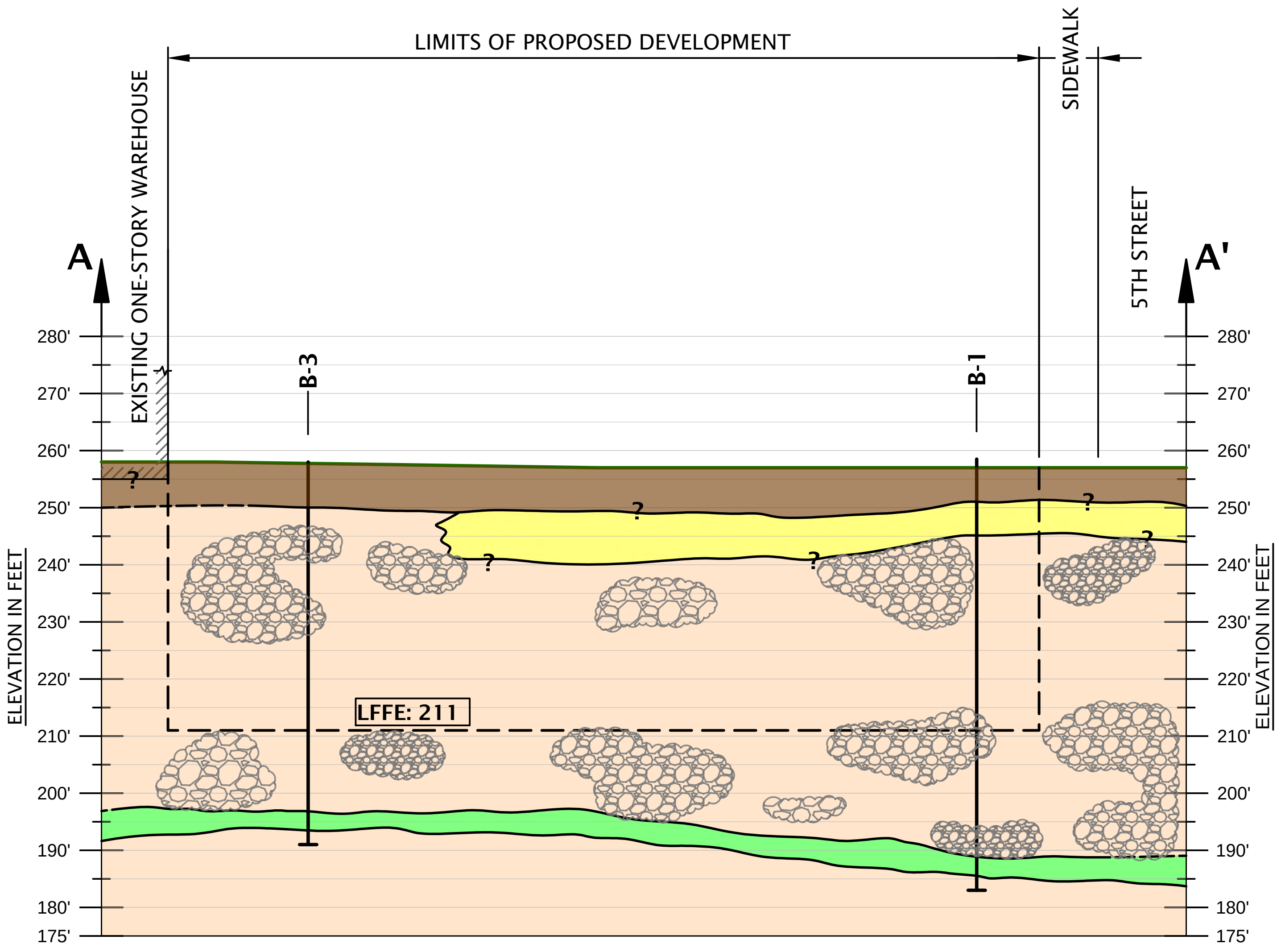
LEGEND:

～	EXISTING TOPOGRAPHY
⊥	BORING
LFFE: 211	PROPOSED LOWEST FINISHED FLOOR ELEVATION (FEET)
🟫	FILL SOILS
🟨	LOOSE TO MEDIUM DENSE SAND
🟧	VERY DENSE SAND WITH GRAVEL AND COBBLES AND SANDY GRAVEL
🟩	SANDY SILT AND SILTY SAND
○	COBBLES AND POTENTIALLY BOULDERS



0 30 60
(VERTICAL SCALE IN FEET)

0 30 60
(HORIZONTAL SCALE IN FEET)

FIGURE 3

CROSS SECTION A-A'

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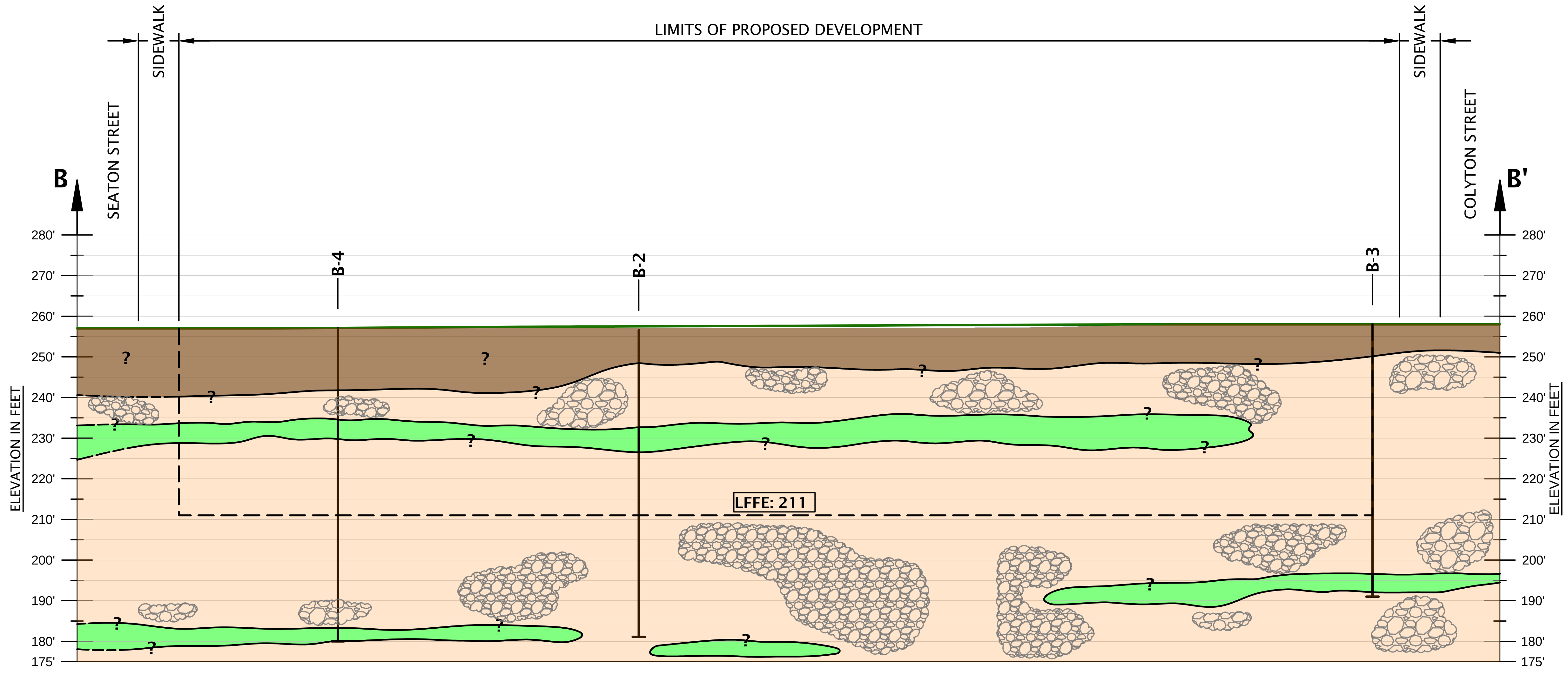

LEGEND:

～	EXISTING TOPOGRAPHY
⊥	BORING
LFFE: 211	PROPOSED LOWEST FINISHED FLOOR ELEVATION (FEET)
🟫	FILL SOILS
▭	VERY DENSE SAND WITH GRAVEL AND COBBLES AND SANDY GRAVEL
🟩	SANDY SILT AND SILTY SAND
○	COBBLES AND POTENTIALLY BOULDERS

0 30 60
(VERTICAL SCALE IN FEET)

0 30 60
(HORIZONTAL SCALE IN FEET)

FIGURE 4

CROSS SECTION B-B'

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File Name: J:\A-D\ADD\ADD-1\ADD-1-01\Figures\CAD\ADD-1-01-SP03.dwg | Layout: FIGURE 4

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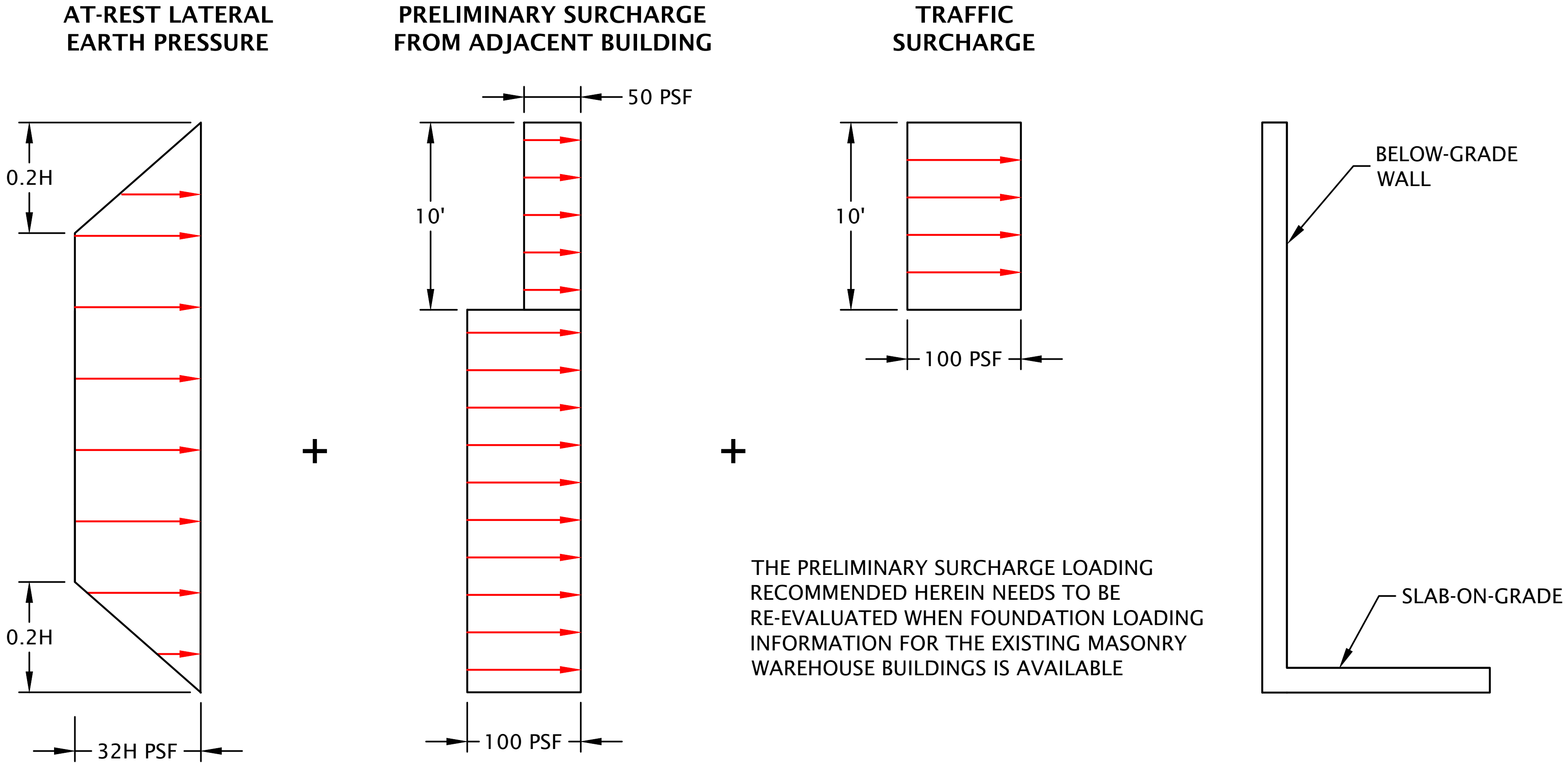
File Name: J:\A-D\ADD\ADD-1\ADD-1-01\Figures\CAD\ADD-1-01-DET01.dwg | Layout: FIGURE 5



THE PRELIMINARY SURCHARGE LOADING RECOMMENDED HEREIN NEEDS TO BE RE-EVALUATED WHEN FOUNDATION LOADING INFORMATION FOR THE EXISTING MASONRY WAREHOUSE BUILDINGS IS AVAILABLE

NOT TO SCALE

NOTES:

1. FIGURE SHOULD BE USED IN CONJUNCTION WITH REPORT TEXT.
2. THE LATERAL EARTH PRESSURES ARE UNFACTORED.

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AT-REST LATERAL EARTH PRESSURE - DRAINED

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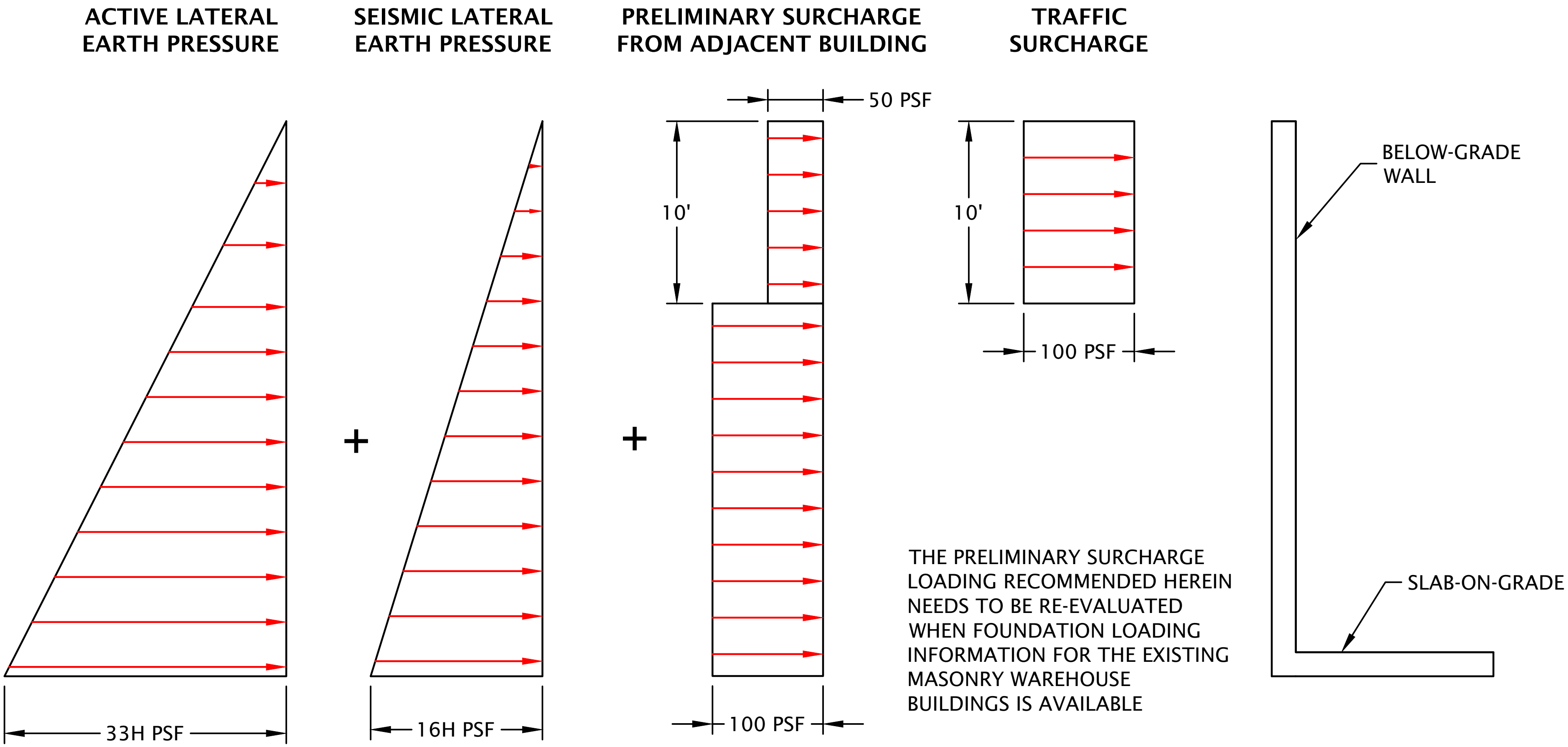
LOS ANGELES, CA

FIGURE 5



THE PRELIMINARY SURCHARGE LOADING RECOMMENDED HEREIN NEEDS TO BE RE-EVALUATED WHEN FOUNDATION LOADING INFORMATION FOR THE EXISTING MASONRY WAREHOUSE BUILDINGS IS AVAILABLE

NOT TO SCALE

NOTES:

1. FIGURE SHOULD BE USED IN CONJUNCTION WITH REPORT TEXT.
2. THE LATERAL EARTH PRESSURES ARE UNFACTORED.

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SEISMIC LATERAL EARTH PRESSURE - DRAINED

PROPOSED ARTS DISTRICT CENTER
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FIGURE 6

APPENDIX A

APPENDIX A

SUBSURFACE EXPLORATIONS

We explored the subsurface conditions at the site by drilling four borings (B-1 through B-4) to depths ranging between 67.0 and 77.0 feet BGS at the locations shown on Figure 2. The borings were drilled in June 2014 by JDK Drilling, Inc. using a limited-access drill rig equipped for hollow-stem auger drilling. The exploration logs are presented in this appendix.

The locations of the explorations were determined in the field by rolling-wheel measurements from surveyed existing site features. This information should be considered accurate only to the degree implied by the methods used.

A member of our geotechnical staff observed and logged the explorations. We collected representative samples of the various soils encountered in the explorations.

SOIL SAMPLING

Samples were collected from the borings using modified California split-spoon samplers in general accordance with ASTM D 3550. The samplers were driven into the soil with a 140-pound hammer free-falling 30 inches. The samplers were driven 18 inches or to refusal as indicated on the exploration logs. The number of blows required to drive the sampler the final 12 inches (or less if refusal is met) is recorded on the exploration logs included in this appendix, unless otherwise noted. Sampling methods and intervals are shown on the exploration logs.

SOIL CLASSIFICATION

The soil samples were classified in accordance with the “Exploration Key” (Table A-1) and “Soil Classification System” (Table A-2), which are presented in this appendix. The exploration logs indicate the depths at which the soils or their characteristics change, although the change actually could be gradual. If the change occurred between sample locations, the depth was interpreted. Classifications are shown on the exploration logs.

LABORATORY TESTING

CLASSIFICATION

The soil samples were classified in the laboratory to confirm field classifications. If those classifications differed from the field classifications, the laboratory classifications are presented on the exploration logs.

MOISTURE CONTENT

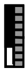
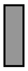
We tested the natural moisture content of selected soil samples in general accordance with ASTM D 2216. The natural moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The test results are presented in this appendix.

DRY DENSITY

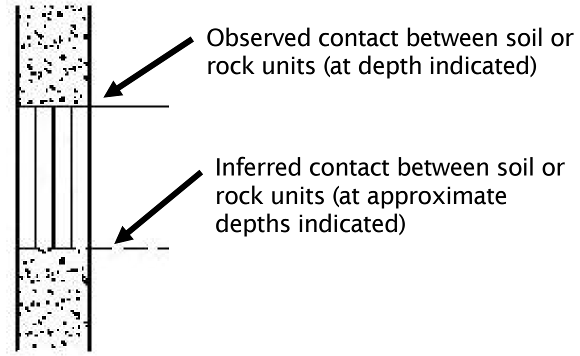
We tested selected soil samples to determine the in situ dry density. The tests were performed in general accordance with ASTM D 2937. The dry density is defined as the ratio of the dry weight of the soil sample to the volume of that sample. The dry density typically is expressed in units of pcf. The test results are presented in this appendix.

STRENGTH TESTING

Direct shear tests were completed on selected soil samples in general accordance with ASTM D 3080. The test results are presented in this appendix.

SYMBOL	SAMPLING DESCRIPTION
	Location of sample obtained in general accordance with ASTM D 1586 Standard Penetration Test with recovery
	Location of sample obtained using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D 1587 with recovery
	Location of sample obtained using Dames & Moore sampler and 300-pound hammer or pushed with recovery
	Location of sample obtained using Dames & Moore and 140-pound hammer or pushed with recovery
	Location of sample obtained using 3-inch-O.D. California split-spoon sampler and 140-pound hammer
	Location of grab sample
	Rock coring interval
▽	Water level during drilling
▼	Water level taken on date shown

Graphic Log of Soil and Rock Types



GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits	PP	Pocket Penetrometer
CBR	California Bearing Ratio	P200	Percent Passing U.S. Standard No. 200 Sieve
CON	Consolidation		
DD	Dry Density	RES	Resilient Modulus
DS	Direct Shear	SIEV	Sieve Gradation
HYD	Hydrometer Gradation	TOR	Torvane
MC	Moisture Content	UC	Unconfined Compressive Strength
MD	Moisture-Density Relationship	VS	Vane Shear
OC	Organic Content	kPa	Kilopascal
P	Pushed Sample		

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
P	Pushed Sample	NS	No Visible Sheen
PID	Photoionization Detector Headspace Analysis	SS	Slight Sheen
		MS	Moderate Sheen
ppm	Parts per Million	HS	Heavy Sheen

EXPLORATION KEY

TABLE A-1

RELATIVE DENSITY - COARSE-GRAINED SOILS

Relative Density	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)
Very Loose	0 – 4	0 - 11	0 - 4
Loose	4 – 10	11 - 26	4 - 10
Medium Dense	10 – 30	26 - 74	10 - 30
Dense	30 – 50	74 - 120	30 - 47
Very Dense	More than 50	More than 120	More than 47

CONSISTENCY - FINE-GRAINED SOILS

Consistency	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)	Unconfined Compressive Strength (tsf)
Very Soft	Less than 2	Less than 3	Less than 2	Less than 0.25
Soft	2 - 4	3 – 6	2 - 5	0.25 - 0.50
Medium Stiff	4 - 8	6 – 12	5 - 9	0.50 - 1.0
Stiff	8 - 15	12 – 25	9 - 19	1.0 - 2.0
Very Stiff	15 - 30	25 – 65	19 – 31	2.0 - 4.0
Hard	More than 30	More than 65	More than 31	More than 4.0

PRIMARY SOIL DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOILS (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVELS (< 5% fines)	GW or GP	GRAVEL
		GRAVEL WITH FINES (≥ 5% and ≤ 12% fines)	GW-GM or GP-GM	GRAVEL with silt
			GW-GC or GP-GC	GRAVEL with clay
		GRAVELS WITH FINES (> 12% fines)	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND (50% or more of coarse fraction passing No. 4 sieve)	CLEAN SANDS (<5% fines)	SW or SP	SAND
		SANDS WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SANDS WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS (50% or more passing No. 200 sieve)	SILT AND CLAY	Liquid limit less than 50	ML	SILT
			CL	CLAY
			CL-ML	silty CLAY
			OL	ORGANIC SILT or ORGANIC CLAY
		Liquid limit 50 or greater	MH	SILT
			CH	CLAY
			OH	ORGANIC SILT or ORGANIC CLAY
HIGHLY ORGANIC SOILS			PT	PEAT

MOISTURE CLASSIFICATION

Term	Field Test
dry	very low moisture, dry to touch
moist	damp, without visible moisture
wet	visible free water, usually saturated

ADDITIONAL CONSTITUENTS

Secondary granular components or other materials such as organics, man-made debris, etc.					
Percent	Silt and Clay In:		Percent	Sand and Gravel In:	
	Fine-Grained Soils	Coarse-Grained Soils		Fine-Grained Soils	Coarse-Grained Soils
< 5	trace	trace	< 5	trace	trace
5 – 12	minor	with	5 – 15	minor	minor
> 12	some	silty/clayey	15 – 30	with	with
			> 30	sandy/gravelly	Indicate %

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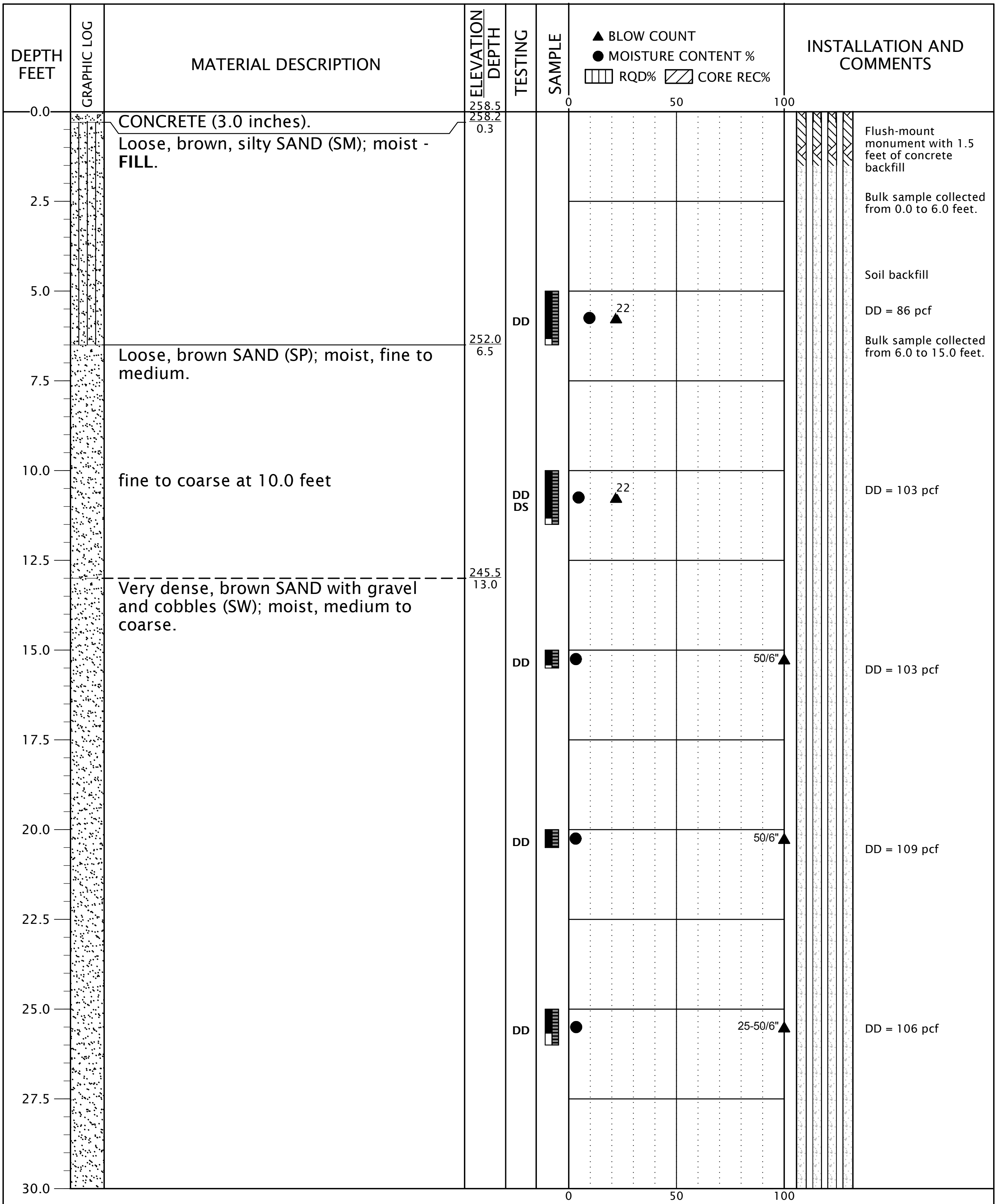
SOIL CLASSIFICATION SYSTEM

TABLE A-2



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	CONCRETE (3.0 inches).	258.5 258.2 0.3			Flush-mount monument with 1.5 feet of concrete backfill
	Loose, brown, silty SAND (SM); moist - **FILL**.				Bulk sample collected from 0.0 to 6.0 feet. Soil backfill
5.0			DD	22	DD = 86 pcf
6.5	Loose, brown SAND (SP); moist, fine to medium.	252.0 6.5			Bulk sample collected from 6.0 to 15.0 feet.
10.0	fine to coarse at 10.0 feet		DD DS	22	DD = 103 pcf
13.0	Very dense, brown SAND with gravel and cobbles (SW); moist, medium to coarse.	245.5 13.0			
15.0			DD	50/6"	DD = 103 pcf
20.0			DD	50/6"	DD = 109 pcf
25.0			DD	25-50/6"	DD = 106 pcf
30.0					

DRILLED BY: JDK Drilling, Inc.
LOGGED BY: WL
COMPLETED: 06/10/14

BORING METHOD: hollow-stem auger (see report text)
BORING BIT DIAMETER: 8-inch

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BORING B-1

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FIGURE A-1

BORING LOG ADD-1-01-B1_4.GPJ GEODESIGN.GDT PRINT DATE: 8/15/17:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
30.0	(continued from previous page)		DD DS	▥	● 15-50/4" ▲	DD = 105 pcf
32.5						
35.0	with cobbles at 35.0 feet			▥	20-50/4" ▲	
37.5						
40.0				▥	25-50/3" ▲	
42.5						
43.0	Very dense, light gray GRAVEL with sand and cobbles (GP); moist.	215.5 43.0				
45.0			DD CON	▥	● 50/6" ▲	DD = 97 pcf Bentonite chips
47.5						
50.0				▥	50/6" ▲	#3 Sand No recovery. Methane probe set at 50.0 feet.
52.5						Bentonite chips Hard drilling at 53.0 feet.
55.0				▥	50/5" ▲	#3 Sand No recovery. Methane probe set at 55.0 feet.
57.5						Bentonite chips
60.0						

DRILLED BY: JDK Drilling, Inc.

LOGGED BY: WL

COMPLETED: 06/10/14

BORING METHOD: hollow-stem auger (see report text)

BORING BIT DIAMETER: 8-inch

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BORING B-1
(continued)

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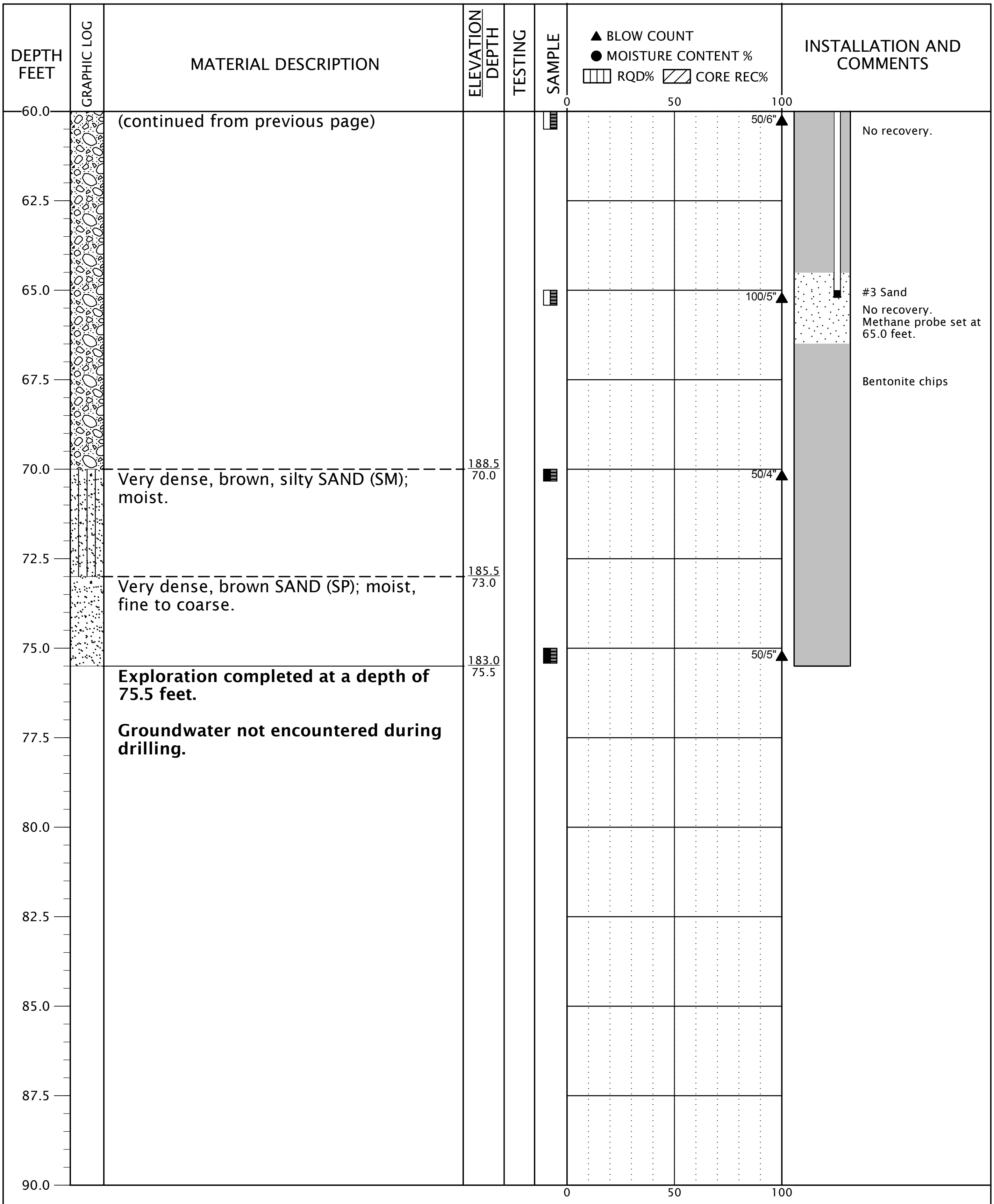
LOS ANGELES, CA

FIGURE A-1

BORING LOG ADD-1-01-B1_4.GPJ GEODESIGN.GDT PRINT DATE: 8/15/17:KT



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	BLOW COUNT	INSTALLATION AND COMMENTS
60.0	(continued from previous page)		50/6"	No recovery.
65.0			100/5"	#3 Sand No recovery. Methane probe set at 65.0 feet.
67.5				Bentonite chips
70.0	Very dense, brown, silty SAND (SM); moist.	188.5 70.0	50/4"	
73.0	Very dense, brown SAND (SP); moist, fine to coarse.	185.5 73.0		
75.0			50/5"	
75.5	**Exploration completed at a depth of 75.5 feet.** **Groundwater not encountered during drilling.**	183.0 75.5		

DRILLED BY: JDK Drilling, Inc. LOGGED BY: WL COMPLETED: 06/10/14

BORING METHOD: hollow-stem auger (see report text) BORING BIT DIAMETER: 8-inch

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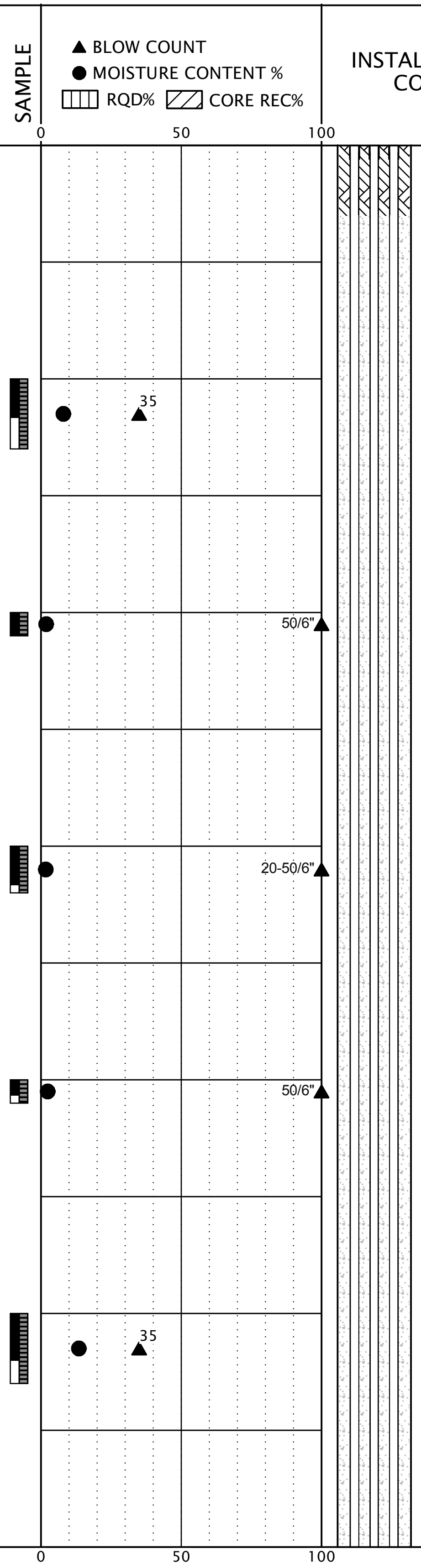
BORING B-1
(continued)

PROPOSED ARTS DISTRICT CENTER
LOS ANGELES, CA

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	CONCRETE (5.0 inches).	257.0 256.6 0.4			Flush-mount monument with 1.5 feet of concrete backfill
	Medium dense, brown, silty SAND (SM), trace construction debris; moist - **FILL**.				
2.5					
5.0			DD	35	Soil backfill DD = 81 pcf
7.5	Very dense, brown SAND with gravel and cobbles (SW); moist.	250.0 7.0			
10.0			DD	50/6"	DD = 116 pcf
12.5					
15.0			DD DS	20-50/6"	DD = 105 pcf Sheared cobble in sample barrel at 15.0 feet.
17.5					
20.0			DD	50/6"	DD = 108 pcf
22.5					
25.0	Medium dense, brown SAND with silt, gravel, and cobbles (SP-SM); moist, fine to coarse.	232.0 25.0	DD	35	DD = 92 pcf
27.5					
30.0	Very dense, brown SAND with gravel	227.5 29.5			



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BORING BIT DIAMETER: 8-inch

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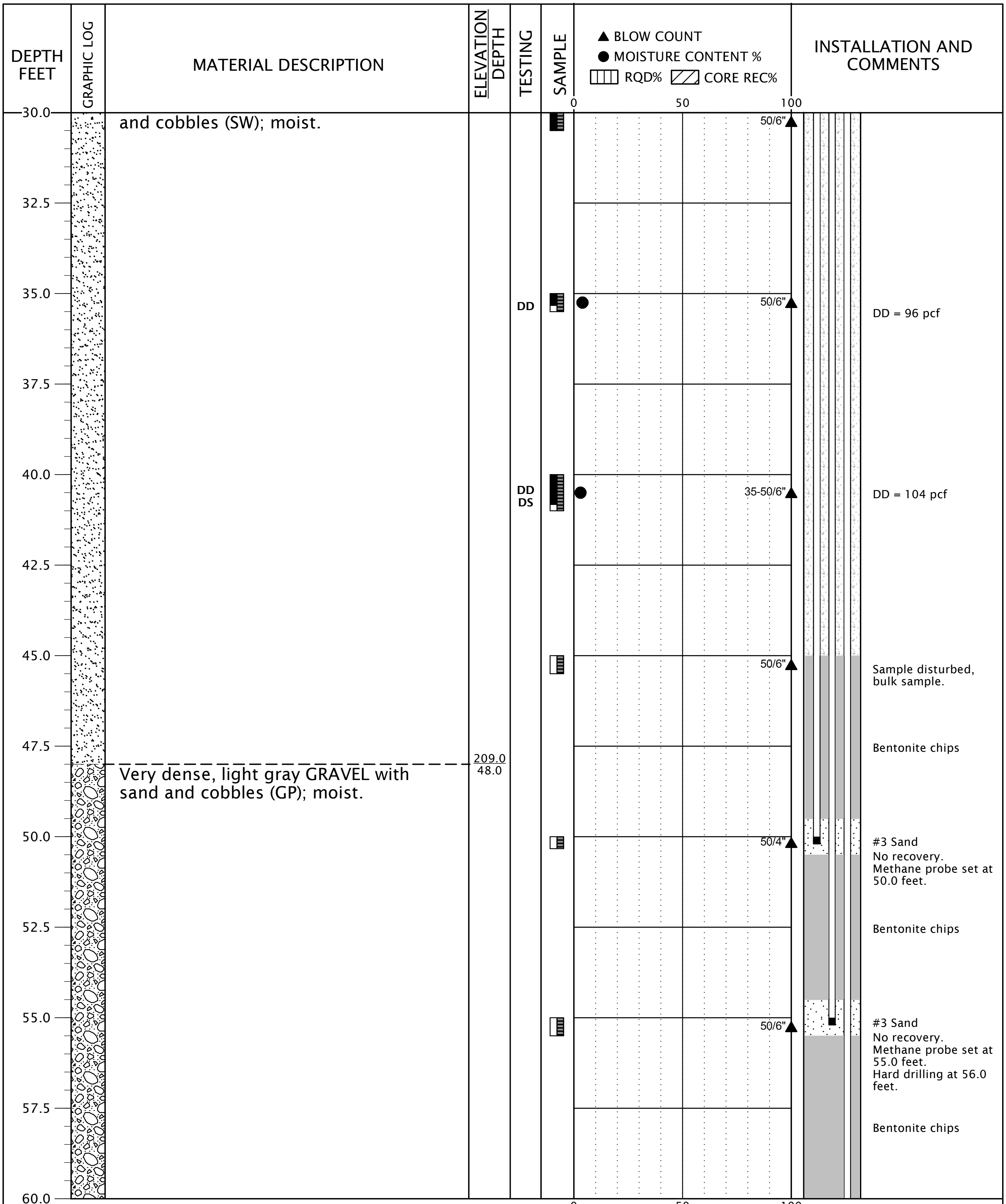
BORING B-2

PROPOSED ARTS DISTRICT CENTER
LOS ANGELES, CA

FIGURE A-2

BORING LOG ADD-1-01-B1_4.GPJ GEODESIGN.GDT PRINT DATE: 8/15/17:KT



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
30.0	and cobbles (SW); moist.			50/6"	
35.0			DD	50/6"	DD = 96 pcf
40.0			DD DS	35-50/6"	DD = 104 pcf
45.0				50/6"	Sample disturbed, bulk sample.
47.5					Bentonite chips
48.0	Very dense, light gray GRAVEL with sand and cobbles (GP); moist.	209.0 48.0			
50.0				50/4"	#3 Sand No recovery. Methane probe set at 50.0 feet.
52.5					Bentonite chips
55.0				50/6"	#3 Sand No recovery. Methane probe set at 55.0 feet. Hard drilling at 56.0 feet.
57.5					Bentonite chips
60.0					

DRILLED BY: JDK Drilling, Inc.

LOGGED BY: WL

COMPLETED: 06/10/14

BORING METHOD: hollow-stem auger (see report text)

BORING BIT DIAMETER: 8-inch

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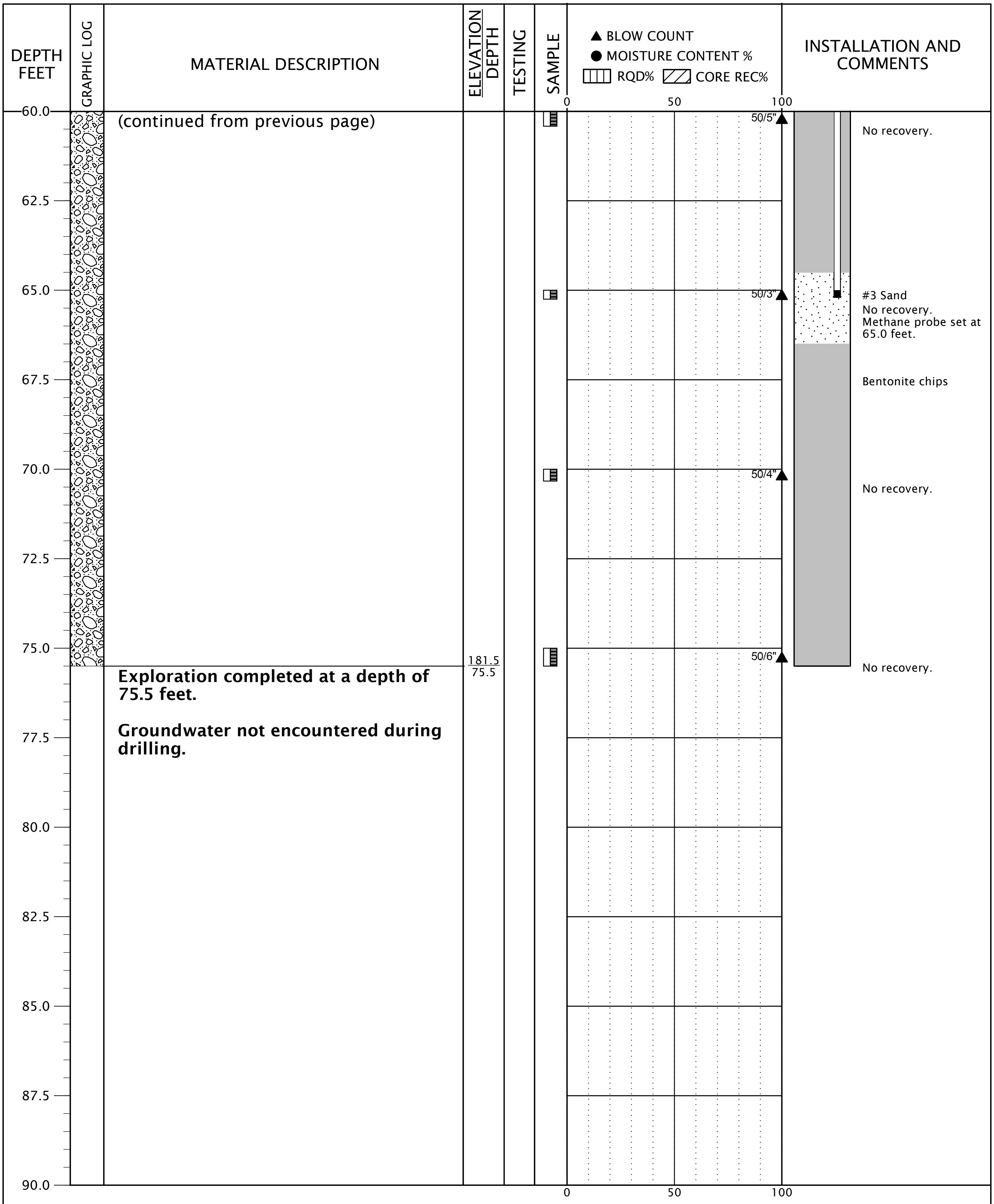
BORING B-2
(continued)

PROPOSED ARTS DISTRICT CENTER
LOS ANGELES, CA

FIGURE A-2



(continued from previous page)

Exploration completed at a depth of 75.5 feet.

Groundwater not encountered during drilling.

No recovery.

#3 Sand
No recovery.
Methane probe set at 65.0 feet.

Bentonite chips

No recovery.

No recovery.

DRILLED BY: JDK Drilling, Inc.
LOGGED BY: WL
COMPLETED: 06/10/14

BORING METHOD: hollow-stem auger (see report text)
BORING BIT DIAMETER: 8-inch

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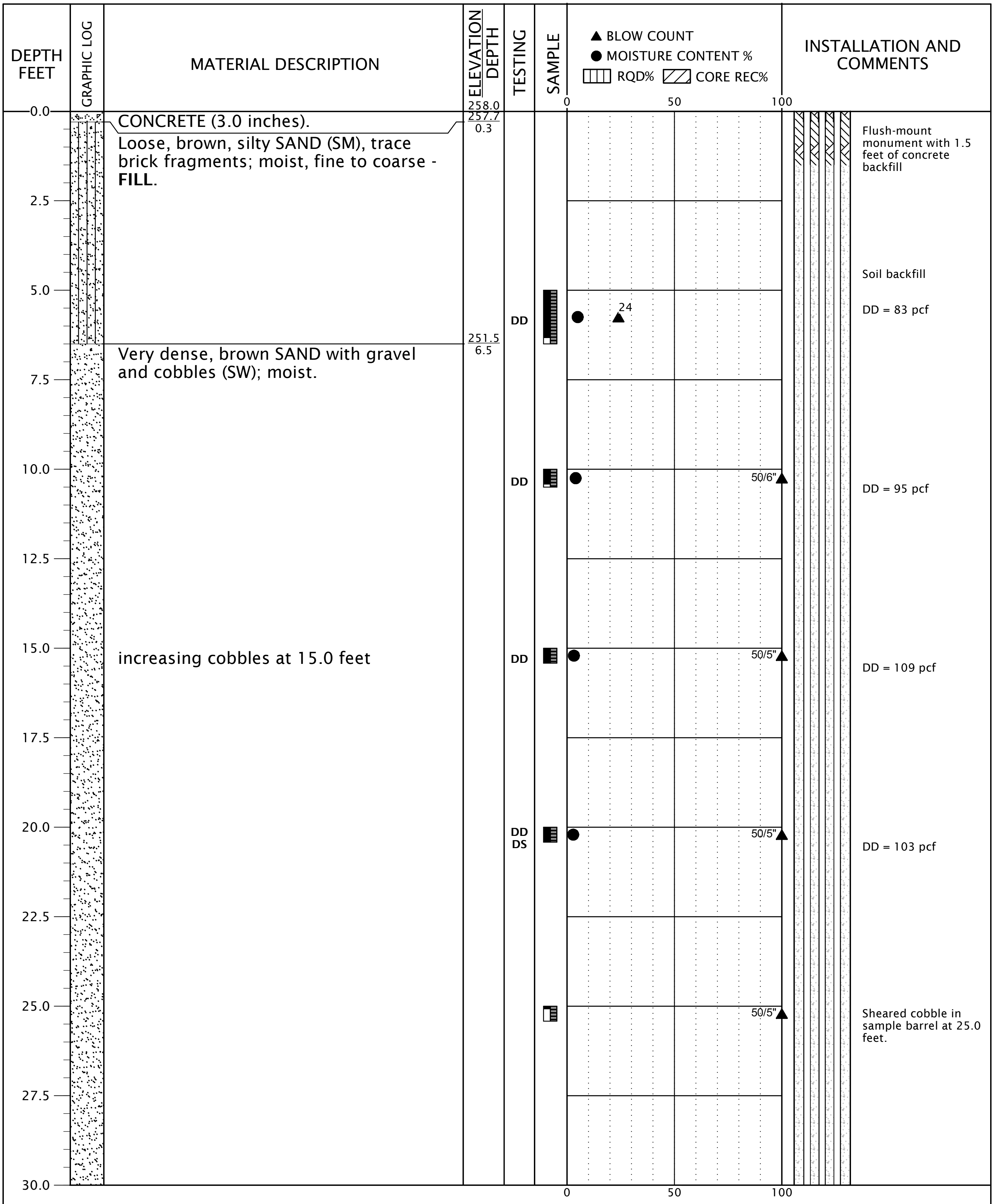
BORING B-2
(continued)

PROPOSED ARTS DISTRICT CENTER
LOS ANGELES, CA

FIGURE A-2

BORING LOG ADD-1-01-B1_4.GPJ GEODESIGN.GDT PRINT DATE: 8/15/17:KT



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	CONCRETE (3.0 inches).	258.0			
	Loose, brown, silty SAND (SM), trace brick fragments; moist, fine to coarse - **FILL**.	257.7 0.3			Flush-mount monument with 1.5 feet of concrete backfill
					Soil backfill
5.0			DD	24	DD = 83 pcf
	Very dense, brown SAND with gravel and cobbles (SW); moist.	251.5 6.5			
10.0			DD	50/6"	DD = 95 pcf
15.0	increasing cobbles at 15.0 feet		DD	50/5"	DD = 109 pcf
20.0			DD DS	50/5"	DD = 103 pcf
25.0				50/5"	Sheared cobble in sample barrel at 25.0 feet.

DRILLED BY: JDK Drilling, Inc.

LOGGED BY: WL

COMPLETED: 06/11/14

BORING METHOD: hollow-stem auger (see report text)

BORING BIT DIAMETER: 8-inch

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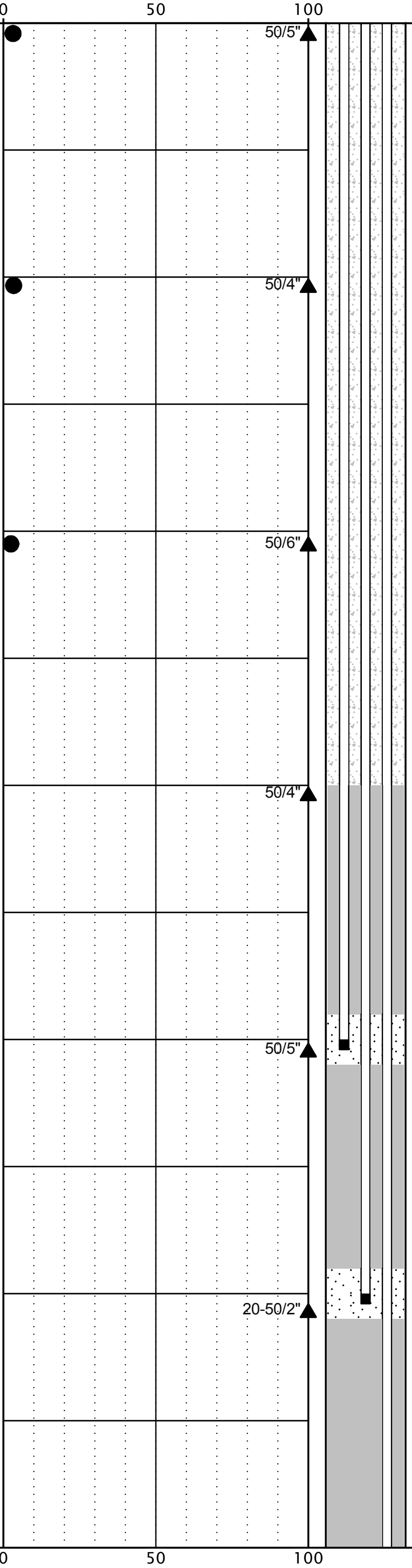
BORING B-3

PROPOSED ARTS DISTRICT CENTER
LOS ANGELES, CA

FIGURE A-3

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT	INSTALLATION AND COMMENTS
30.0	(continued from previous page)		DD	■	50/5"	DD = 113 pcf
35.0			DD DS	■	50/4"	DD = 111 pcf
40.0			DD	■	50/6"	DD = 98 pcf
45.0				■	50/4"	Bentonite chips
48.0	Very dense, light gray GRAVEL with sand and cobbles (GP); dry.	210.0 48.0				
50.0				☐	50/5"	#3 Sand Sample disturbed. Methane probe set at 50.0 feet.
52.5						Bentonite chips
55.0				☐	20-50/2"	#3 Sand No recovery. Methane probe set at 55.0 feet.
57.5						Bentonite chips
58.0	Medium dense brown SAND (SP); moist, fine.	200.0 58.0				



DRILLED BY: JDK Drilling, Inc. LOGGED BY: WL COMPLETED: 06/11/14

BORING METHOD: hollow-stem auger (see report text) BORING BIT DIAMETER: 8-inch

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BORING B-3
(continued)

PROPOSED ARTS DISTRICT CENTER
LOS ANGELES, CA

FIGURE A-3

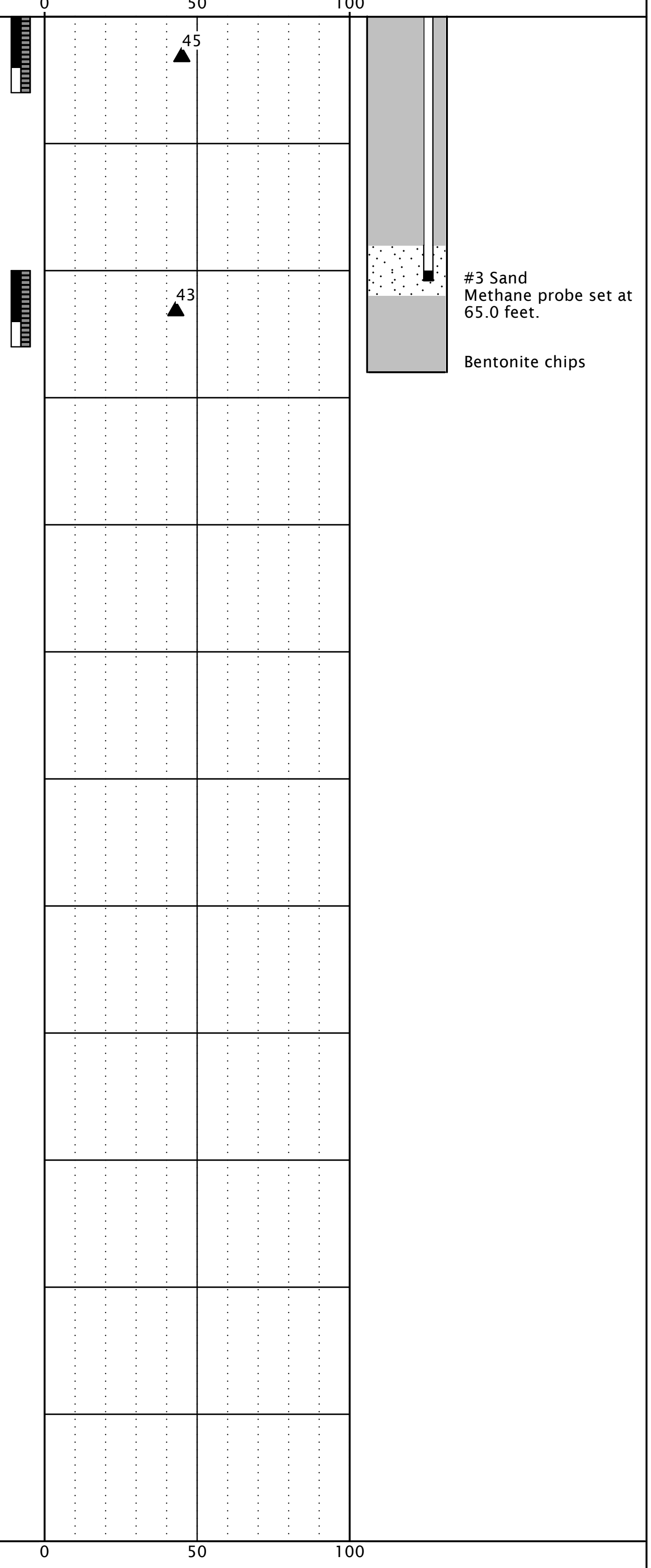
DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE
60.0		(continued from previous page)			
		Stiff, brown SILT (ML); moist.	197.0 61.0		
62.5					
65.0		sandy at 65.0 feet			
		Dense, brown SAND (SP); moist, fine to coarse.	191.6 66.4		
67.5		**Exploration completed at a depth of 67.0 feet.** **Groundwater not encountered during drilling.**	191.0 67.0		
70.0					
72.5					
75.0					
77.5					
80.0					
82.5					
85.0					
87.5					
90.0					

▲ BLOW COUNT

● MOISTURE CONTENT %

▥ RQD% ▨ CORE REC%

INSTALLATION AND COMMENTS



DRILLED BY: JDK Drilling, Inc.

LOGGED BY: WL

COMPLETED: 06/11/14

BORING METHOD: hollow-stem auger (see report text)

BORING BIT DIAMETER: 8-inch

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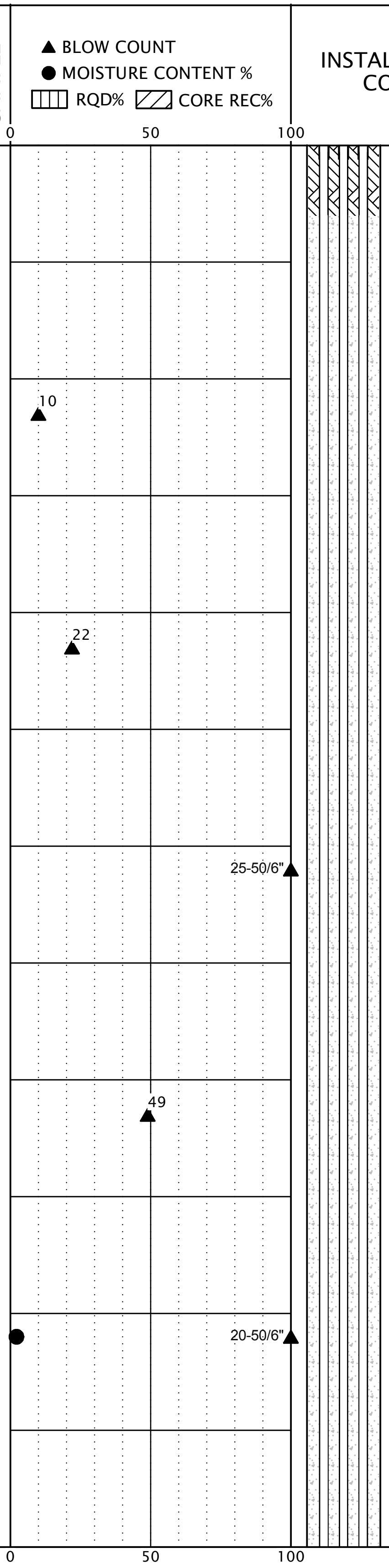
BORING B-3
(continued)

PROPOSED ARTS DISTRICT CENTER
LOS ANGELES, CA

FIGURE A-3

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT	INSTALLATION AND COMMENTS
0.0	CONCRETE (3.0 inches).	257.0				Flush-mount monument with 1.5 feet of concrete backfill
	Loose to medium dense, red to red-brown, silty SAND with brick fragments (SM); moist, fine to coarse - **FILL**.	256.7 0.3				
2.5						Bulk sample collected from 0.0 to 1.0 foot. Bulk sample collected from 1.0 foot to 6.0 feet.
5.0				▮	10	Grout Bulk sample collected from 6.0 to 15.0 feet.
10.0				▮	22	
12.0	Dense to very dense, brown SAND with gravel and cobbles (SW); moist.	245.0 12.0				
15.0				▮	25-50/6"	
20.0				▮	49	
25.0			DD DS	▮	20-50/6"	DD = 104 pcf
30.0						



DRILLED BY: JDK Drilling, Inc.

LOGGED BY: WL

COMPLETED: 06/09/14

BORING METHOD: hollow-stem auger (see report text)

BORING BIT DIAMETER: 8-inch

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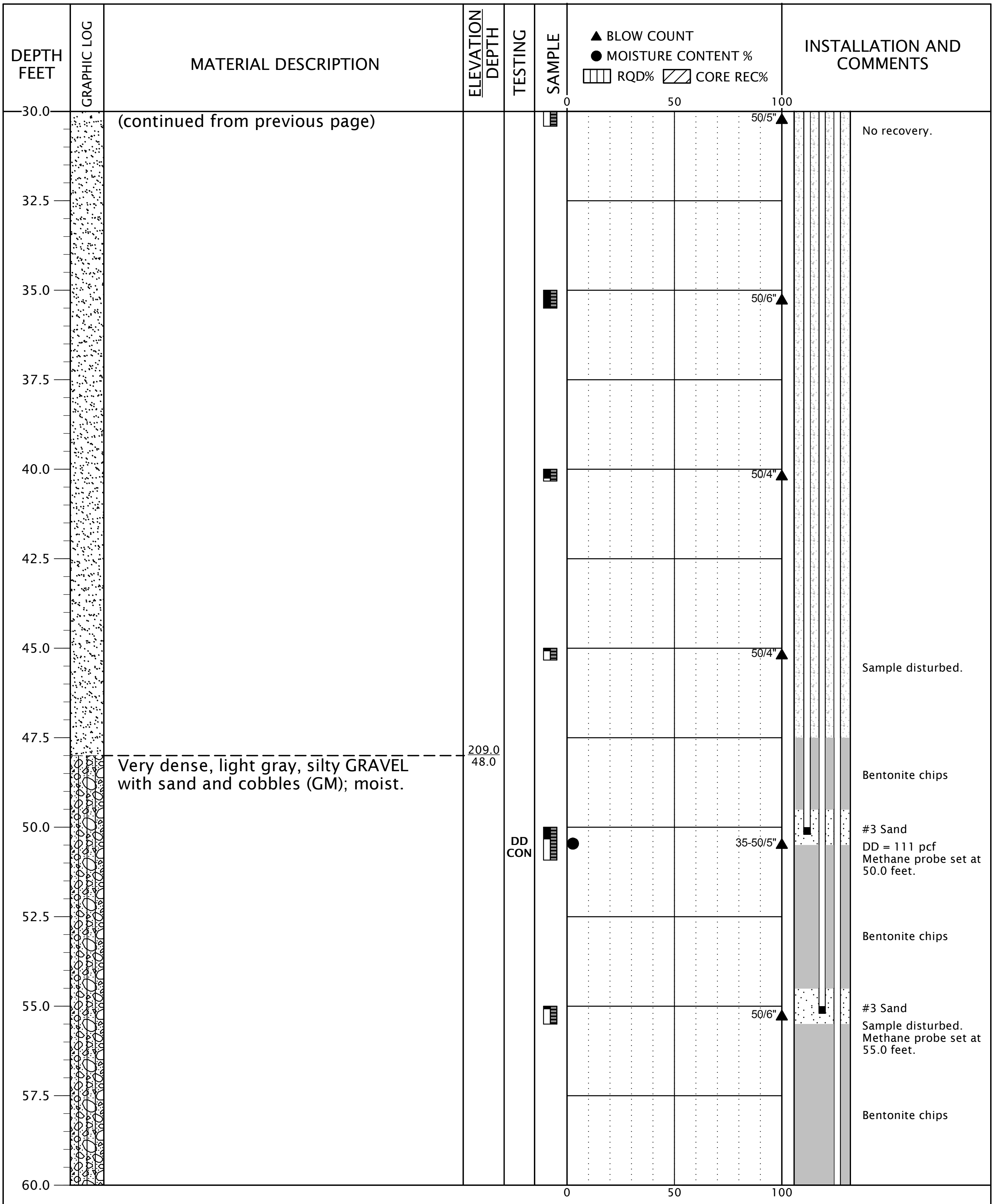
BORING B-4

PROPOSED ARTS DISTRICT CENTER
LOS ANGELES, CA

FIGURE A-4



(continued from previous page)

Very dense, light gray, silty GRAVEL with sand and cobbles (GM); moist.

No recovery.

Sample disturbed.

Bentonite chips

#3 Sand

DD = 111 pcf
Methane probe set at 50.0 feet.

Bentonite chips

#3 Sand

Sample disturbed.
Methane probe set at 55.0 feet.

Bentonite chips

DRILLED BY: JDK Drilling, Inc.
LOGGED BY: WL
COMPLETED: 06/09/14

BORING METHOD: hollow-stem auger (see report text)
BORING BIT DIAMETER: 8-inch

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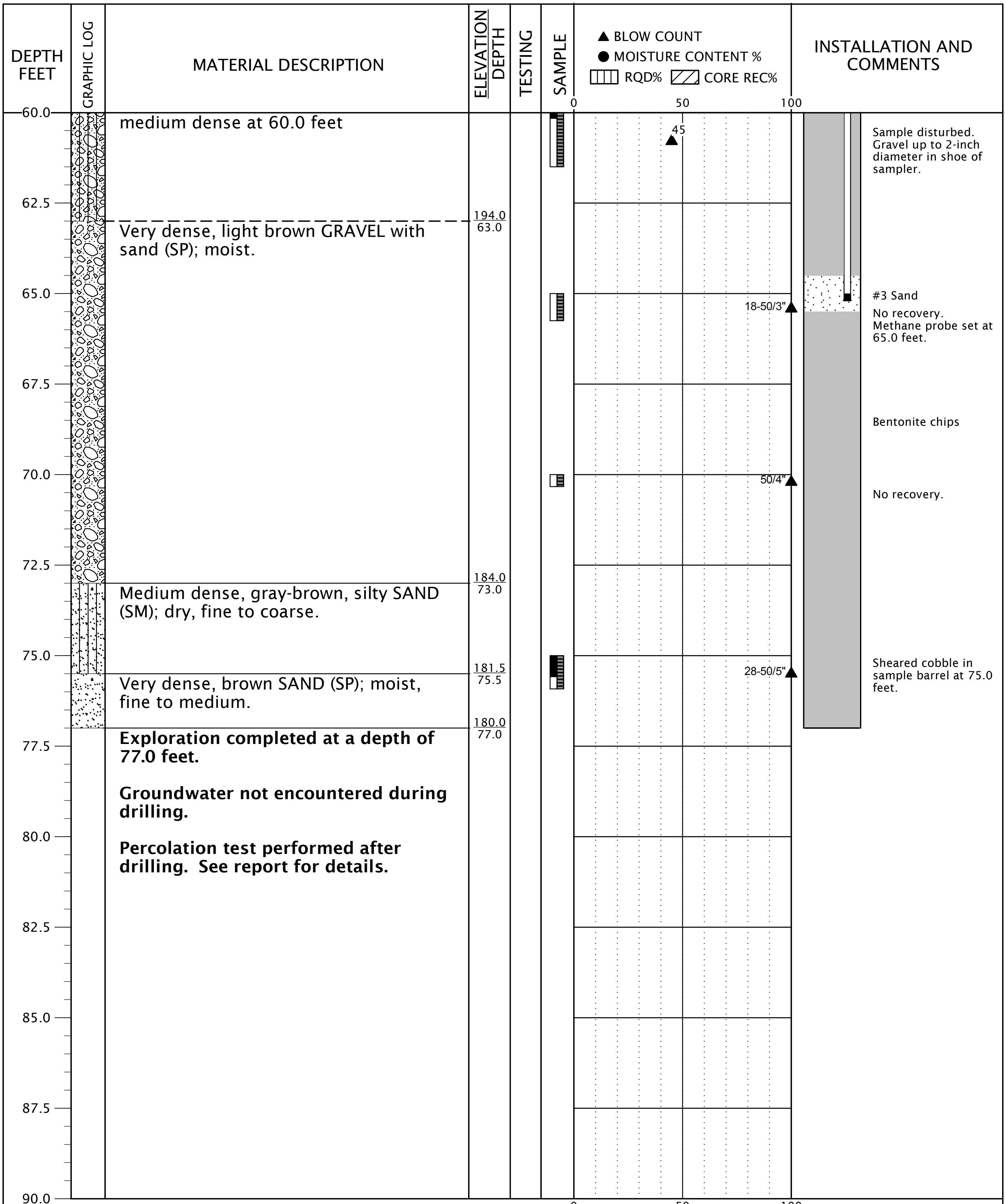
BORING B-4
(continued)

PROPOSED ARTS DISTRICT CENTER
LOS ANGELES, CA

FIGURE A-4



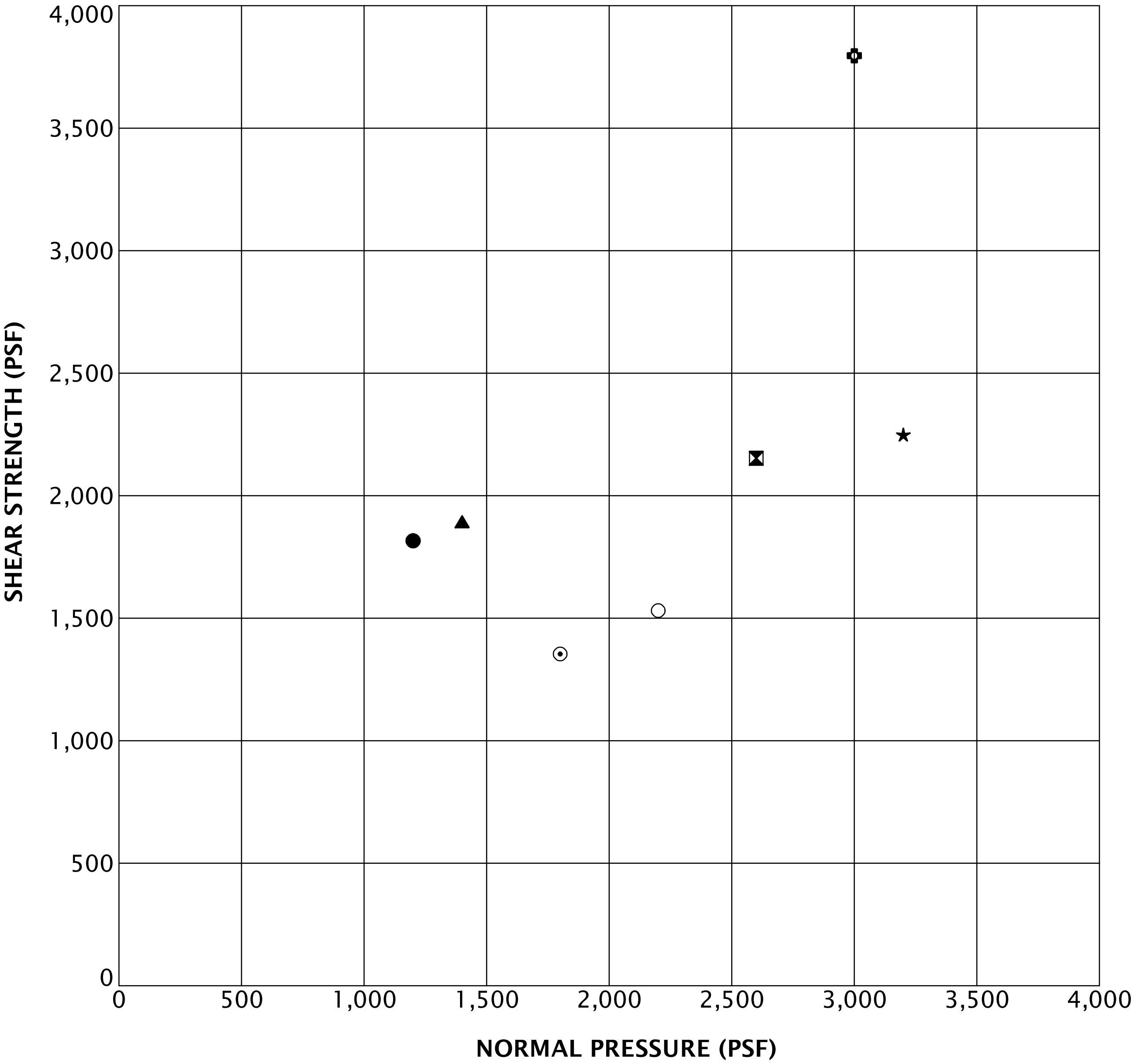
DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	BLOW COUNT	INSTALLATION AND COMMENTS
60.0	medium dense at 60.0 feet		45	Sample disturbed. Gravel up to 2-inch diameter in shoe of sampler.
63.0	Very dense, light brown GRAVEL with sand (SP); moist.	194.0 / 63.0		
65.0			18-50/3"	#3 Sand No recovery. Methane probe set at 65.0 feet.
				Bentonite chips
70.0			50/4"	No recovery.
73.0	Medium dense, gray-brown, silty SAND (SM); dry, fine to coarse.	184.0 / 73.0		
75.0			28-50/5"	Sheared cobble in sample barrel at 75.0 feet.
75.5	Very dense, brown SAND (SP); moist, fine to medium.	181.5 / 75.5		
77.0	**Exploration completed at a depth of 77.0 feet.** **Groundwater not encountered during drilling.** **Percolation test performed after drilling. See report for details.**	180.0 / 77.0		

DRILLED BY: JDK Drilling, Inc. LOGGED BY: WL COMPLETED: 06/09/14

BORING METHOD: hollow-stem auger (see report text) BORING BIT DIAMETER: 8-inch

BORING B-4
(continued)

PROPOSED ARTS DISTRICT CENTER
LOS ANGELES, CA



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SOAKED
●	B-1	10.0	6	101	YES
⧗	B-1	30.0	4	105	YES
▲	B-2	15.0	2	105	YES
★	B-2	40.0	3	104	YES
⊙	B-3	20.0	3	103	YES
✿	B-3	35.0	3	111	YES
○	B-4	25.0	2	104	YES

DIRECT_SHEAR_FAIL_ENV_NO BOX ADD-1-01-B1_4.GPJ GEODESIGN.GDT PRINT DATE: 8/15/17:KT

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DIRECT SHEAR TEST RESULTS

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LOS ANGELES, CA

FIGURE A-5

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-1	5.0	253.5	10	86						
B-1	10.0	248.5	5	103						
B-1	15.0	243.5	3	103						
B-1	20.0	238.5	3	109						
B-1	25.0	233.5	3	106						
B-1	30.0	228.5	4	105						
B-1	45.0	213.5	4	97						
B-2	5.0	252.0	8	81						
B-2	10.0	247.0	2	116						
B-2	15.0	242.0	2	105						
B-2	20.0	237.0	2	108						
B-2	25.0	232.0	13	92						
B-2	35.0	222.0	4	96						
B-2	40.0	217.0	3	104						
B-3	5.0	253.0	5	83						
B-3	10.0	248.0	4	95						
B-3	15.0	243.0	3	109						
B-3	20.0	238.0	3	103						
B-3	30.0	228.0	3	113						
B-3	35.0	223.0	3	111						
B-3	40.0	218.0	2	98						
B-4	25.0	232.0	2	104						
B-4	50.0	207.0	3	111						

LAB SUMMARY ADD-1-01-B1_4.GPJ GEODESIGN.GDT PRINT DATE: 8/15/17:KT

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SUMMARY OF LABORATORY DATA

PROPOSED ARTS DISTRICT CENTER
LOS ANGELES, CA

FIGURE A-6

APPENDIX B

PERCOLATION TEST AND CALCULATION HIGH FLOWRATE

Project Number: **ADD-1-01**
Boring Number: **B-4**
Diameter of Hole: **0.354 ft**
Hours Pre-Soak: **<1**
Date Pre-Soak Initiated: **6/9/2014**
Depth of Bottom (Below Grade): **77.00**
Name of Tester: **Wayne Liu**
Date Tested: **6/9/2014**
Method to Prevent Caving: **Auger**
Checked by: **Date:**

Status	t-intial	t-final	delta t (hours)	d-bottom (feet)	d-initial	d-final	delta d=F	D (feet)	Pre-adjusted Percolation Rate (cubic in/hr)	Area of infiltration (square in)	Adjusted Percolation Rate (in/hr)
Presoak	12:00	12:04	0.07	77.00	70.00	77.00	7.00	0.354	17857.86	1135.19	15.73
	12:15	12:19	0.07	77.00	70.00	77.00	7.00	0.354	17857.86	1135.19	15.73
Percolation Test Steady State	12:25	12:29	0.07	77.00	71.00	76.85	5.85	0.354	14924.07	951.03	15.69
	12:36	12:40	0.07	77.00	71.00	76.88	5.88	0.354	15000.60	955.83	15.69
	12:48	12:52	0.07	77.00	71.00	76.80	5.80	0.354	14796.51	943.02	15.69
	13:04	13:08	0.07	77.00	71.00	76.78	5.78	0.354	14745.49	939.82	15.69
	13:15	13:19	0.07	77.00	71.00	76.81	5.81	0.354	14822.02	944.62	15.69
										Average	15.69

ACRONYMS AND ABBREVIATIONS

ACRONYMS AND ABBREVIATIONS

ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BGS	below ground surface
CDMG	California Division of Mines and Geology
CGS	California Geological Survey
g	gravitational acceleration (32.2 feet/second^2)
H:V	horizontal to vertical
HDPE	high-density polyethylene
LABC	Los Angeles Building Code
LADBS	Los Angeles Department of Building and Safety
MCE	maximum considered earthquake
pcf	pounds per cubic foot
pci	pounds per cubic inch
psf	pounds per square foot
psi	pounds per square inch
PVC	polyvinyl chloride

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