APPENDIX C
Geotechnical Investigation

GEOTECHNICAL ENGINEERING SERVICES
AVIATION BOULEVARD AT ARTESIA BOULEVARD SOUTHBOUND TO WESTBOUND RIGHT TURN LANE IMPROVEMENT PROJECT
MANHATTAN BEACH, LOS ANGELES COUNTY, CALIFORNIA

Prepared for

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June 5, 2017

Mr. Michael J. Bruz
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Attention: Mr. Bruz

**SUBJECT: GEOTECHNICAL ENGINEERING SERVICES
AVIATION BOULEVARD AT ARTESIA BOULEVARD SOUTHBOUND TO WESTBOUND RIGHT TURN LANE IMPROVEMENT PROJECT
MANHATTAN BEACH, LOS ANGELES COUNTY, CALIFORNIA
HAI Project No. MBI-17-001**

Dear Mr. Bruz:

Hushmand Associates, Inc. (HAI) is pleased to submit the geotechnical engineering report for the *Aviation Boulevard at Artesia Boulevard Southbound to Westbound Right Turn Lane Improvement Project* located within the City of Manhattan Beach, California. This report has been prepared in accordance with the scope of work of HAI's Proposal No. P16-0817a dated August 17, 2016.

HAI appreciates the opportunity of being of service to Michael Baker International, Inc. Should you need additional information or any clarifications please call the undersigned.

Sincerely yours,

HUSHMAND ASSOCIATES, INC.

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HAI Project No. MBI-17-001

1.0 INTRODUCTION

1.1 Purpose and Scope of Services

This report presents the results of the geotechnical investigation performed by Hushmand Associates, Inc. (HAI) for the *Aviation Boulevard at Artesia Boulevard Southbound to Westbound Right Turn Lane Improvement Project* (Project) located in Manhattan Beach (City), California. The approximate project location is shown on Figure 1. This report is prepared in accordance with the scope of work of HAI's Proposal No. P16-0817a dated August 17, 2016. The scope of work comprised a geotechnical investigation including field investigation, laboratory testing, interpretation of field and laboratory test data, engineering analysis, and preparation of this report providing geotechnical engineering data and parameters for design and construction of the new retaining wall and pavement structure.

Specifically, the scope of work included the following tasks:

- Project coordination and review of existing information provided to us by Michael Baker International, Inc.
- Site reconnaissance to document the existing condition of the site, and to select and mark the proposed boring locations. Coordinate with Underground Service Alert for marking underground utility locations prior to drilling.
- Drilling and sampling within the top 16.5 feet of subsurface material to characterize onsite soil with depth.
- Laboratory testing of soil samples from the drilling program.
- Field and laboratory data compilation and engineering analyses.
- Preparation of this written report presenting our findings, conclusions and recommendations for the project.

The engineering conclusions and recommendations presented herein address the following:

- Potential seismic hazards;
- Site seismic design coefficients;
- Earthwork and compaction criteria;
- Lateral earth pressures;
- Shallow foundation design parameters;
- Pipe bedding and shading and trench zone requirements; and
- Corrosion potential of soils.

Our scope of services did not include evaluations or recommendations regarding groundwater quality, hazardous waste, asbestos or lead abatement, or demolition of existing structures, utilities, or other facilities.

1.2 Project Description and Background

The City plans to widen the west side of Aviation Boulevard and north side of Artesia Boulevard to provide a southbound Aviation to Westbound Artesia Right-turn lane. Additional right-of-way is required on both Aviation Boulevard and Artesia Boulevard. A retaining wall of various heights (maximum 7-foot high) at the northwest corner of the intersection is needed. A storm catch basin and connector pipe need to be added adjacent to the end of curb return to drain the proposed sump in the curb and gutter.

2.0 FIELD EXPLORATION

Prior to the field investigation, a site reconnaissance was performed by our staff to mark the boring location and to evaluate this location with respect to utility lines and other subsurface structures. Underground Service Alert was then notified for the proposed boring location (Ticket number A70940854). The field investigation was performed on April 7, 2017 and consisted of drilling two (2) borings (Borings B-1 and B-2) to a maximum depth of 4 feet below ground surface (bgs) in the planter area, and two (2) borings (Borings B-3 and B-4) to a maximum depth of 16.5 feet bgs on Aviation Boulevard close to the intersection with Artesia Boulevard. Boring location, tentative depth, and soil sampling intervals were specified by Michael Baker International, Inc. and the project geotechnical engineer based on the anticipated subsurface conditions.

The two (2) 4-foot deep borings (Borings B-1 and B-2) were drilled with hand auger drilling equipment. Bulk samples were retrieved from the upper four (4) feet of onsite soils at the drilling location. A Modified California (MC) ring sampler for hand auguring was used to collect relatively undisturbed samples at a depth of approximately 2.5 feet bgs.

The two (2) 16.5-foot deep borings (Borings B-3 and B-4) were drilled with an 8-inch outside diameter hollow-stem auger (HSA) on a truck-mounted drill rig. 2R Drilling from Chino, California was subcontracted to drill the borings under the field supervision of HAI personnel. In order not to interfere with any utilities at the proposed drilling location, the upper five (5) feet of onsite soils were drilled using hand auger drilling equipment. Bulk samples were retrieved from the upper five (5) feet of onsite soils at the drilling location. From five (5) feet to the maximum drilling depth, relatively undisturbed samples were recovered from the test borings. Samples were taken at about every 5 feet using a Modified California (MC) ring sampler. The MC sampler has a 2.42-inch inside diameter and a 3.0-inch outside diameter and was used to collect relatively undisturbed samples.

After the sampler was withdrawn from the boring, soil samples were carefully removed, visually inspected and classified according to the Unified Soil Classification System (USCS), sealed to reduce moisture loss, and delivered to our laboratory for further inspection, soil classification, and testing. The borings were backfilled with compacted soil cuttings upon completion of the field investigation. The pavement surface at the locations of Borings B-3 and B-4 was patched with cold asphalt concrete.

Approximate location of the exploratory borings are shown on Figure 1. Logs of exploratory borings, as well as a key to these logs, are presented in Appendix A.

3.0 LABORATORY TESTING

Soil samples collected during the field investigation were examined in our laboratory and selected samples were tested to evaluate their physical characteristics, in-situ conditions, classification, index, and engineering properties. Laboratory tests performed included:

- In-Situ Moisture Content and Dry Density (ASTM D2937);
- Particle-Size Analysis of Soils (ASTM D6913);
- Expansion Index (ASTM D4829);
- Direct Shear (ASTM D3080);
- R-Value (CTM 301); and
- Corrosion Potential (including minimum resistivity, pH, soluble sulfates and soluble chlorides tests, in accordance with ASTM G187, CTM 643, 417 and 422).

Classifications made in the field were modified as appropriate based on the laboratory test results. These modifications and the type of tests performed on the selected soil samples are reflected in boring logs in Appendix A. Laboratory test results are presented in Appendix B.

4.0 SITE CONDITIONS

4.1 Local Geology

According to the California Geological Survey (CGS) Seismic Hazard Zone Report for the Redondo Beach 7.5-Minute Quadrangle (CGS, 2006), the project site is located in a region with late Pleistocene marine terrace deposits (Qter), mapped in places as Qoa, generally consisting of silty sand with local gravels that are found throughout the Palos Verdes Peninsula. Regional geology is presented in Figure 2.

4.2 Surface and Subsurface Conditions

Table 1 summarizes the thicknesses of the existing pavement structural sections and R-value test results for Borings B-3 and B-4. The log of borings in Appendix A also provides a graphical depiction of the existing pavement section and soil type.

Table 1. Existing Pavement Structural Sections and R-value Test Results

Boring I.D.	Asphalt Concrete (inches)	Aggregate Base (inches)	R-value
B-3	6	10	68
B-4	6	6	---

According to the USCS, soils along the exploration depth at the four (4) boring locations classify as Well-graded or Poorly graded Sand (SW or SP) and Silty Sand (SM). Based on laboratory test results, the in-place moisture content of the samples collected at varies between approximately 5 and 11 percent. Similarly, the in-place dry unit weight of the soil varies between approximately 84 and 125 pounds per cubic feet (pcf).

4.3 Groundwater

Groundwater was not encountered in any of the borings. According to the groundwater map for the Redondo Beach Quadrangle (CGS, 2006), the historical highest groundwater depth at the project site is greater than 10 feet bgs. However, groundwater levels may fluctuate due to seasonal variation, nearby construction, irrigation, and numerous other anthropogenic and natural influences. The groundwater contour map is presented in Figure 3.

4.4 Expansion Potential

According to the 2016 California Building Code (CBC), if the expansion index is greater than 20, soils are considered expansive. Based on the result of one (1) Expansion Index test (ASTM D4829) performed on a soil sample collected within the upper 5 feet at the location of Boring B-4, the near-surface soils at this location have an expansion index value of 0 and are considered non-expansive.

4.5 Corrosion Potential

Two (2) samples collected within the upper 5 feet were submitted to Project X Laboratory for pH, minimum resistivity, soluble sulfates and soluble chlorides content testing. The results of the tests are summarized in Table 2. Details of the test results are presented in Appendix B.

Table 2. Results of Corrosivity Testing

Boring/ Sample No.	Depth (feet)	Chloride (mg/kg)[1]	Sulfate (mg/kg)[1]	pH	Resistivity (ohm-cm)	Estimated Corrosivity Based on Resistivity[2]	Estimated Corrosivity Based on Sulfates[3]
B-2/ Bulk 1	0-4	30	30	8.65	6,968	Mildly Corrosive	S0
B-4/ Bulk 4	1-5	39	60	7.76	16,750	Very Mildly Corrosive	S0

Notes:
1. mg/kg = milligrams per kilogram (parts per million) of dry soil.
2. The approximate relationship between soil resistivity and soil corrosivity was developed based on the findings of studies presented in ASTM STP 1013 titled "Effects of Soil Characteristics on Corrosion" (February, 1989).
3. The approximate relationship between water-soluble sulfate (SO_4) in soil (percent by weight) and soil corrosivity was developed based on the 2013 California Building Code (CBC), referring to ACI 318M-14.

The above tests were performed for screening purposes only. Our firm does not practice corrosion engineering; therefore, we recommend that a corrosion engineer be retained to evaluate the corrosion potential of the onsite soils and any impact on the proposed project structures.

5.0 SEISMIC DESIGN CONSIDERATIONS

5.1 Seismic Design Coefficients and Site Seismicity

The seismic design coefficients based on Chapter 11 of the ASCE 7-10 are provided in Table 3.

Table 3. Site Categorization and Site Coefficients

Categorization/Coefficient	Design Value*
Site Coordinates	33.87306°N 118.38157°W
Site Soil Classification	S_D
Short Period Spectral Acceleration S_S (g)	1.610
1-sec. Period Spectral Acceleration S_1 (g)	0.606
Short Period (MCE_R) Spectral Acceleration S_{MS} (g)	1.610
1-sec. Period (MCE_R) Spectral Acceleration S_{M1} (g)	0.909
Short Period Design Spectral Acceleration S_{DS} (g)	1.073
1-sec. Period Design Spectral Acceleration S_{D1} (g)	0.606

Note: MCE_R stands for Risk-Targeted Maximum Considered Earthquake.
* Values obtained from *USGS U.S. Seismic Design Maps* tool, based on 2010 ASCE 7 Standard with March 2013 errata,
http://earthquake.usgs.gov/designmaps/us/application.php?

Based on the number of blow counts measured in our borings and the results of laboratory tests, the site classification was assumed as Site Class S_D. The Mapped Peak Ground Acceleration (PGA_M) adjusted for site effects at the sites was calculated to be 0.612g.

The Compton and Palos Verdes Fault Zones are located about 6.5 km south of the site. Other active faults within the project area include the Redondo Canyon alt 1 Fault Zone, located at approximately 6.9 km to the southwest of the project site, and the North Los Angeles Basin Section of Newport Inglewood Fault Zone, located approximately 9.5 km to the northeast of the project site.

5.3 Fault Rupture Hazards

Primary ground rupture is ground deformation that occurs along the surface trace of an active fault during an earthquake. CGS defines an active fault as one that has experienced surface rupture within the last approximately 11,000 years (Holocene time). According to CGS Special Publication 42 (1997a), the proposed improvements are not located within an Alquist-Priolo Earthquake Fault Zone. The site fault activity map is presented on Figure 4. No known surface expression of active faults is believed to exist within the site. Based on the above mentioned reference, the potential for a fault rupture through the site is considered low.

5.4 Liquefaction

Soil liquefaction results in loss of strength during cyclic loading, such as imposed by earthquakes. Soils most susceptible to liquefaction are loose to moderately dense, saturated granular soils with poor drainage, such as silty sands or sands and gravels capped by or containing seams of impermeable sediment or non-plastic fine-grained soils. When seismic ground shaking occurs, the cohesionless soil is subjected to cyclic shear stresses that can cause increased pore water pressure that induces liquefaction. Liquefaction can cause softening and large cyclic deformations. In loose granular soils, softening can also be accompanied by a loss of shear strength that may lead to large shear deformations or even flow failure under moderate to high shear stresses, such as beneath a foundation or sloping ground (NCEER/NSF, 1998). Loose granular soil can also settle (densify) during liquefaction and as pore pressures dissipate following an earthquake.

According to the CGS Seismic Hazard Map for the Redondo Beach 2006 (CGS, 2006), the project site is not located within liquefaction-prone areas (see Figure 5). Therefore, a liquefaction potential analysis in accordance with the "Guidelines for Evaluating and Mitigating Seismic Hazards in California" Special Publication 117A of the CGS (formerly the Division of Mines and Geology) (CGS, 2008) was not performed for this project.

5.5 Landslides

The subject of landslides is a widely encompassing subject and cannot be fully covered in a brief summary; however, landslides are downslope motions of conglomerations of earth materials or bedrock or combinations of both. Landslides are a more defined unit and are similar to slumps, but are on a larger scale. They can move in a translational movement or rotational settlement or motion. It occurs because of the loss of ability of earth materials to maintain their integrity at a specific gradient and settle or deform into a lesser gradient or position of greater equilibrium. The internal strength of the material is lost and the material settles into a form where the mass is centralized on the downhill side of motion. Landslides are usually associated with water increasing the unit weight and decreasing the internal strength of the materials. The chances of a landslide occurring are increased by increases in slope gradient, looseness of materials, unfavorable bedding (out of slope), clay content of the bedrock, underground springs, unfavorable slope orientation with existing fault boundaries, human disturbance of the landslide or its boundaries, rise of groundwater, earthquake forces helping to mobilize the mass, looseness of in-situ materials, increases in water content, and disturbance of the lateral confining forces and/or the toe of a slope.

According to the CGS Seismic Hazard Map for the Redondo Beach Quadrangle (CGS, 2006), the project site is not located in earthquake-induced landslide areas (see Figure 5) and landslide is not expected to be an issue.

6.0 CONCLUSIONS AND RECOMMENDATIONS

The discussions and recommendations presented in the following sections are based on our understanding of the proposed project requirements, the results of our geotechnical investigation, and our professional judgment.

It is our opinion that based on the above-cited geotechnical findings, the site is suitable for construction of the proposed improvements, provided that the recommendations in this report are followed, and onsite construction observations and field testing are performed.

6.1 Site Preparation and Shoring Requirements

Prior to construction, the site should be cleared of all above-ground obstacles and structures. Existing utility and irrigation lines should be protected in-place, rerouted, or removed if they interfere with the proposed construction. The resulting cavities from removal of utility lines should be properly backfilled and compacted under the supervision of the project geotechnical engineer. Vegetation, debris, and organic matter should not be incorporated into the structural fill.

Excavations deeper than 4 feet should be either laid back or shored according to appropriate jurisdiction guidelines before personnel are allowed to enter. In addition, special care should be taken for excavations near existing improvements to ensure that their integrity is not impacted. For pipes where the pipe invert depths are greater than 4 feet, wherever there is an apparent physical restriction on the lateral land for trench layback, shoring is recommended. This also applies to shallow excavations where the depth of the excavation is equal or less than 4 feet.

Trenching should be performed in short (less than 200 feet) sections. Each 200-foot section should be properly shored prior to excavation of the next segment. Long-term storage of excess earth material or equipment along the top of the excavation should be avoided within a 1H: 1V (horizontal: vertical) projection from the base of the trench.

Typical cantilever shoring should be designed based on an active fluid pressure of 43H psf. If excavations are braced at the top and at specific design intervals, the shoring pressure may then be approximated by a rectangular soil pressure distribution with a pressure per foot of width equal to 33H (psf), where H is equal to the depth of the excavation being shored.

The project geotechnical engineer should review the contractor's shoring design prior to implementation. In addition to the abovementioned pressures, the shoring system must be designed to resist horizontal pressures that may be generated by surcharge loads applied at the ground surface such as from uniform loads or vehicle loads. The edges of all excavations should be kept away from the property line a minimum horizontal distance equal to 2H (where H is the height of the excavation) or as set forth by local ordinances, whichever is more strict.

6.2 Lateral Earth Pressure for Retaining Structures

Based on laboratory test results and encountered soil conditions, the recommended lateral earth pressures are shown below:

Table 4. Recommended Lateral Earth Pressures

Design Parameter	Design Value
Active Pressure (P_a)	0.36q + 43H
At-Rest Pressure (P_o)	0.53q + 64H
Passive Pressure (P_p)	270H (maximum 2,700 psf)
Seismic Force (F_e)	$F_e = 16H^2$ (for cantilever walls, expect some deformations)
Coefficient of Friction (μ)	0.34

Notes:
1. All Design Values were calculated based on zero cohesion, an internal friction angle of 28°, and a unit weight of 120 pcf. The coefficient of friction was calculated with a safety factor of 1.5.
2. All values of height (H) in feet (ft), pressure (P) and surcharge (q) in pounds per square feet (psf) and force (F) in pounds (lb) are for unit width of walls.
3. The above pressure values apply to horizontal backfill and do not include hydrostatic pressures that might be caused by groundwater or water trapped behind the structure.
4. For 2:1 and 3:1 slopes above the wall, increase P_a and P_o by 80 and 35 percent, respectively.
5. μ is the friction coefficient applied to dead normal (buoyant) loads. F_e is in addition to the active and at-rest pressures, P_a and P_o.
6. For passive pressure use a factor of safety of 2.5 if wall rotation (D/H) is smaller than 0.04. The passive pressure might not be used if soil is subjected to scour.
7. Neglect the upper 1 foot for passive pressure unless the surface is contained by pavement or a slab.
8. The earthquake load (F_e) should be distributed as a triangle along the wall height.
9. In addition to the abovementioned pressures, retaining walls must be designed to resist horizontal pressures that may be generated by surcharge loads applied at the ground surface such as from uniform loads or vehicle loads.
10. An equivalent mapped ground acceleration of 0.31g (½ of PGA_M) was used to calculate Fe for the Design Earthquake. The earthquake load (Fe) may be distributed as an inverted triangular along the wall height for restrained or cantilever conditions.

An efficient drainage system should be provided behind retaining walls, which should consist of a curtain of free-draining material, such as Caltrans permeable Class 2 Aggregate. This drain curtain should be a minimum of 2 feet wide and extend from the bottom of the wall to within 1.5 feet of finish grade. Additionally, drainage geocomposite (Miradrain or equivalent) should be used to wrap the gravel material. The upper 1.5 feet should be a select material of low permeability (clayey soil) to minimize infiltration. A perforated pipe should be placed along the base of the wall and should be sloped at least two percent to drain water by gravity to a suitable discharge facility.

6.3 Foundations

Relatively light structures could be supported on continuous or spread footings bearing on 3 feet of compacted clean "granular" soils (soils having less than 20 percent passing standard sieve #200, free of debris, vegetation, and with rocks less than 6 inches in diameter with no more than 15 percent greater than 3 inches in diameter, confirmed with laboratory testing prior to construction) and extending to a zone of 3 feet beyond the edge of the footings, compacted in 8-inch-thick lifts (measured in loose state) to a minimum of 90 percent relative compaction per ASTM D1557 (Modified Proctor, latest edition).

Prior to placing any clean "granular" soil, the upper 18 inches of the excavation bottom should be scarified, moisture-conditioned to approximately 2 percent above the optimum moisture content, and recompacted to at least 90 percent of the maximum dry density per ASTM D1557 (Modified Proctor Test, latest edition). Based on laboratory test results, an allowable bearing capacity of 2,500 pounds per square foot may be used for design of 24-inch wide continuous footings embedded a minimum of 18 inches below adjacent level ground. This value may be increased by 250 and 500 pounds per square foot for every additional foot of width or depth increase, respectively, to a maximum of 3,500 pounds per square foot.

A lateral passive soil resistance on footing walls embedded in compacted engineered fill of 270 psf per foot of depth below the lowest adjacent finished grade, to a maximum of 2,700 psf, may be used for design. This lateral passive resistance may be combined with a lateral base friction resistance. A base friction coefficient of 0.34 may be used. The coefficient of friction should be multiplied by the dead load to obtain the lateral base friction resistance.

Where footings are adjacent to below-grade walls or underground utilities, the footings should extend below a 45-degree plane projected upward from the backside of the wall footing or bottom of the underground utility. Structural loads were not available at the time of our investigation. We should be retained to review the final foundation plans and structural loads for soil settlement estimation.

6.4 Pavement Design

Based on R-value test results from representative subgrade soil samples collected during the field investigation, Table 5 presents our recommendations for minimum pavement structural sections for traffic index (TI) values from 6 to 12.

Table 5. Recommended Pavement Section
(Design R-value = 60)

Alternative Pavement Sections	Traffic Index (TI)						
	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Hot-Mix Asphalt (HMA) over Aggregate Base (AB)*	4.0" 4.5"	4.5" 4.5"	5.5" 4.5"	6.0" 4.5"	7.0" 4.5"	7.5" 4.5"	8.0" 4.5"
Full Depth Hot-Mix Asphalt (HMA)	4.5''	5.5''	6.5''	7.5"	8.5"	9.5"	10.0"

* Caltrans Class 2 aggregate base; minimum R-Value equal to 78.

The recommendations provided below are considered general and should be complemented with latest editions of Caltrans' Highway Design Manual (HDM) and the Standard Specifications of Public Work Construction "Greenbook", including all subsequent amendments, supplements and additions. In case of a conflict, the most stringent recommendations should prevail.

Subgrade Preparation: Upon removal of the pavement, the subgrade surface should be observed by the geotechnical engineer to verify that suitable competent bearing soil is exposed. The top 12 inches of the subgrade soil should then be scarified, brought to 2 to 3 percent above optimum moisture content and re-compacted to a minimum of 90 percent relative compaction in accordance with ASTM D1557, latest version. Unsuitable materials encountered during grading should be removed to the satisfaction of the geotechnical engineer and replaced with aggregate base or other materials approved by the geotechnical engineer.

Aggregate Base (AB): In all cases, AB should be in accordance with Class 2 AB per Caltrans' Standard Specifications (latest edition). AB material should be compacted in 6-inch thick lifts to a minimum of 95 percent relative compaction per ASTM D1557, latest edition.

Hot Mix Asphalt (HMA): HMA should be compacted to a minimum of 95 percent of the HMA density determined by ASTM D1561 test procedure (latest version) in lifts not exceeding 3 inches. A "tack coat" should be applied between HMA layers.

Special recommendations:

1. Subgrade, AB, and HMA for future curbs, gutters, and sidewalks should also be worked following the above mentioned recommendations.

2. In all cases we recommend designing an efficient drainage system by the Civil Engineer of the project in order to avoid the adverse effects of surficial water penetrating into the pavement structural section. It is important to highlight that poor drainage is by far the most detrimental geotechnical factor for asphalt concrete pavement roads because it adversely contributes to degrading the stiffness and strength of the unbound (non-bituminous) materials the structural section is comprised of.

6.5 Pipelines

6.5.1 Pipe Zone Bedding and Shading Backfill

Pipe bedding should extend to a depth of at least 6 inches or pipe manufacturer's recommendation below the pipeline invert and the shading should extend from the top of the bedding to a height of at least 12 inches over the top of the pipe or pipe manufacturer's recommendation. In addition, there should be a minimum range of 6 to 8 inches of pipe zone backfill material on either side of the pipe or pipe manufacturer's recommendation.

The bedding and shading material may consist of compacted, free draining sand, gravel, or crushed rock, having a sand equivalent of not less than 30, and meeting the gradation and compaction requirements of the Greenbook, latest edition, or pipe manufacturer's recommendation. If open grade rock or crushed slag base is used around the pipe and within any portion of trench backfill, it should be separated from surrounding finer-grained material by installation of a geo-filter fabric. Properties of the pipe zone bedding and shading material should be confirmed with laboratory testing prior to construction. Onsite soils are not recommended to be used as pipe zone bedding or shading backfill.

The bedding material should be compacted to a minimum relative compaction of 90 percent per ASTM D1557 (Modified Proctor). Backfilling should be carried on simultaneously on each side of the pipe to ensure proper protection of the pipe. The bedding layer should be supported on firm, competent material, as determined by the project geotechnical engineer. Disturbed, loose/soft materials at the excavation bottom should be removed to expose firm native material per the project geotechnical engineer recommendations. If firm material is not encountered, the upper 1 foot of the onsite soils below the pipe bedding should be scarified, moisture-conditioned to approximately 2 percent above the optimum moisture content, and recompacted to at least 90 percent of the maximum dry density per ASTM D1557 (Modified Proctor). If compaction of the native soils below the bedding material is not feasible at any location, a 12-inch thick layer of crushed rock wrapped in geofabric should be placed below the pipe bedding. Questionable areas should be reviewed individually by the project geotechnical engineer to evaluate and recommend corrective measures, as necessary.

Placement of bedding and shading backfill should be observed by the project geotechnical engineer or his representative in the field and tested for compliance with the recommended relative compaction and moisture conditions.

Field density testing should conform to ASTM D6938 (Nuclear Method) and D1556 (Sand Cone Method), latest editions. Tests should be taken at a minimum of every 2 vertical feet of fill placed and every 200 feet of length, or at a frequency otherwise specified by the local regulations, whichever is stricter. Actual test intervals may vary with field conditions. Backfill found not to be in conformance should be removed or recompacted as recommended by the project geotechnical engineer.

Densification by water jetting within the pipe bedding and shading zone is not recommended. During removal of the shoring system, gaps should be filled and compacted. Pipes that are deeper than 5 feet should be able to handle stresses due to moving traffic. Casing of the pipeline might be necessary if pipes are placed shallower than 5 feet.

6.5.2 Trench Zone

Non expansive clean "granular" soils (confirmed with laboratory testing prior to construction) may be used as compacted structural fill, provided they are free of organic material, construction debris, and not containing rocks greater than 6 inches in diameter, with no more than 15 percent rocks greater than 3 inches in diameter. Clean "granular" soils should be placed in thin, loose lifts not more than 8 inches in thickness, moisture-conditioned to approximately 2 percent above the optimum moisture content and compacted to at least 90 percent of the maximum dry density per ASTM D1557 (Modified Proctor). For pavement areas, the upper 12 inches of the trench zone should be moisture-conditioned to approximately 2 percent above the optimum moisture content and compacted to at least 95 percent of the maximum dry density per ASTM D1557 (Modified Proctor). Onsite soils meeting these requirements may be used as trench zone backfill.

Placement of backfill should be observed by the project geotechnical engineer or his representative in the field and tested for compliance with the recommended relative compaction and moisture conditions.

Field density testing should conform to ASTM D6938 (Nuclear Method) and D1556 (Sand Cone Method). Tests should be taken at a minimum of every 2 vertical feet of fill placed and every 200 feet of length, or at a frequency otherwise specified by the local regulations, whichever is stricter. Actual test intervals may vary with field conditions.

Backfill found not to be in conformance should be removed or recompacted as recommended by the project geotechnical engineer. Densification by water jetting within the trench zone is not recommended. During removal of the shoring system, gaps should be filled and compacted.

6.6 Existing Utilities

The proposed improvement locaitons located near to and/or cross several existing utilities. The contractor should exercise care not to disturb these utilities and to support them during construction. Compacting backfill above the pipe zone could be detrimental to surrounding utilities; we recommend a weak slurry mix (minimum compressive strength of 100 pound per square inch [psi]) to be used for the backfilling operations wherever soil compaction is not feasible. These areas should be limited to zones between two pipes and not exceeding 2 feet on either side of the crossing.

6.7 Site Drainage

The site should be graded to provide adequate drainage away from building foundations and to prevent ponding on pavements in accordance with guidelines established by the City, Greenbook (latest edition), and the 2016 CBC. Special surface drainage features should be incorporated to drain surface sheet flow of water from retaining walls and intercept sheet flow over the paved areas.

6.8 Construction Observations and Field Testing

Construction observations and field testing should be performed by representatives of a qualified geotechnical engineer to confirm that the conditions and assumptions described in this report are the best representation of the actual conditions.

At a minimum, we recommend that the geotechnical engineer and/or his representative be present to observe and test during the following construction activities:

- Excavation, site grading of cuts and fills;
- Placement of all backfill;
- Backfilling of utility trenches and pits; and
- When any unusual conditions are encountered during grading.

Onsite observation and field testing will be a key component to a suitable geotechnical design for this project. A final report of grading should be submitted to the City.

7.0 ADDITIONAL SERVICES AND LIMITATIONS

7.1 Additional Services

If considerable modifications to the concepts included herein are implemented over the course of the design, specific geotechnical consultation and input will be required. Accordingly, we recommend that HAI be retained to provide such consultation during site preparation and grading on an as-needed basis. As a minimum HAI should be retained to review the grading and design plans prior to their issuance for conformance and compatibility with the recommendations presented in this report.

7.2 Limitations

This report has been prepared for the sole use of Michael Baker International, Inc., specifically for design of this project in Manhattan Beach, California. The opinions presented in this report have been formulated in accordance with existing accepted geotechnical engineering practices in the southern California at the time this report was written. No other warranty, expressed or implied, is made or should be inferred.

The opinions, conclusions and recommendations contained in this report are based upon the information obtained from our investigation, which includes data from widely separated discrete sampling locations, visual observations from our site reconnaissance, along with local experience and engineering judgment. The recommendations presented in this report are based on the assumption that soil and geologic conditions at or between borings do not deviate substantially from those encountered during our investigation. We are not responsible for the data presented by others. We should be retained to review the geotechnical aspects of the final plans and specifications for conformance with our recommendations. The recommendations provided in this report are based on the assumption that we will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications.

If we are not retained for these services, HAI cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of HAI's report by others. Furthermore, HAI will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services and/or at the time another consultant is retained for follow up service to this report.

The opinions presented in this report are valid as of the present date for the site evaluated. Changes in the condition of the site will likely occur with the passage of time due to natural processes and/or the works of man. In addition, changes in applicable standards of practice can occur as a result of legislation and/or the broadening of knowledge. Furthermore, geotechnical issues may arise that were not apparent at the time of our investigation. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any other properties.

8.0 REFERENCES

American Concrete Institute (ACI), 2011, Building Code Requirements for Strucftural Concrete (ACI 318-11) and Commentary.

American Society for Testing and Materials (ASTM) Standards, latest edition.

ASTM STP 1013 titled "Effects of Soil Characteristics on Corrosion" (February, 1989).

California Department of Transportation (Caltrans)'s ARS Online Tool, version 2.3.06.

Caltrans' Highway Design Manual (HDM), latest edition.

Caltrans' Standard Specifications, latest edition.

California Building Standards Commission and International Conference of Building Officials, 2016, 2016 California Building Code.

California Geological Survey (CGS), 2010, Fault Activity Map and Geologic Map of California.

CGS, 1998, Seismic Hazard Zone Report 010 for the Yorba Linda 7.5-Minute Quadrangle.

"Greenbook" Standard Specifications for Public Works Construction, latest edition.

Hart, E.W., and Bryant, W.A., 1999, Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zones Maps, California Division of Mines and Geology Special Publication 42, 38 p.

Ishihara, K., and Yoshimine, M., 1992, Evaluation of Settlements in Sand Deposits Following Liquefaction during Earthquakes, Soils and Foundations, 32 (1): 173-188.

O'Rourke, M.J. and Liu, X., 1999, Response of Buried Pipelines Subject to Earthquake Effects, Published by Multidisciplinary Center for Earthquake Engineering Research, Monograph Series No. 3.

O'Rourke, T.D., Grigoriu, M.D., and Khater, M.M., 1985, A State of the Art Review: Seismic Response of Buried Pipelines, in C. Sundararajan (editor), Decade of Progress in Pressure Vessel Technology: ASME.

Pradel, D., 1998, Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils: Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 124, No. 4, pp. 364-368.

Tokimatsu, K. and Seed, H.B., 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking: Journal of Geotechnical Engineering, Volume 113, No. 8, pp. 861-878.

United States Geological Survey (USGS) U.S. Seismic Design Maps tool, based on 2010 ASCE 7 Standard and 2012 International Building Code (IBC).

Youd, T.L., Idriss, I. M., et al., 2001, "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol 127, No. 10, October, 2001.

FIGURES

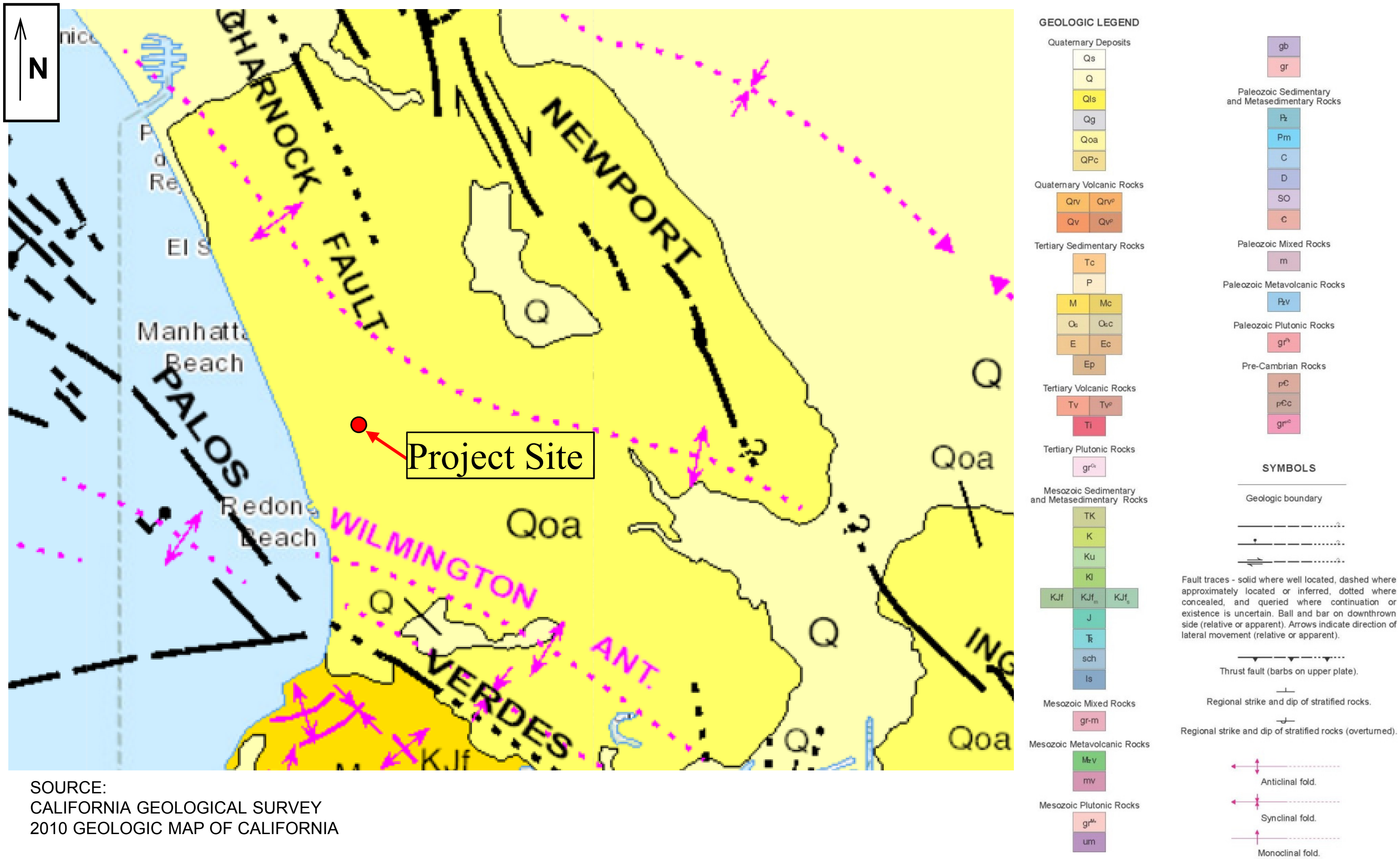
NOT TO SCALE

LEGEND

B-4 Approximate Boring Location

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Aviation Blvd at Artesia Blvd Improvement Project Manhattan Beach, California Project No. MBI-17-001	**SITE VICINITY AND APPROXIMATE BORING LOCATION MAP**	Figure 1



SOURCE:
CALIFORNIA GEOLOGICAL SURVEY
2010 GEOLOGIC MAP OF CALIFORNIA

NOT TO SCALE

HUSHMAND ASSOCIATES, INC. Geotechnical and Earthquake Engineers	**Aviation Blvd at Artesia Blvd Improvement Project** Manhattan Beach, California Project No. MBI-17-001	**REGIONAL GEOLOGY MAP**	Figure 2

30

Depth to anticipated ground water in feet

SOURCE:
CALIFORNIA GEOLOGICAL SURVEY, 2006
SEISMIC HAZARD ZONE REPORT 031
FOR THE REDONDO BEACH 7.5-MINUTE QUADRANGLE

NOT TO SCALE

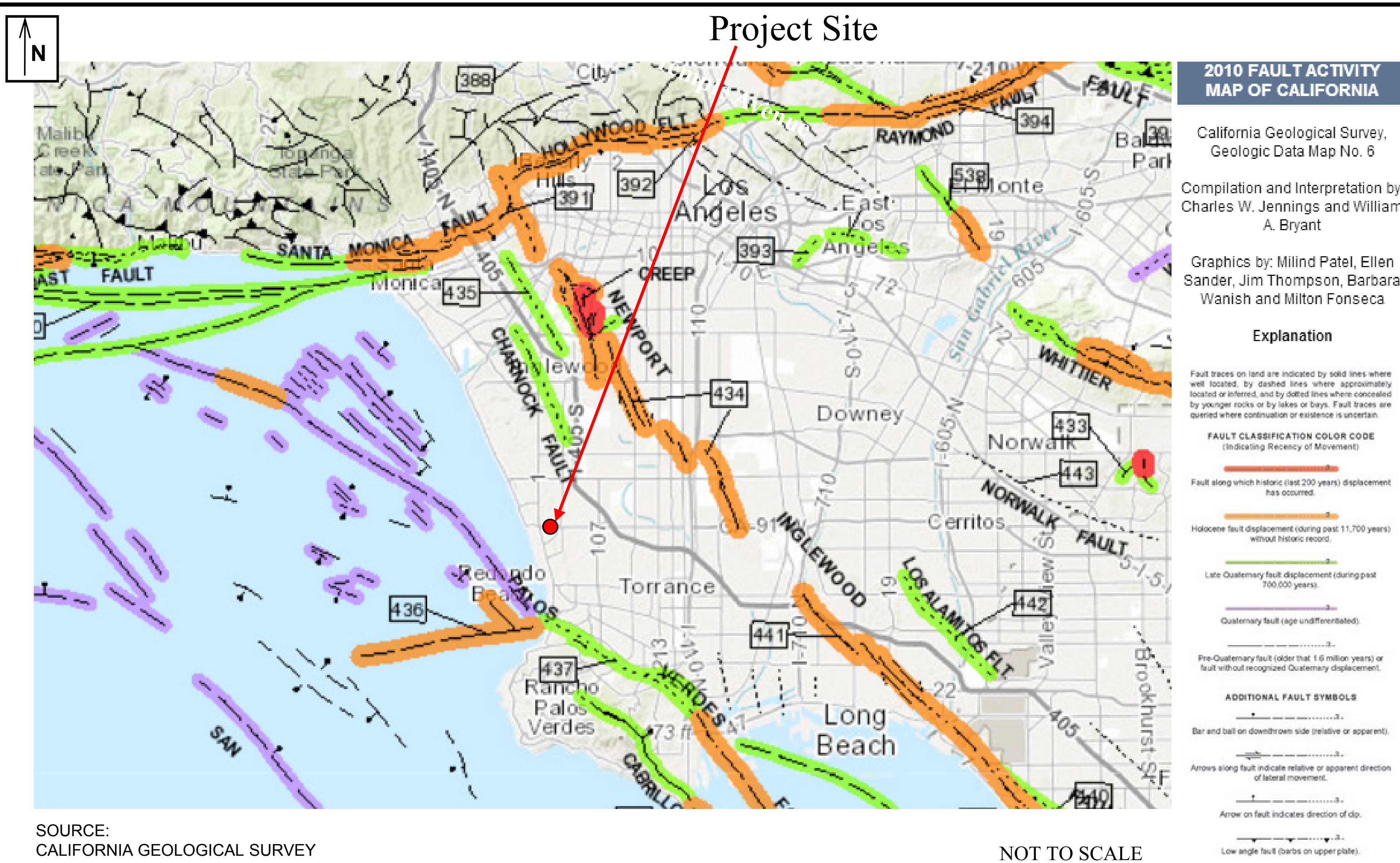
HAI
HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Aviation Blvd at Artesia Blvd Improvement Project
Manhattan Beach, California

Project No. MBI-17-001

HISTORICAL HIGH GROUND WATER CONTOUR MAP

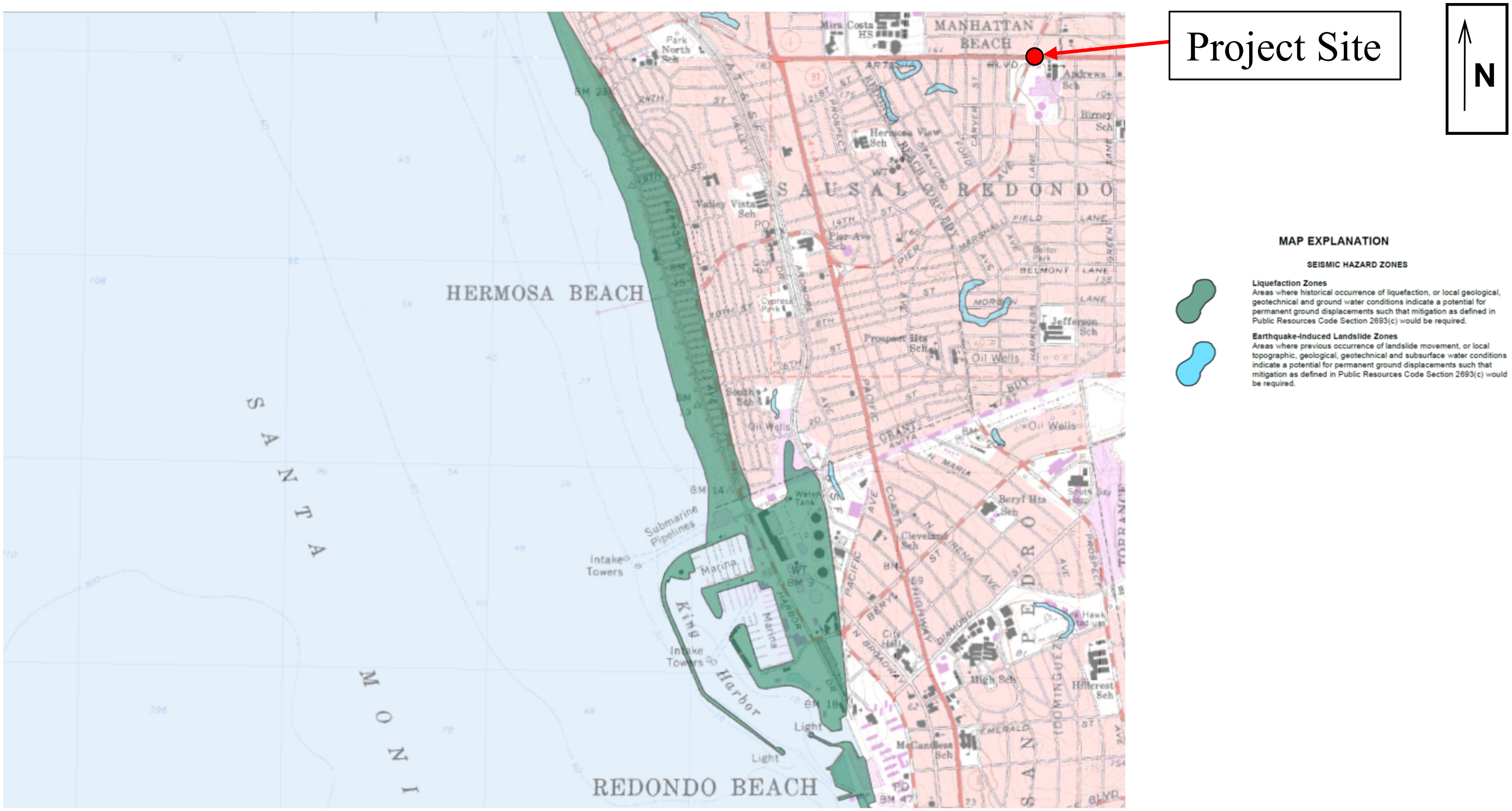
Figure 3



SOURCE:
CALIFORNIA GEOLOGICAL SURVEY
2010 FAULT ACTIVITY MAP OF CALIFORNIA

NOT TO SCALE

HUSHMAND ASSOCIATES, INC. Geotechnical and Earthquake Engineers	**Aviation Blvd at Artesia Blvd Improvement Project** Manhattan Beach, California	**FAULT ACTIVITY MAP**	**Figure** 4
	Project No.: MBI-17-001		



SOURCE:
CALIFORNIA GEOLOGICAL SURVEY, 1999
EARTHQUAKE ZONES OF REQUIRED INVESTIGATION FOR REDONDO BEACH 7.5-MINUTE QUADRANGLE

NOT TO SCALE

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Aviation Blvd at Artesia Blvd Improvement Project
Manhattan Beach, California

Project No. MBI-17-001

SEISMIC HAZARD MAP

Figure 5

APPENDIX A
LOGS OF EXPLORATORY BORINGS

SOIL CLASSIFICATION SYSTEM - ASTM D2487

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	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 SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SAMPLERS

- AU Auger Cuttings
- GB Grab Sample
- MC Modified California Sampler
- RC Rock Core
- SPT Standard Penetration Test Sampler
- ST Shelby Tube

LABORATORY TESTS

RV	: R Value
CONS	: Consolidation
SA	: Sieve Analysis
COMP	: Compaction
EI	: Expansion Index
SE	: Sand Equivalent
UC	: Unconfined Compression
DS	: Direct Shear
HA	: Hydrometer Analysis
%200	: Percentage Passing No. 200 Sieve
AL	: Atterberg Limits
CORR	: Corrosion Potential
SW	: Swell Potential
OM	: Organic Matter

BORING NUMBER B-1

Hushmand Associates, Inc.

CLIENT Michael Baker International, Inc.
PROJECT NAME Aviation Blvd at Artesia Blvd Intersection Improvement
PROJECT NUMBER MBI-17-001
PROJECT LOCATION Aviation Blvd and Artesia Blvd, Manhattan Beach, CA
DATE STARTED 4/7/17 **COMPLETED** 4/7/17
GROUND ELEVATION --- **HOLE SIZE** 8"
DRILLING CONTRACTOR 2R DRILLING INCORPORATED
GROUND WATER LEVELS:
DRILLING METHOD Hand Auger
AT TIME OF DRILLING Not Encountered
LOGGED BY RH **CHECKED BY** JT
AT END OF DRILLING Not Encountered
NOTES Backfilled with soil cuttings
AFTER DRILLING Not Encountered

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	CORE SAMPLE	BULK SAMPLE	SAMPLE NUMBER	BLOW COUNTS (N VALUE)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	OTHER LABORATORY TESTS
		Loose planter barkchips							
		Well-graded SAND with SILT (SW-SM): Light tan to brown, slightly moist to moist, fine to medium grained			AU				
					MC 1		105	6	SA DS
		Boring completed at 4 feet.							

BORING NUMBER B-2

Hushmand Associates, Inc.

CLIENT Michael Baker International, Inc.

PROJECT NAME Aviation Blvd at Artesia Blvd Intersection Improvement

PROJECT NUMBER MBI-17-001

PROJECT LOCATION Aviation Blvd and Artesia Blvd, Manhattan Beach, CA

DATE STARTED 4/7/17 **COMPLETED** 4/7/17

GROUND ELEVATION --- **HOLE SIZE** 8"

DRILLING CONTRACTOR 2R DRILLING INCORPORATED

GROUND WATER LEVELS:

DRILLING METHOD Hand Auger

AT TIME OF DRILLING Not Encountered

LOGGED BY RH **CHECKED BY** JT

AT END OF DRILLING Not Encountered

NOTES Backfilled with soil cuttings

AFTER DRILLING Not Encountered

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	CORE SAMPLE	BULK SAMPLE	SAMPLE NUMBER	BLOW COUNTS (N VALUE)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	OTHER LABORATORY TESTS
		Loose planter barkchips							
		Poorly graded SAND (SP): Light tan to brown, slightly moist to moist, fine to medium grained			AU				CORR
					MC 1		84	8	
		Boring completed at 4 feet.							

BORING NUMBER B-3

Hushmand Associates, Inc.

CLIENT Michael Baker International, Inc.

PROJECT NAME Aviation Blvd at Artesia Blvd Intersection Improvement

PROJECT NUMBER MBI-17-001

PROJECT LOCATION Aviation Blvd and Artesia Blvd, Manhattan Beach, CA

DATE STARTED 4/7/17 **COMPLETED** 4/7/17

GROUND ELEVATION --- **HOLE SIZE** 8"

DRILLING CONTRACTOR 2R DRILLING INCORPORATED

GROUND WATER LEVELS:

DRILLING METHOD HSA

AT TIME OF DRILLING Not Encountered

LOGGED BY RH **CHECKED BY** JT

AT END OF DRILLING Not Encountered

NOTES Backfilled with soil cuttings and patched with cold AC

AFTER DRILLING Not Encountered

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	CORE SAMPLE	BULK SAMPLE	SAMPLE NUMBER	BLOW COUNTS (N VALUE)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	OTHER LABORATORY TESTS
		6" Asphaltic Concrete							
		10" Base Material							
		Silty SAND (SM): Dark brown, moist, fine to medium grained			AU				RV
5		Silty SAND (SM): Tan brown, moist, medium dense, fine grained			MC 1 A,B	7-8-9 (17)	125	10	SA DS
10		Silty SAND (SM): Brown, slightly moist, medium dense, fine to medium grained			MC 2 A,B	5-6-12 (18)	101	6	SA
15		Silty SAND (SM): Reddish brown, slightly moist, dense, fine grained, trace of clay			MC 3 A,B	11-18-30 (48)	113	11	
		Boring completed at 16.5 feet.							

Hushmand Associates, Inc.

BORING NUMBER B-4

CLIENT Michael Baker International, Inc.
PROJECT NAME Aviation Blvd at Artesia Blvd Intersection Improvement
PROJECT NUMBER MBI-17-001
PROJECT LOCATION Aviation Blvd and Artesia Blvd, Manhattan Beach, CA
DATE STARTED 4/7/17 **COMPLETED** 4/7/17
GROUND ELEVATION --- **HOLE SIZE** 8"
DRILLING CONTRACTOR 2R DRILLING INCORPORATED
GROUND WATER LEVELS:
DRILLING METHOD HSA
AT TIME OF DRILLING Not Encountered
LOGGED BY RH **CHECKED BY** JT
AT END OF DRILLING Not Encountered
NOTES Backfilled with soil cuttings and patched with cold AC
AFTER DRILLING Not Encountered

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	CORE SAMPLE	BULK SAMPLE	SAMPLE NUMBER	BLOW COUNTS (N VALUE)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	OTHER LABORATORY TESTS
		6" Asphaltic Concrete							
		6" Base Material							
		Silty SAND (SM): Dark brown, slightly moist, fine to medium grained			AU				EI CORR
5		Silty SAND (SM): Reddish medium brown, slightly moist to moist, medium dense, fine grained			MC 1	5-7-10 (17)	116	10	
10		Poorly graded SAND (SP): Tan, dry to slightly moist, medium dense, fine to medium grained			MC 2	5-7-10 (17)	95	5	
15		Silty SAND (SM): Reddish brown and black, slightly moist, very dense, fine to medium grained, trace of clay			MC 3	13-30-50 (80)	104	11	
		Boring completed at 16.5 feet.							

APPENDIX B
LABORATORY TEST RESULTS

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

MOISTURE CONTENT AND DRY DENSITY OF RING SAMPLES
(ASTM D2216, 2937)

Client: Michael Baker International, Inc.
Project Name: Aviation Blvd at Artesia Blvd Improvements
Project No.: ---

HAI Project No.: MBI-17-001
Performed by: MB/KL
Checked by: MZ
Date: 4/20/2017

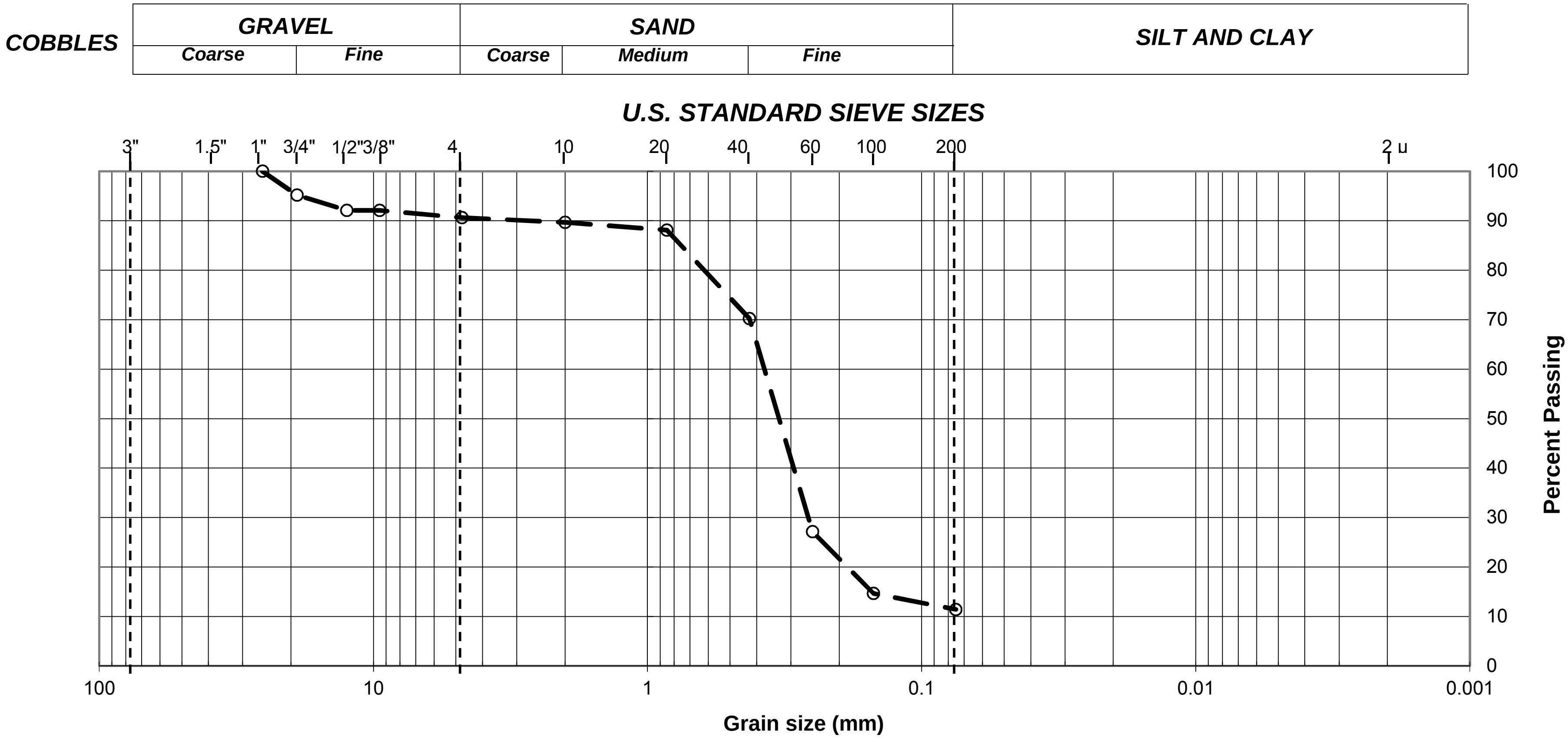
Boring No.		**B-2**	**B-3**		**B-4**		
Sample Type		**MC-1**	**MC-2B**	**MC-3B**	**MC-1B**	**MC-2B**	**MC-3B**
Depth (ft)		**2.5-3**	**10.5-11**	**15.5-16**	**5.5-6**	**10.5-11**	**15.5-16.0**
Total wt of rings and soil	gr	151.05	859.15	967.50	982.13	653.39	181.72
Height of sample	in	1	5	5	5	4	1
Diameter of sample	in	2.416	2.416	2.416	2.416	2.416	2.416
Volume of sample	cu.ft	0.0027	0.0133	0.0133	0.0133	0.0106	0.0027
Weight of rings	gr	42.87	214.36	214.36	214.36	171.49	42.87
Weight of soil	lbs.	0.238	1.422	1.660	1.693	1.062	0.306
Wet Density	pcf	89.9	107.2	125.2	127.6	100.1	115.4
Container No.		28	H20	E92	200	P16	26
Weight of cont.+ wet soil	gr	117.85	392.92	325.60	111.22	224.30	109.22
Weight of cont.+ dry soil	gr	109.93	371.98	295.04	101.90	213.47	99.23
Weight of container	gr	4.91	8.00	8.36	6.34	10.81	4.94
Weight of water	gr	7.92	20.94	30.56	9.32	10.83	9.99
Weight of dry soil	gr	105.02	363.98	286.68	95.56	202.66	94.29
Moisture Content	**%**	**7.5**	**5.8**	**10.7**	**9.8**	**5.3**	**10.6**
Dry Density	**pcf**	**83.6**	**101.3**	**113.1**	**116.3**	**95.0**	**104.3**

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

PARTICLE-SIZE ANALYSIS OF SOILS

(ASTM D422)

Client:	Michael Baker International, Inc.	***HAI Project No.:***	MBI-17-001
Project Name:	Aviation Blvd at Artesia Blvd Improvements	***Tested by:***	MB/KL
Project No.:	---	***Checked by:***	MZ
		Date:	4/20/2017

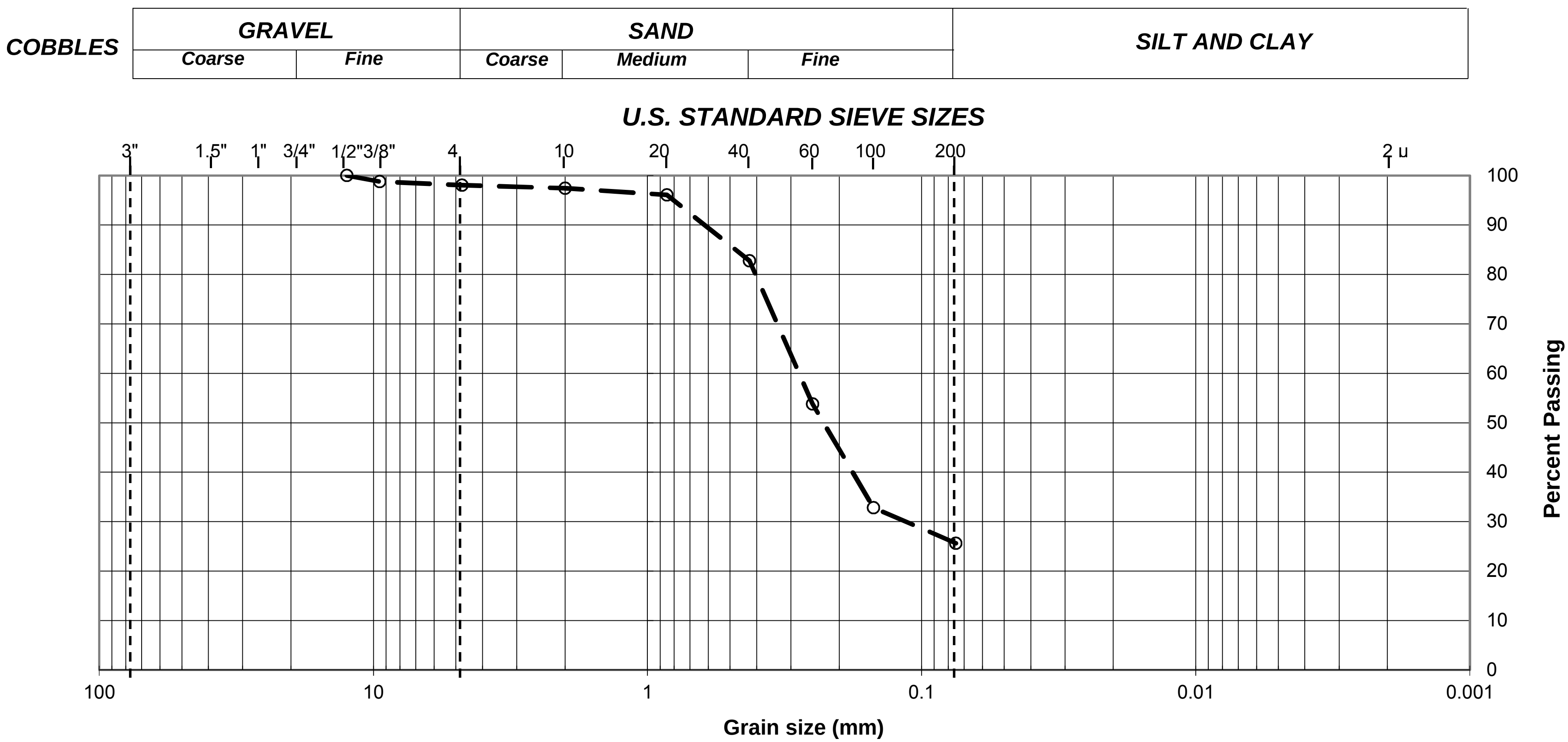


Boring No.	Sample No.	Depth	Symbol	USCS	% Gravel	% Sand	% Fines
B-1	MC-1	2.5-3'	○	Tan Brown, Well-graded Sand with Silt (SW-SM)	9.4	79.2	11.4

PARTICLE-SIZE ANALYSIS OF SOILS

(ASTM D422)

Client:	Michael Baker International, Inc.	***HAI Project No.:***	MBI-17-001
Project Name:	Aviation Blvd at Artesia Blvd Improvements	***Tested by:***	MB/KL
Project No.:	---	***Checked by:***	MZ
		Date:	4/20/2017

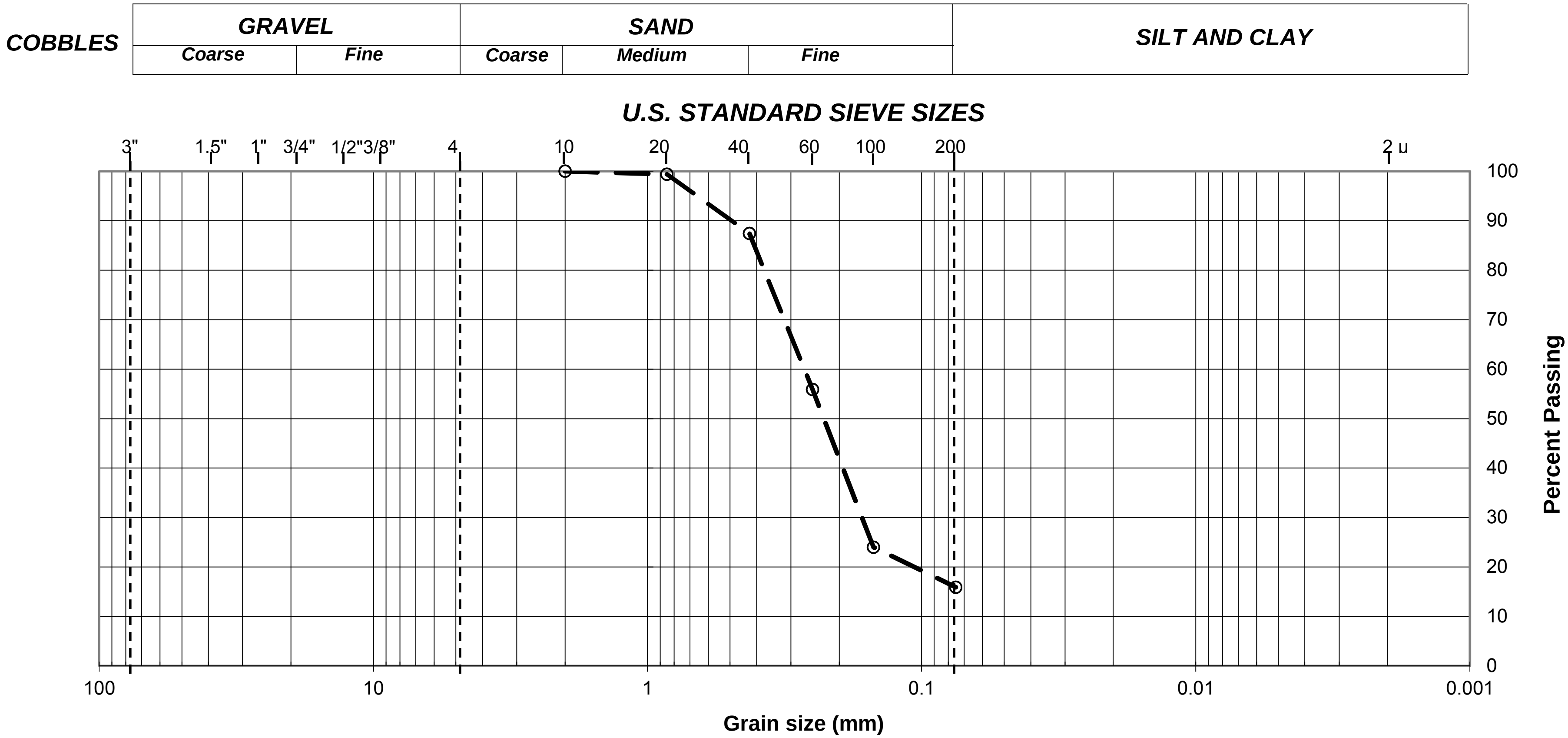


Boring No.	Sample No.	Depth	Symbol	USCS	% Gravel	% Sand	% Fines
B-3	MC-1B	5.5-6'	○	Tan Brown, Silty Sand (SM)	2.0	72.4	25.6

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

PARTICLE-SIZE ANALYSIS OF SOILS
(ASTM D422)

Client:	Michael Baker International, Inc.	***HAI Project No.:***	MBI-17-001
Project Name:	Aviation Blvd at Artesia Blvd Improvements	***Tested by:***	MB/KL
Project No.:	---	***Checked by:***	MZ
		Date:	4/20/2017



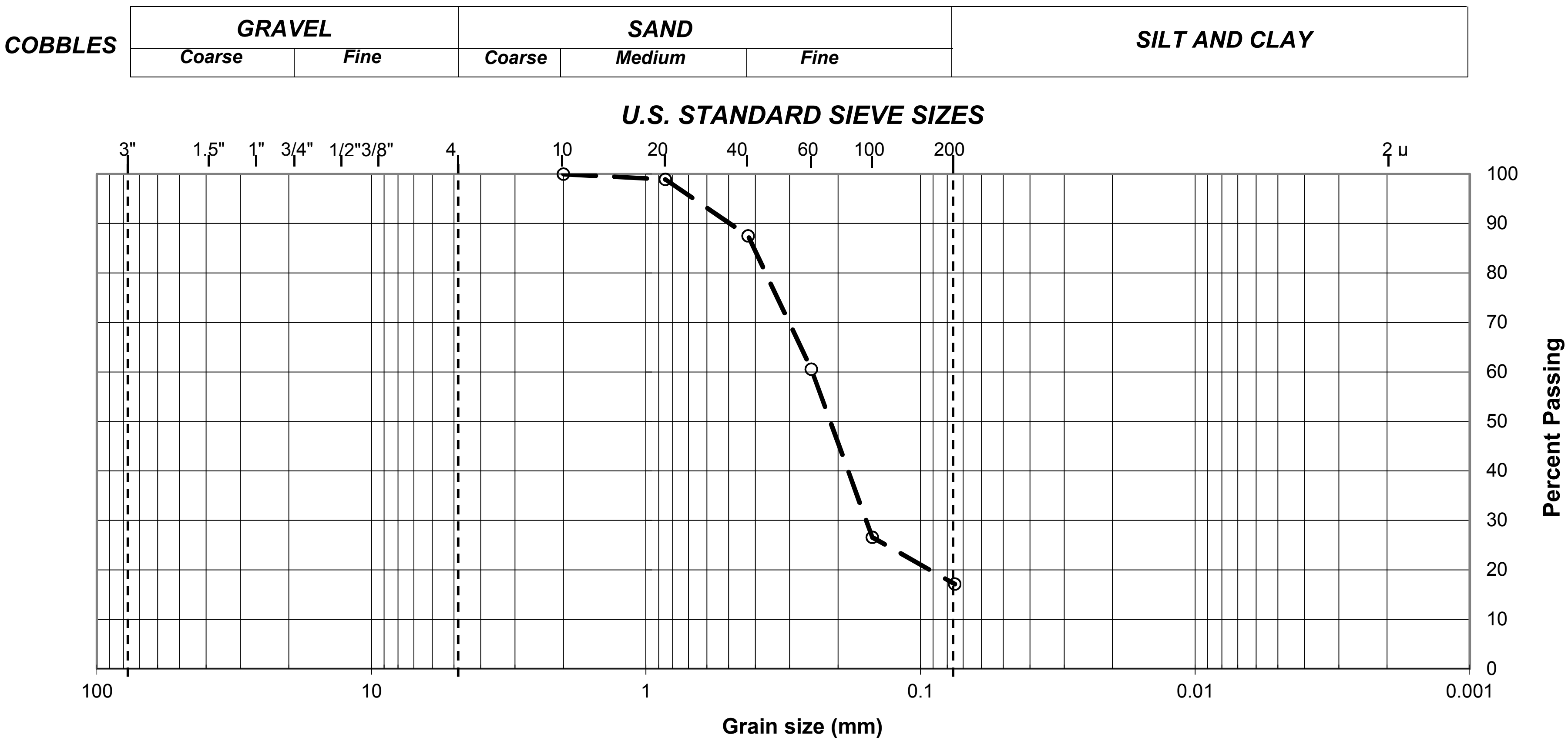
Boring No.	Sample No.	Depth	Symbol	USCS	% Gravel	% Sand	% Fines
B-3	MC-2B	10.5-11'	○	Brown, Silty Sand (SM)	0.0	84.1	15.9

PARTICLE-SIZE ANALYSIS OF SOILS

(ASTM D422)

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client:	Michael Baker International, Inc.	***HAI Project No.:***	MBI-17-001
Project Name:	Aviation Blvd at Artesia Blvd Improvements	***Tested by:***	MB/KL
Project No.:	---	***Checked by:***	MZ
		Date:	4/20/2017



Boring No.	Sample No.	Depth	Symbol	USCS	% Gravel	% Sand	% Fines
B-3	MC-3B	15.5-16'	○	Reddish Brown, Silty Sand (SM)	0.0	82.9	17.1

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

EXPANSION INDEX (ASTM D4829)

Client:	Michael Baker International, Inc.	***HAI Project No.:***	MBI-17-001
Project Name:	Aviation Blvd at Artesia Blvd Improvements	***Tested by:***	AH
Boring No.:	B-4	***Checked by:***	MZ
Sample No.:	Bulk-4	***Depth:*** 0-5' / ***Date:***	4/20/2017
Soil Description:	Dark Brown, Silty Sand (SM)		

MOLDED SPECIMEN		
Wt. of wet soil + cont.	**105.56**	g
Wt. of dry soil + cont.	**97.65**	g
Wt. of container	**6.35**	g
Wt. of water	**7.91**	g
Wt. of dry soil	**91.30**	g
Moisture Content	**8.7**	%
Wt. of wet soil + ring	**613.34**	g
Wt. of ring	**206.92**	g
Wt. of wet soil	**406.42**	g
Wet density of soil	**123.2**	pcf
Dry density of soil	**113.3**	pcf
Specific gravity of soil	**2.65**	
Saturation	**50.0**	%

MOISTURE CONTENT AFTER TEST		
Wt. of wet soil + cont.	**615.15**	g
Wt. of dry soil + cont.	**572.72**	g
Wt. of container	**206.92**	g
Wt. of water	**42.43**	g
Wt. of dry soil	**365.80**	g
Moisture Content	**11.6**	%

Date & time	Elapsed time (min)	Dial Reading	Δh, Expansion
4/12/2017 11:55	0	0	
4/12/2017 12:05	10	0.0000	
Add distilled water to sample			
4/13/2017 11:55	1440	0.0000	0.0000

Expansion Index = 0

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

DIRECT SHEAR TEST
ASTM D3080

HAI Pr No.: MBI-17-001
Tested by: KL
Checked by: MZ
Date: 4/20/2017

Client: Michael Baker International, Inc.
Project Name: Aviation Blvd at Artesia Blvd Improvements
Project Number: ---

Boring No.: B-1

Sample No.: MC-1

Depth (ft): 2.5-3

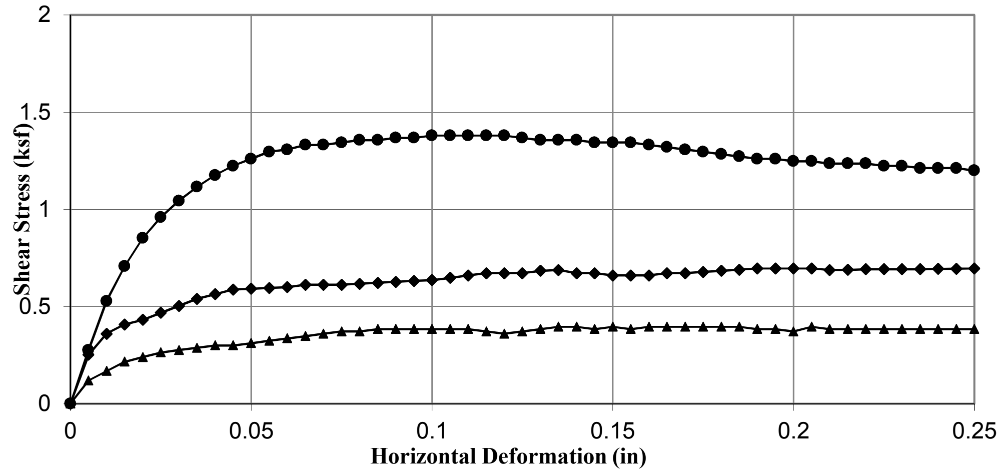
Soil Description: Tan Brown, Well-graded Sand with Silt (SW-SM)
Sample Type: Undisturbed ring

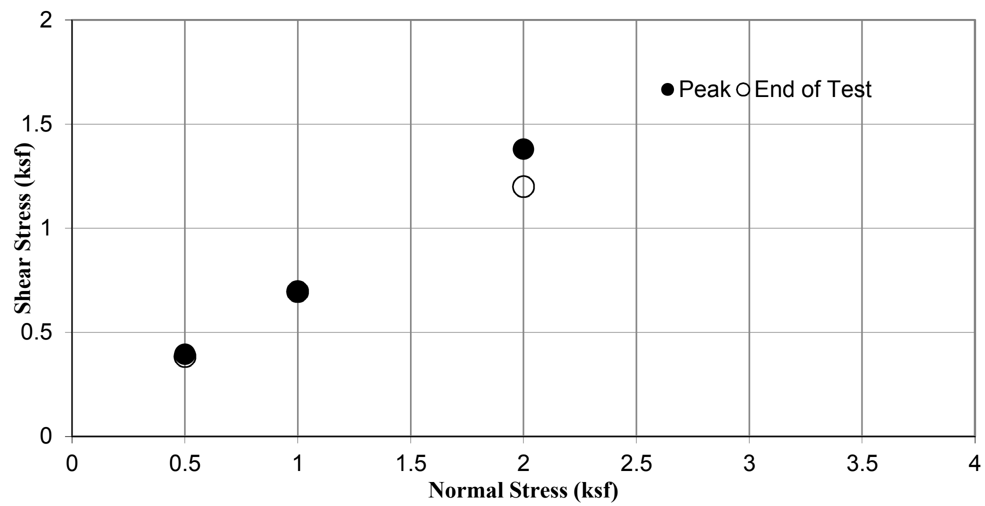
Type of Test: Consolidated, Drained

	▲	◆	●
Normal Stress (ksf)	**0.5**	**1**	**2**
Deformation Rate (in/min)	**0.002**		

Peak Shear Stress (ksf) ●	**0.40**	**0.70**	**1.38**
Shear stress @ end of test (ksf) ○	**0.38**	**0.70**	**1.20**

Initial height of sample (in)	**1**	**1**	**1**
Height of sample before shear (in)	**0.9983**	**0.9995**	**1.0024**
Diameter of sample (in)	**2.42**	**2.42**	**2.42**
Initial Moisture Content (%)	**6.4**	**6.4**	**6.4**
Final Moisture Content (%)	**18.4**	**15.4**	**16.7**
Initial Dry Density (pcf)	**99.6**	**105.4**	**108.5**





HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

DIRECT SHEAR TEST

ASTM D3080

HAI Pr No.: MBI-17-001
Tested by: KL
Checked by: MZ
Date: 4/20/2017

Client: Michael Baker International, Inc.
Project Name: Aviation Blvd at Artesia Blvd Improvements
Project No.: ---

Boring No.: B-3

Sample No.: MC-1B

Depth (ft): 5.5-6

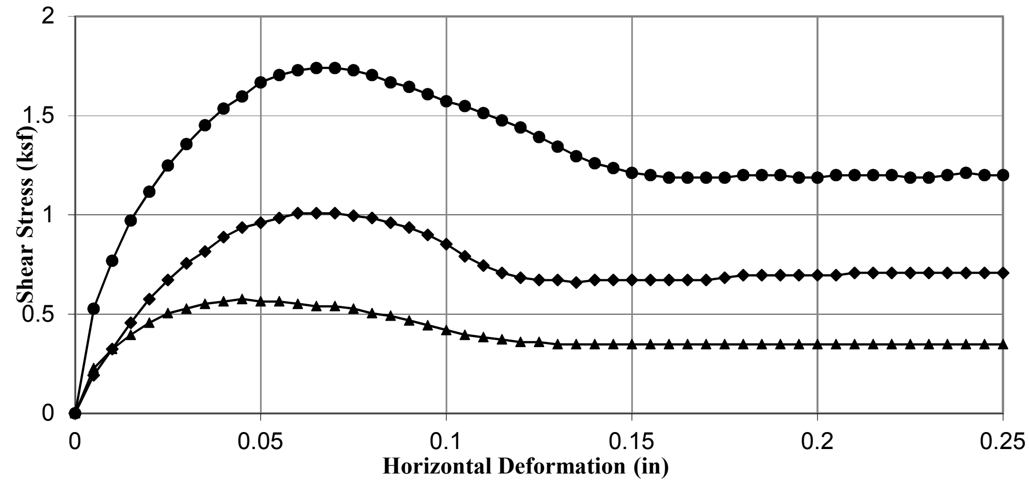
Soil Description: Tan Brown, Silty Sand (SM)
Sample Type: Undisturbed ring

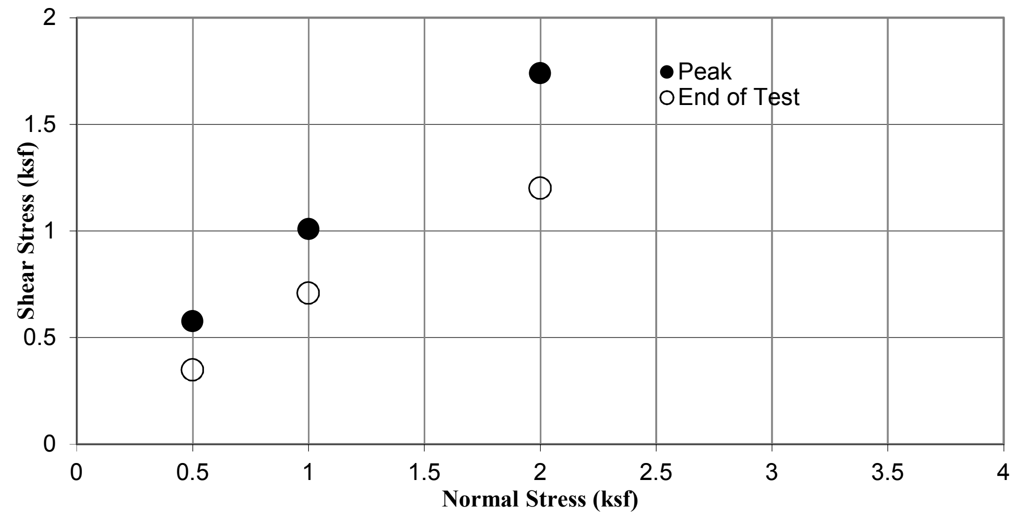
Type of Test: Consolidated, Drained

	▲	◆	●
Normal Stress (ksf)	0.5	1	2
Deformation Rate (in/min)	0.002		

Peak Shear Stress (ksf) ●	0.58	1.01	1.74
Shear stress @ end of test (ksf) ○	0.35	0.71	1.20

Initial height of sample (in)	1	1	1
Height of sample before shear (in)	1.0056	1.0079	0.9909
Diameter of sample (in)	2.42	2.42	2.42
Initial Moisture Content (%)	9.6	9.6	9.6
Final Moisture Content (%)	11.7	7.0	10.9
Initial Dry Density (pcf)	125.4	125.1	125.7





LaBelle Marvin

R - VALUE DATA SHEET

PROJECT No. 42350

DATE: 4/25/2017

BORING NO. B-3 @ 0-5'
Aviation Blvd @ Artesia Blvd Impr.
P.N. MBI-17-001

SAMPLE DESCRIPTION: Brown Silty Sand

R-VALUE TESTING DATA | CA TEST 301

	SPECIMEN ID		
	a	b	c
Mold ID Number	1	2	3
Water added, grams	55	38	33
Initial Test Water, %	9.7	8.1	7.6
Compact Gage Pressure,psi	50	190	240
Exudation Pressure, psi	193	346	474
Height Sample, Inches	2.52	2.50	2.46
Gross Weight Mold, grams	3085	3083	3062
Tare Weight Mold, grams	1965	1969	1977
Sample Wet Weight, grams	1120	1114	1085
Expansion, Inches x 10exp-4	0	4	7
Stability 2,000 lbs (160psi)	21 / 43	15 / 29	14 / 28
Turns Displacement	5.23	4.65	4.48
R-Value Uncorrected	57	71	72
R-Value Corrected	57	71	72
Dry Density, pcf	122.8	124.9	124.2

DESIGN CALCULATION DATA

Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.44	0.30	0.29
G. E. by Expansion	0.00	0.13	0.23

Equilibrium R-Value	**68** by EXUDATION	Examined & Checked: 4 /25/ 17

REMARKS:
Gf = 1.25
0.0% Retained on the
3/4" Sieve.

REGISTERED PROFESSIONAL ENGINEER STEVEN R. MARVIN C 30659 CIVIL STATE OF CALIFORNIA

Steven R. Marvin, RCE 30659

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

R-VALUE GRAPHICAL PRESENTATION

LaBelle Marvin

PROJECT NO. 42350

DATE: 4 /25/ 16

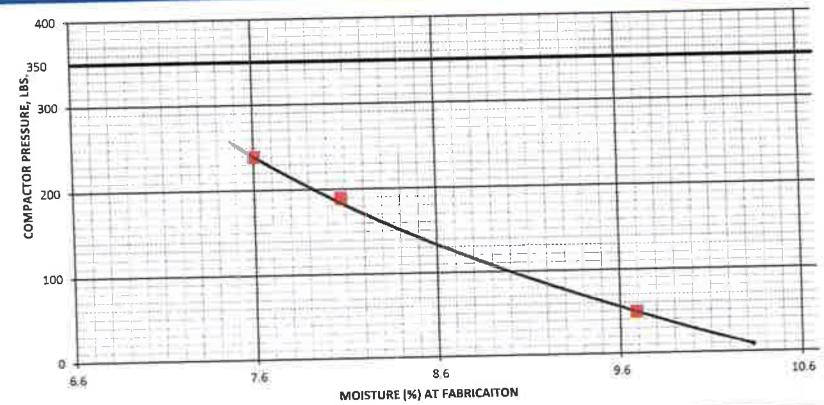
REMARKS:

BORING NO. B-3 @ 0-5'

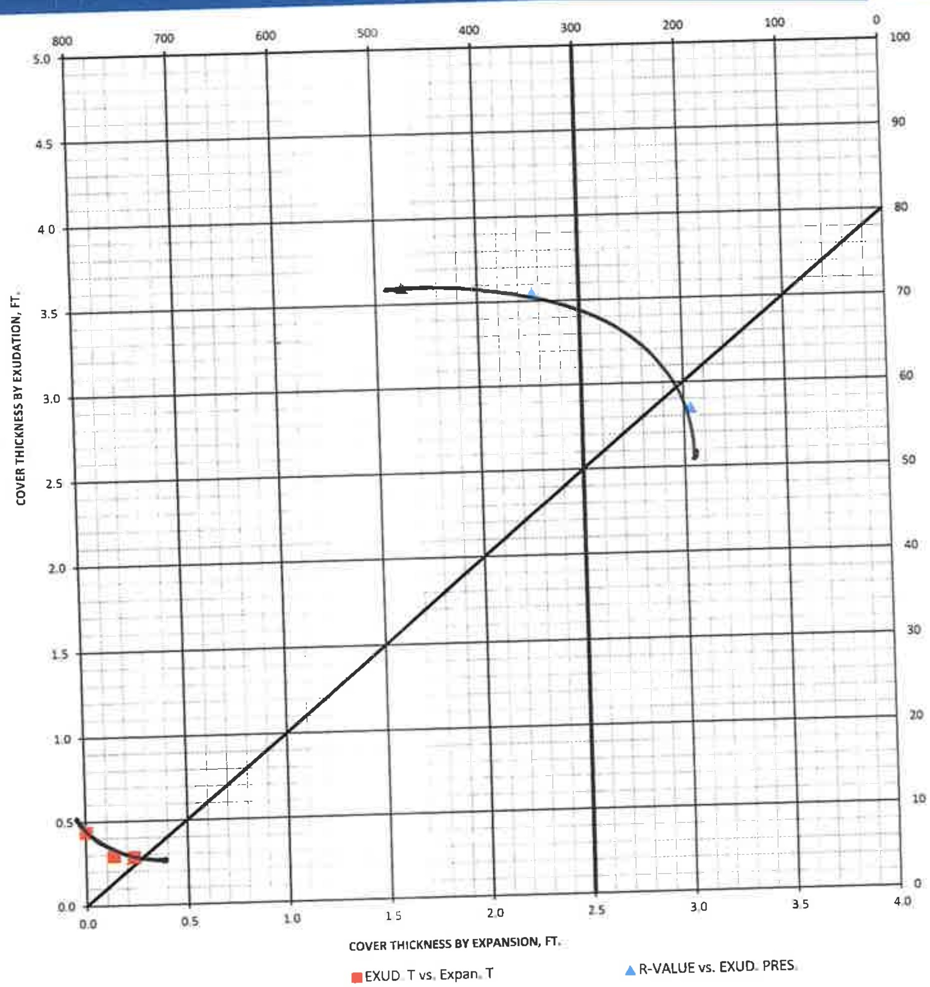
Aviation Blvd @ Artesia Blvd Impr.

P.N. MBI-17-001

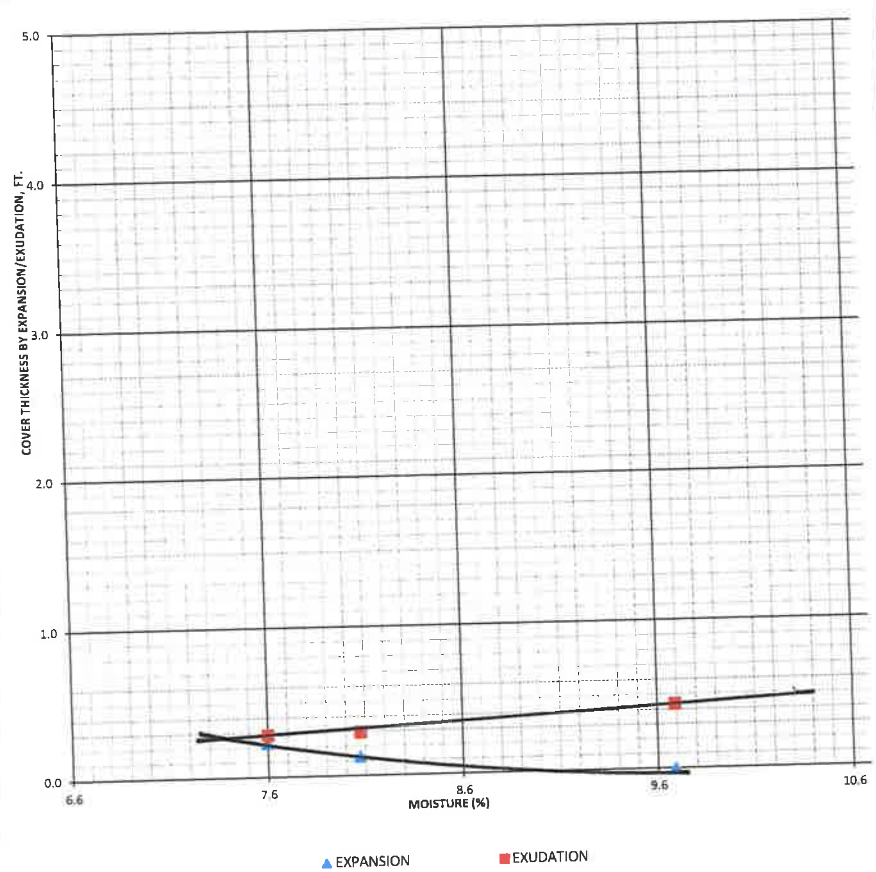
COMPACTOR PRESSURE vs MOISTURE %



COVER THICKNESS BY EXUDATION vs COVER THICKNESS BY EXPANSION



COVER THICKNESS vs MOISTURE %



SOIL ANALYSIS LAB RESULTS

Client: HAI
Job Name: Aviation Blvd. and Artesia Blvd. Improvement
Client Job Number: MBI-17-001
Project X Job Number: S170411A
April 13, 2017

	Method	ASTM G187	CTM 417		CTM 422		CTM 643
Bore# / Description	**Depth**	**Min-Resistivity**	**Sulfates**		**Chlorides**		**pH**
	(ft)	**(Ohm-cm)**	**(mg/kg)**	**(wt%)**	**(mg/kg)**	**(wt%)**	
B-2 Bulk 2	4.0	6,968	30	0.0030	30	0.0030	8.65
B-4 Bulk 4	5.0	16,750	60	0.0060	39	0.0039	7.76

Unk = Unknown
NT = Not Tested
ND = 0 = Not Detected
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
mg/L - milligrams per liter of liquid volume
Chemical Analysis performed on 1:3 Soil-To-Water extract

Please call if you have any questions.

Prepared by,

Ernesto Padilla, BSME
Field Engineer

Respectfully Submitted,

Eddie Hernandez, M.Sc., P.E.
Sr. Corrosion Consultant
NACE Corrosion Technologist #16592
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