Appendix D Preliminary Geotechnical Exploration

Appendix

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PRELIMINARY GEOTECHNICAL EXPLORATION
GRAND VIEW ELEMENTARY SCHOOL
455 24TH STREET
MANHATTAN BEACH, CALIFORNIA

Prepared for:

MANHATTAN BEACH UNIFIED SCHOOL DISTRICT
325 South Peck Avenue
Manhattan Beach, California 90266-6946

Project No. 11671.003

Friday, September 14, 2018

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

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Project No. 11671.003

Manhattan Beach Unified School District
325 South Peck Avenue
Manhattan Beach, California 90266-6946

Attention: Dr. Dawnalyn Murakawa-Leopard
Deputy Superintendent

Subject: Preliminary Geotechnical Exploration
Grand View Elementary School
Portion Lot 2 (Parcel 1) and Portion Lot 3 (Parcel 2) of Tract 2356
455 24th Street
Manhattan Beach, Los Angeles County, California

In accordance with our July 11, 2018 proposal, authorized by your July 18, 2018 *Consultant Services Agreement*, Leighton Consulting, Inc. has completed this preliminary geotechnical exploration for modernization of Manhattan Beach Unified School District's existing Grand View Elementary School; located within northwestern Manhattan Beach close to the coast. We understand that the currently-proposed layout of improvements incorporated in this report is conceptual, and subject to future refinement and revision. We also understand the District intends to submit this report to the California Geological Survey (CGS) for site-specific geologic hazards review.

This campus is **not** located within a currently-designated Alquist-Priolo Earthquake Fault Zone of potential surface fault rupture, nor zone of potential seismically-induced liquefaction. However, CGS does designate the east facing (leeward) side of the sand dune on the northern portion of this site (Sand Dune Park) as regionally-mapped within a zone of potential seismically-induced landsliding. Based on a record of historical seismic activity, strong ground shaking has occurred and should be anticipated in the future at this site, as is the case for most of Southern California.

As a generalized description, this site can be characterized as sand dunes, underlain predominantly with uniform fine sands (≤5% fines silt or clay), with subsets as undocumented fill soils, more recent and older aeolian deposits. Existing fill soils should be recompacted to support new one- to two-story structures; but otherwise,

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undisturbed native dune sands should provide adequate support for spread footings. Primary concern with all dune sands, including this site, is the propensity for ongoing sand migration near the surface. Slopes should be cut and constructed no-steeper-than 2:1 (horizontal:vertical) and will require artificial stabilization of the surface with deep-rooted, drought-resistant vegetation and augmented topsoil, geogrids, geocells, cribs and/or other imported materials and systems. In summary, these cohesionless sands must be confined to provide continued support for site improvements and to mitigate blowing sand hazards.

We appreciate the opportunity to be of additional service to the District. If you have any questions or if we can be of further service, please contact us at ***(866) LEIGHTON***, directly at the phone extensions and/or e-mail addresses listed below.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

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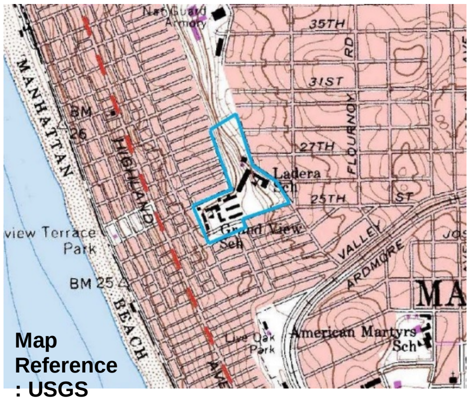
1.0 PROJECT DESCRIPTION

1.1 Site Description

As shown on Figure 1, *Site Location Map*, and Plate 1, *Geotechnical Exploration Map* (in pocket), this campus is currently developed as Grand View Elementary School at 455 24th Street, and adjacent (to the northeast) Montessori School of Manhattan Beach at 2617 Bell Avenue, within northwestern Manhattan Beach, Los Angeles County, California, at Latitude N33.895° Longitude W-118.411°. This irregular-shaped site (as shown on the inset figure to the right) encompass two contiguous parcels including a portion of Parcel 1 (Lot 2) on the south, and a portion of Parcel 2 (Lot 3) on the north. This site is located within the Venice 7.5-Minute Topographic Quadrangle Map (USGS, 1981). The adjacent insert map shows approximate site boundaries in ***blue outline***, and quadrangle-mapped (1981) building footprints in ***black***.



Map Reference : USGS

This contiguous elementary school campus is surrounded by a single-family residential neighborhood, bounded on the northeast by Bell Avenue, on the south by 24th Street Place and Manor Avenue, by Vista Drive, 26th Street and Grandview Avenue on the west, and Sand Dune Park on the north. A site topographic survey by PBLA (2018) is used as the base of our Plate 1 (in pocket). Approximate surface elevations along Bell Avenue on the east, range between elevation (El.) 75 and El. 85 feet above mean sea level (amsl). Along 24th Street on the south, site elevations range from El. 80 to El. 147 feet amsl. Elevations along the westerly site margin range from El. 161 to El. 175 feet amsl. As can be seen on Plate 1 (in pocket) there is a relatively steep slope within the northern portion of this site, as the leeward side of a large sand dune, rising roughly 80 feet from the existing campus up to Grandview Avenue (west property line).

1.2 Proposed Conceptual Improvements

Our current understand of this project is based on DLR Group's June 20, 2018 Schematic Design "*New Site Plan*," DLR Project No. 75-18214-00, which is reproduced in part on Plate 1 (in pocket). Two new buildings are proposed as follows:

Table 1 – Proposed New Buildings

New Building	Stories	Footprint (square-feet)
Multi-Purpose Room (MPR) + Administation Building	2	≤12,000
Classroom Building	≤2	10,850
TOTAL:		**10,852**

Finish floor elevations for these two buildings are unknown at this time. Based on this current conceptual site plan by DLR Group, as depicted on Plate 1 (in pocket), in addition to constructing these two new buildings, improvements planned include demolition of some classroom buildings, cut-reconfiguration of the central internal terrace slopes, new fire access driveways, a significant stairway structure on-grade and modernization of existing buildings. Appurtenant improvements will include a new sports field and hardscaped walkways at the northern portion of the site.

A grading plan was unavailable at the time we prepared this report, although conceptual grading (terrace) concepts are depicted on Plate 1 (in pocket) and interpreted on Plate 2, *Geotechnical Cross Sections A-A', B-B', C-C', D-D', E-E'* (in pocket). A retaining wall will likely be required on the west edge of the north-south aligned fire access driveway (although new heights cannot be discerned at this time).

1.3 Purpose and Scope of Work

Purpose of our exploration was to: (1) evaluate geotechnical conditions in the vicinity of proposed building footprints, (2) identify significant geotechnical or geologic issues that would impact this proposed campus modernization, and (3) provide geotechnical recommendations for design and construction of proposed buildings and appurtenant site improvements. In accordance with our July 11, 2018 proposal, authorized by your July 18, 2018 *Consultant Services Agreement*, scope of our exploration included the following:

- **Document Review**: We reviewed available and provided geologic literature, reports, and historical topographic maps and aerial photographs relevant to this site. Pertinent geotechnical documents are referenced at the end of this report text.

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- **Subsurface Exploration**: Our Certified Engineering Geologist (CEG) visited this site to observe exposed geologic conditions. On August 7th. and 8th., 2018, nine hollow-stem-auger borings were drilled, logged and sampled to depths ranging from 31½ to 51½ feet below existing grades. On August 7th, five shallow hand-auger borings were advanced, logged and sampled to depths of five to six feet, in areas not accessible by drill rig. After sampling and logging, all borings were immediately backfilled with drill tailings. Approximate boring locations are depicted in **blue** on Plate 1, *Geotechnical Exploration Map* (in pocket). A description of our field exploration and boring logs are presented in Appendix A, *Field Exploration*.

- **Geotechnical Laboratory Testing**: Geotechnical laboratory tests were conducted on a selected suite of relatively undisturbed and bulk soil samples obtained from our borings. This laboratory testing program was designed to evaluate physical geotechnical characteristics of site soils. A description of geotechnical test procedures and results are presented in Appendix B, *Geotechnical Laboratory Testing,* and summarized within our boring logs in Appendix A, *Field Exploration*.

- **Engineering Analysis**: Resultant field exploration and geotechnical laboratory testing data were analyzed to develop geotechnical conclusions and provide preliminary geotechnical recommendations, in accordance with California Geological Survey (CGS) Note 48 (October 2013 version). Geotechnical cross sections were specifically located to highlight conditions in areas of planned improvements, and are depicted on Plate 2, *Geotechnical Cross Sections A-A', B-B', C-C', D-D', and E-E'*.

- **Report Preparation**: Results of our geologic hazards review and geotechnical exploration have been summarized in this report, presenting our findings, conclusions and preliminary geotechnical design recommendations for proposed improvements depicted on Plate 1 (in pocket) and described in Section 1.2 of this report.

This report does not address potential for encountering hazardous materials in site soils nor groundwater. Important information about limitations of geotechnical reports in general, is presented in Appendix C, *GBA's Important Information About This Geotechnical-Engineering Report*.

2.0 GEOTECHNICAL FINDINGS

2.1 Pre-Development Conditions

Site conditions existing prior to development were determined based on a review of historical topographic maps and aerial photographs (see references). The regional location of the site with reference to major geographic landforms is depicted on Figure 2, *Regional Geology Map*. Prior to 1942, this property was undeveloped (USGS, 1942). Original topography consisted of gentle to moderately northeast-sloping terrain with an overall relief of approximately 75 vertical feet. As further described in Section 2.3 of this report, this site is situated on the eastern “lee” side of a northwesterly trending sand dune ridge that parallels the nearby Pacific Ocean coastline. This ridge parallels the coast for approximately 11 miles between the Palos Verdes Peninsula and Playa del Rey.

Within the southerly two-thirds of the property was a relatively broad and shallow northeasterly-flowing drainage swale, eroded into the east side of the dune ridge. This swale descended easterly from the upper area of the ridge, at approximate El. 145 feet amsl, to an area of low-lying enclosed topographic depressions that bordered the site on the east, at approximate El. 75 feet amsl (USGS, 1942).

The northern third of the property was occupied by a disposal stockpile, placed sometime prior to 1953 (historical aerials, 1953). It appears placement of this stockpile was associated with development of neighboring residential area offsite to the west during the 1930’s. It appears the source of this stockpile material was from the upper portion of the dune ridge, and composed of sands similar in composition to those encountered beneath this site. Approximate limits of this stockpile extended beyond the landward toe of the dune ridge, from north to south between a projection of 27th and 29th Streets, and easterly to within approximately 160 feet of future Bell Avenue. By 1963, the stockpile had been extended further east to abut Bell Avenue, and further north to a projection of 29th Place. The upper surface of the stockpile had been leveled by that time, generally consistent with the currently existing topography. Based on current topography, we estimate the stockpile is on the order of 36 to 40 feet thick.

2.2 Post-Development Conditions

Our review of historical topographic maps and aerial photographs revealed that by 1952, a first phase of earthwork grading and construction had been completed for the Grand View Elementary School improvements, within the Parcel 2/Lot 1 area encompassing the southwestern third of the site (USGS, 1952; Historical Aerial, 1953). This area is referred to as the “upper” campus. Grading resulted in relief that is largely consistent with present day topography, including a broad, relatively flat-lying surface occupied by several single-story classroom, administration and library buildings and paved parking. The initial grading phase encroached into the southeastern third of the Parcel 2/Lot 1 area, referred to as

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the "lower" campus, including establishment of a level pad (location of present day grass field, playground, and paved sports courts), and internal centrally located north/south-trending terrace cut slope.

By 1963, as part of a second phase of grading, the remaining lower campus area was configured to its present-day layout, including establishment of another north/south-trending internal terrace slope, level building pads, retaining walls and a perimeter fill slope along Bell Avenue. Lower Campus construction included an "L" shaped building(s) for the Grand View School and detached square-shaped Montessori School building, located furthest north on Parcel 2/Lot 1. The resultant layout is depicted on Figure 1, *Site Location Map*.

In comparing elevation contours on 1942 and 1982 topographic maps, it appears a majority of the building pads and internal slopes in upper and lower campus areas were established by a "cutting" of original ground surfaces (USGS, 1942, 1982). It appears fill was placed to raise grades within the eastern lower campus area, to infill low-lying areas of the former erosional swale, possibly top off terrace slopes, and create the perimeter slope along Bell Avenue.

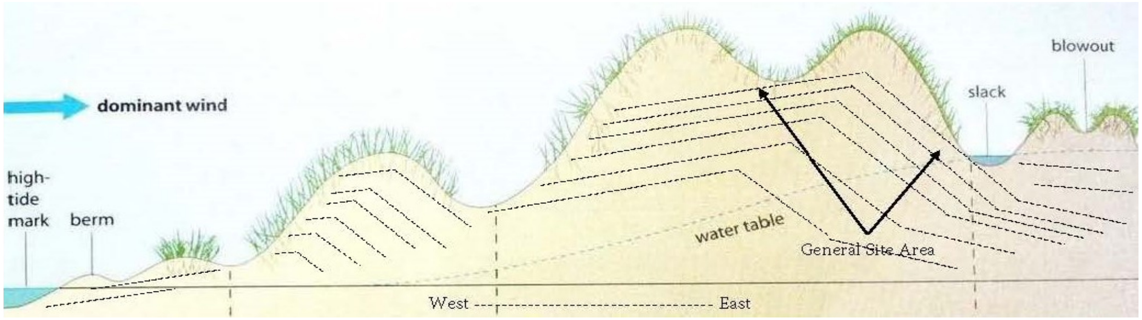
The Montessori School of Manhattan Beach consists of a relatively small one-story building and parking area located on the north end of the lower campus. The western portion of this area encroaches into the toe of the dune ridge and old stockpile fill, with changes in relief accommodated by a retaining wall(s). By 1972, a portion of the stockpile surface within the northerly third of the property had been paved (Historical Aerials, 1972). This area has remained unchanged since that time. The present-day pavement surface is extensively cracked, which may be a manifestation of long-term stockpile settlement. Dune sands have slumped and washed over the northwestern portion of this pavement.

Interpreted distribution of cut and fill conditions in plan view are shown on Plate 1, *Geotechnical Map.* Pre- and post-development topographic profiles, along with the currently conceptualized locations of proposed improvements, are illustrated in section view on Plate 2, *Geotechnical Cross Sections A-A', B-B', C-C', D-D', and E-E'.*

2.3 Geologic Setting

The site is located within the northern portion of the Peninsular Ranges geomorphic province of California, at the western margin of the Los Angeles Basin (Basin), a broad gently seaward-sloping coastal plain. The Basin is generally northwest-orientated and has dimensions of approximately 50 miles in length and 20 miles in width. It is divided geographically into southwest, central and northeasterly structural blocks based on the location of major active steeply-dipping and northwesterly trending fault zones (Poland and Piper, 1956). The Basin is bounded by the Pacific Ocean on the west and a series of elevated mountain ranges and hills on the north, northeast, east and southeast. Up to around 31,000 feet of alluvial sediments underlie the central portion of the Basin, down-warped into a synclinal trough structure. It has been the location of marine and terrestrial deposition since the early Cenozoic Period. Source of sediments has been from erosion of late Cretaceous to late Pleistocene-age sedimentary and igneous rocks that outcrop within the surrounding mountains (Yerkes, 1965).

As regionally mapped on Figure 2, this site is situated along a linear triangular-shaped landform that extends roughly continuously for at least 11 miles along the coastline between the Palos Verdes Peninsula and Playa del Rey. This ridge-shaped feature forms a geographic boundary between the Pacific Ocean on the west, and inland areas of the Los Angeles Basin to the east. The ridge is a barrier dune complex composed of poorly consolidated, well-sorted eolian (wind-blown) sands of late Holocene age. From the coast near sea level, the profile of the complex ramps/ascends landward to an apex at approximately EL. 170 feet amsl, then descends to a toe at approximately El. 80 feet amsl. This campus is located on the leeward/landward side of these dunes, as depicted on the diagrammatic (schematic) east-west profile shown below (IB Geo, 2010):



SCHEMATIC DUNE SECTION, NOT TO SCALE (LOOKING NORTH)

From west to east, following the path of prevailing on-shore winds, the internal composite sand structure consists of gently dipping topset beds that ramp up the face of the active dunes, to moderately east-dipping foreset beds along the lee face of the complex, and flatter bottomset beds further eastward. The foreset beds typically dip approximately 33 degrees, following angle-of-repose rules. Along the toe of the complex are “slack” topographically depressed areas, which

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tend to be areas of periodic storm runoff collection. Prior to development of the site and surrounding area, the slack areas tended to be a source of groundwater infiltration within the complex.

2.4 Local Geologic Units and Subsurface Conditions

No documented records of fill placement on the site were obtained as part of our literature review. Based on our review of available historic photographs and topographic maps, and subsurface conditions observed within our exploratory borings, the surface of the site is locally mantled by deposits of undocumented Artificial Fill (map symbol Afu). This fill is underlain by Holocene age deposits of eolian (wind-blown) dune sand, and at depth by Old Eolian and Sand Dune Deposits of Pleistocene age. Brief descriptions of these units are presented below, as encountered within our exploratory borings:

2.4.1. Undocumented Artificial Fill (Afu): Within our exploratory borings, deposits of artificial fill were encountered to depths up to six feet below ground surface (bgs) as the depths explored in hand augers. Therefore, deeper fill is anticipated particularly at the toe of the leeward dune slope. Encountered fill typically consisted of loose, fine-grained silty sands containing scattered pebbles/gravels, concrete and other rubble. It is possible deeper fills exist locally between unexplored locations.

Our interpretation of historical topographic maps and aerial photographs suggest fills within upper and lower campus areas are limited in lateral extent and thickness. However, the stockpile within the northern site area may contain fills up to 40 feet thick. Evaluating the presence, absence and thickness of on-site fill conditions using this indirect methodology, provides only a speculative degree of accuracy. For purposes of this report, all fill deposits on the site is considered undocumented, and unsuitable for support where occurring within the structural influence of proposed improvements.

2.4.2. Eolian Sand and Dune Deposits – Late Holocene (Qe): Directly underlying deposits of fill on this campus or exposed at the surface, are Eolian Sand and Dune Deposits of late Holocene age. These sediments typically consist of loose, well sorted, fine-grained sand (SP) that is poorly consolidated, friable, dry and prone to erosion.

2.4.3. Old Eolian Sand and Dune Deposits – Pleistocene (Qoe): Underlying deposits of Holocene dune sands as inferred from our borings LB-6 and LB-9, are Old Eolian Sand and Dune Deposits of middle to late Pleistocene age. These sediments are similar in composition and depositional environment to the younger dune sand unit, but exhibit a richer soil chroma and slightly increased density.

Copies of borings logs are presented in Appendix A, *Field Exploration*. Geotechnical conditions described on the logs represent conditions at actual exploratory excavation locations. Variations in units and lithology may occur beyond and/or between exploratory locations. In our logs, lines demarcating boundaries between geologic units and/or earth materials, where present, represent approximated boundaries, and (unless otherwise noted) actual transitions may be gradual. Table 2, below, provides a synopsis of geotechnical properties of shallow site soils, based on results of our geotechnical laboratory testing.

Table 2 – Soil Geotechnical Properties Synopsis

Parameters	Soil Properties
In-situ Moisture:	Dry to moist
In-situ Density:	Medium dense to dense
Swell/Expansion Potential:	Mostly dune sands, swell/expansion potential is **negligible**.
Collapse Potential:	Dune sands not susceptible to collapse when wetted
Strength:	Dune sand adequate to provide shallow spread footing support
Corrosivity:	**Low soluble sulfate** (≤50 ppm) and **low ferrous corrosivity**

In summary, native dune sands are not expansive and not corrosive, but are easily eroded. Results of geotechnical laboratory testing are presented in Appendix B, *Geotechnical Laboratory Testing*.

2.5 Groundwater

Groundwater was not encountered in our borings drilled on August 7th. and 8th., 2018 to the maximum explored depth of approximately 51½ feet bgs (El. ±53' amsl). Highest historic groundwater beneath the site, reported at a depth of approximately >40 feet below ground surface, is interpreted from groundwater contours noted in Seismic Hazard Zone Report No 036, for the Venice 7.5 Minute Quadrangle, Los Angeles County, California (CGS, 1998):

http://gmw.conservation.ca.gov/SHP/EZRIM/Reports/SHZR/SHZR_036_Venice.pdf

Another indication of groundwater depths beneath the site, are water levels measured approximately 430 feet northeast of the site, within Well No. 690C (State Well ID No. 3S15W24M01) maintained by the Los Angeles County

Department of Public Works. Between 1957 and 2007, groundwater within this well show an average depth of 87 feet (EL. ±6-feet amsl). An extrapolation of this data suggests groundwater occurs at a depth of approximately 79 feet beneath the site.

Groundwater conditions beneath the site can be expected to fluctuate depending upon volumes of landscape irrigation, storm water infiltration and rainfall. However, on-site geologic units (uniform and clean fine sand), are expected to exhibit a relatively high rate of percolation/infiltration, and pose no significant constraint to construction as currently planned.

3.0 SITE-SPECIFIC SEISMIC HAZARDS

Depending upon the geographic location and geologic setting of a particular site in southern California, seismic-related hazards can include surface fault rupture, strong ground shaking, seismically-induced landslides, liquefaction and/or settlement, lateral spreading, and seiches, tsunamis and flooding. The following sections discuss each of these hazards and their potential impact at the subject site.

3.1 Surface Fault Rupture

Known surface faults in the region are mapped on Figure 3, *Regional Surface Fault and Historical Seismicity Map*. Our review of available in-house literature indicates that no known active faults are mapped as crossing the site, and the site is **not** located within a designated Alquist-Priolo Earthquake Fault Zone (CGS, 1999; Bryant and Hart, 2007). A surface fault rupture hazard evaluation is therefore not mandated for this site. There is also no currently known active faults mapped within the vicinity of the site having a potential for surface fault rupture. Given an absence of the above faults, potential risk for surface fault rupture at this site is low.

3.2 Regional Faulting

The closest active faults to the site were determined using a software program contained within the United States Geological Survey (USGS) Earthquake Hazards Program (USGS, 2008c). These include the Palos Verdes Fault, Newport-Inglewood Fault Zone (NIFZ), and Santa Monica / Hollywood Fault Zones, located approximately 2.7 miles, 5.9 miles, 10.4 miles, and 13.2 miles, from the site, respectively (see Figure 4). The San Andreas Fault, which is the largest active fault in California, is located approximately 48 miles northeast of

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the site at its nearest point (off to the east of Figure 4). The State of California (CGS, 2017) has recently recommended that principle traces of the Santa Monica/Hollywood and NIFZ be zoned as active, as these faults are reportedly meet zoning criteria (Bryant and Hart, 2007). These and other nearby faults are discussed in more detail as follows:

3.2.1. <u>Palos Verdes Fault</u>: The Palos Verdes Fault is considered active and is located approximately 2.7 miles (4.3 km) southwest of the project site offshore. It forms the western offshore boundary of the Los Angeles Basin. The Palos Verdes Fault is made up of a system of three segments that collectively exhibit a complex right-lateral reverse sense of displacement (Brankman and Shaw, 2009). The modeled “right-lateral” slip rate along the zone is between 2.5 and 3.8 mm/year. The “reverse” slip rate component is between 0.26 and 0.38 mm/year. Calculated slip rates within the northern portion of the Palos Verdes Fault zone are estimated to be 0.35 mm/year reverse slip rate and 1.1 mm/year right-lateral slip rate (Brankman and Shaw, 2009). Estimated maximum moment magnitude along this fault complex is on the order of 7.1.

3.2.2. <u>Newport Inglewood Fault (NIFZ)</u>: The NIFZ is located approximately 5.9 miles (9.5 km) northeast of the project site, which is an active, zoned, northwest-trending, approximately 2- to 4-mile-wide belt of anticlinal folds and faults disrupting early Holocene to late Pleistocene-age and older deposits (Barrows, 1974). The NIFZ is characterized by trends related to right-lateral shearing at depth (Moody and Hill, 1956). The zone defines the boundary between the western basement complex of Catalina type schist and related rocks to the southwest, and the eastern basement complex of metasedimentary, metavolcanic and plutonic rocks to the northeast (Yerkes, et al., 1965). Right-lateral, strike-slip displacement of 3,000 to 5,000 feet has been measured in Lower Pliocene strata along the NIFZ (Dudley, 1954; Hill, 1954). Apparent vertical offset across faults of the NIFZ ranges from 4,000 feet at the basement interface, to 1,000 feet in the Pliocene strata, and 200 feet at the Plio-Pleistocene boundary (Yerkes, et al., 1965). Movement along this structural zone is inferred to have been initiated during middle Miocene time (circa 15-million-years ago), with seismic activity continuing to the present time (e.g. Long Beach Earthquake). There is abundant seismic evidence that the zone is tectonically active; thus, the surrounding metropolitan area is subject to certain seismic risks. At least five earthquakes of magnitude 4.9 or larger have been associated with the NIFZ since 1920 (Barrows, 1974). Estimated maximum deterministic magnitude earthquake is generally modeled between Magnitude (Mw) 6.58 and 7.2.

3.2.3. <u>Santa Monica Fault (SMFZ)</u>: The SMFZ is located approximately 10.4 miles (16.7 km) north-northwest of the site, primarily paralleling Santa Monica

Boulevard. Although not yet recognized by the State of California as a Special Studies Zone, the SMFZ is considered to be well defined, but not proven to be active. This fault zone trends east-west along the southern boundary of the Santa Monica Mountains for more than 24.8 miles (40 km) and is included as part of the Transverse Ranges Southern Boundary fault system, which consists of east-west trending, left-lateral and oblique-reverse movements along several active faults. The SMFZ consists of one or more strands, is about 40 km (24.8 miles) in length, and is one of a series of east-southeast trending reverse, left-lateral oblique-slip structures that extend more-than (>) 200 km (125 miles) across southern California and accommodate westward motion of the Transverse Ranges (Dolan et al., 1997). It has been delineated locally at depths of several-thousand feet through exploratory oil well drilling and oil field development (Wills et al., 2008).

High resolution seismic reflection profiles across the SMFZ were acquired (Pratt, et al., 1998) as part of an integrated hazard assessment of this fault, which showed a series of near vertical strike-slip faults beneath topographic scarps inferred to be caused by thrust faulting. Pleistocene or Holocene movement is postulated, but not directly proven along some upper plate secondary fault segments related to the SMFZ (Dolan et al., 2000). Recurrence interval and recency of movement along many fault segments are neither well documented nor understood, mainly because intense urbanization has modified or destroyed any surface traces of the fault (Hill et al., 1979). The Southern California Earthquake Center (SCEC) identifies the most recent rupture as Late Quaternary with intervals between events unknown. North-dip, west-slip rate across the SMFZ is estimated to vary with location along en-echelon faults to be minimally on the order of 0.6 mm/year (Dolan et. al., 2000) and as high as 3.9 to 5.9 mm/year (Davis and Namson, 1994). A deterministic estimated maximum magnitude earthquake is generally modeled between Magnitude (Mw) 6.0 and 7.0 if the entire SMFZ ruptured at once.

The City of Santa Monica Geologic Hazards map (City of Santa Monica, 2014) depicts the presence of two principal fault branches, designated within a “Fault Hazard Management Zone”. The nearest of these strands is on the order of 9.9 miles (15.9 Km) north-northwest of the project site.

3.2.4. <u>Hollywood Fault</u>: Located approximately 13.2 miles (21.3 km) north-northwest of the site, the Hollywood Fault begins near the Los Angeles River and eastern edge of the Santa Monica Mountains and extends westward for approximately 9½ miles where it is thought to shift its locus of active deformation to the area of the West Beverly Hills Lineament (WBHL), where faulting takes a left step to the Santa Monica Fault. The Hollywood Fault is capable of producing a Mw 6.4 to 6.6 earthquake (Dolan et al., 1997).

Investigators have estimated the lateral slip rate to be about 1.0 ±0.5 mm/year, with a vertical slip rate to be 0.25 mm/year (Dolan et al., 1997). Conversely, a lower slip rate of 0.04 - 0.4 mm/year (Ziony and Yerkes, 1985) leads to a long return period.

Recent detailed geologic and geotechnical studies have provided cumulative physical evidence for Holocene displacements resulting in an Alquist-Priolo Special Study Zone being established for the Hollywood Fault (CGS, 2014). Exposures identified by prior investigators (Crook and Proctor, 1992), coupled with bulk-soil radiocarbon ages provide scant evidence for an early to mid-Holocene age for the most recent surface rupture approximately 6,000 years to 11,000 years ago; suggesting a long period of quiescence between surface rupturing on the Hollywood Fault (Dolan, 1997, 2000) (Ziony and Yerkes, 1985).

3.3 Historical Seismicity

An evaluation of historical seismicity associated with significant past earthquakes, related to the site was performed, with recorded epicenters plotted on Figure 4, *Regional Surface Fault and Historical Seismicity Map*. Peak ground accelerations (PGA) at the site resulting from significant past earthquakes between 1800 to 2018, with magnitudes M4.0 or greater, were estimated using the *EQSEARCH* computer program (Blake, 2000), with 2018 updates. This historical seismicity search was performed for a 100-kilometer (62-mile) radius from the subject site.

Largest earthquake magnitude found in this search was the M7.7 earthquake that occurred on July 21, 1952 (Kern County Earthquake) approximately 83.7 miles (134.7 kilometers) north of the site, producing an estimated peak ground acceleration (PGA) near the epicenter of approximately 0.046g. Largest estimated PGA at this campus found in our search was approximately 0.098g, generated by the January 17, 1994 Northridge Earthquake approximately 23 miles (37 kilometers) north of the site. Another noteworthy event affecting the site was the magnitude 6.3 Long Beach Earthquake of March 11, 1933, which generated a PGA of 0.093g at the site. That earthquake represents the most dramatic example of consequences relating to a disregard of seismic hazards associated with the NIFZ (Richter, 1958, Barrows, 1974). The quake resulted in passage of the Field Act, which regulates construction of school buildings.

A review of additional seismic data available on the Center for Engineering Strong Motion Data (CESMD) website (http://strongmotioncenter.org/) was reviewed for stations in the vicinity of the project site. The data indicates that Station OLI (Lat. 33.9299°; long. -117.9201°) registered a peak ground acceleration of 0.145g during the M5.1 La Habra Earthquake of March 29, 2014. This earthquake occurred along the Whittier Fault approximately 28.3 miles (45.5 km) east-northeast of the site, causing minor damage throughout Los Angeles, Ventura, Orange, and San Bernardino Counties. We are unaware of any recorded earthquake damage on this campus.

3.4 Site-Specific Ground Motions (Seismic Design Coefficients)

The principal seismic hazard that could affect the site is ground shaking resulting from an earthquake occurring within any of the above-noted faults, or other major active or potentially active faults in southern California. The site is expected to experience moderate to strong ground shaking resulting from such future earthquakes.

The code-based Maximum Credible Earthquake (MCE) anticipated to affect the site, corresponds to an earthquake with a 2% probability of exceedance in 50 years. The PGA_M for the MCE was calculated at 0.610g using the United States Geological Survey (USGS) web-based Seismic Design Maps application. By deaggregating the PGA_M with respect to magnitude and distance, the modal earthquake may be assumed to be a magnitude 7.3 earthquake with a distance of approximately 2.7 miles (4.3 km) southwest of the site. Corresponding site coefficients and spectra acceleration parameters at 5% damping are presented later in this report as Table 3 in Section 4,4, *Seismic Design Parameters*.

As indicated in the 2016 CBC Section 1613A.3.5., because S_1 is less-than (<) 0.75g, structures at the site are assigned to Seismic Design Category D. As such, a site-specific ground motion study is not required (CGS Note 48).

3.5 Liquefaction and Lateral Spreading

Liquefaction is defined as a loss of soil shear strength due to a buildup of pore-water pressure in soils, when subjected to cyclic or monotonic loading by sustained severe ground shaking. Liquefaction is associated primarily with loose (low density), saturated, fine- to medium-grained, cohesionless soils, occurring within 50 feet of the ground surface. Effects of severe liquefaction can include sand boils, excessive settlement, bearing capacity failures, and lateral spreading.

As regionally mapped on Figure 4, *Seismic Hazard Zone Map*, based on the Venice Quadrangle Seismic Hazard Zone Map (CGS, March 25, 1999):

http://gmw.conservation.ca.gov/SHP/EZRIM/Maps/VENICE.pdf

and the City of Manhattan Beach Hazard Mitigation Plan (City of Manhattan Beach, 2008) indicates that the site is **not** within an area potentially susceptible to liquefaction (Figure 3, *Seismic Hazard Map*). Based on lack of shallow groundwater at the site, the liquefaction potential is considered low. Therefore, the potential for lateral spreading to occur at the site is also considered low.

3.6 Seismically-Induced Landslides

The upper and lower campus areas, where new building structures are planned, are not located in a zone of potential seismically-induced landsliding (see Figure 4). However, the leeward dune slope to the north is, and has slumped onto the paved upper terrace play area and encroaches into Sand Dune Park, predominantly as a surficial erosion and slumping process.

Although surficial erosion and slumping of sand is a concern at this campus, dune sands are homogeneous and isotropic, uniform deposits, so deep-seated instability is not a concern. Also, no known landslides exist on the site or within the nearby vicinity, based on our review of published geologic maps. A description of slopes on/or abutting the subject site is presented below:

3.6.1. **Lower Campus Areas:** As interpreted from our analysis of historical topographic maps and aerial photographs, and the PBLA topographic survey plan, the lower campus area is transected north to south by two terrace slopes, approximately 20-feet high, inferred to mainly consist of cut. These slopes have gradients of approximately 2:1 (horizontal:vertical). A 16-foot high fill slope descends to Bell Avenue along the eastern site perimeter. These slopes were covered by a dense mature landscape vegetation including large trees, which appear to offer a good measure of surficial stabilization as of August 2018.

3.6.2. **Upper Campus Areas:** Ascending from the northern boundary of the upper campus is a slope on the order of approximately 23 feet in height, inferred to be a cut slope, with 2:1 (horizontal:vertical) and locally slightly steeper ratios. The slope is underlain by Holocene-age dune sand. The surface of the slope is covered by mature landscape vegetation, including large trees. Based on the generally massive structure of the underlying sands comprising the above slopes, their 2:1 (horizontal:vertical) ratios, an absence of any mapped landslides, their landscape-stabilized surfaces, and exclusion from regulatory landslide maps, potential for seismically-induced landsliding is considered low in this area.

3.6.3. **Parcel 2/Lot 3 Area:** Bordering the northern end of the site (Parcel 2/Lot 3) is a westerly-ascending slope on the order of approximately 50 feet in height. This slope also ascends at a ratio of approximately 2:1 (horizontal:vertical) from a level undeveloped pad in this area. Our interpretation of historical aerial photos and topographic maps suggests this is a natural slope underlain by Holocene-age dune sand. A slope on the order of 36 feet in height descends from the eastern edge of the pad to Bell Avenue. We interpret the pad and east slope are underlain by stockpile material composed of locally derived sand. The stockpile fill buries, and essentially buttresses, the lower portion of the natural westerly slope. The subsurface configuration of the above conditions are noted in our cross-section E-E' on Plate 1.

As regionally mapped on Figure 4, *Seismic Hazard Zone Map*, the northern portion of the western slope and level pad **is** mapped by the state as an area of required investigation for potential seismically-induced landsliding. Mapped limits of this hazard are similar in geographic area to other similar areas within Manhattan Beach, confined to a narrow northwesterly trending area along the lee-side of the dune ridge. The Manhattan Beach Hazard Mitigation Plan (2008) indicates the probability of landsliding in these areas is low. Proposed improvements in this area include an on-grade playfield, dog park and fire access road, confined to the level pad.

Once a grading plan for this area is developed, slope stability analyses should be performed. However, deep-seated instability is not expected to be a concern for homogeneous and isotropic, uniform dune sands. Surficial stability will need to be addressed as described later in this report.

3.7 Seismically-Induced Settlement

Seismically-induced settlement consists of dry dynamic settlement (above groundwater) and liquefaction-induced settlement (below groundwater). These settlements occur primarily within loose to moderately dense sandy soil due to reduction in volume during and shortly after an earthquake event.

Soil profiles obtained from our borings are interpreted from samples taken at 5-foot depth intervals. Boring LB-6 was used as a representative soil column with N-values ranging from 14 to 31 (uncorrected. Based on evaluation of blow-count data (N-values) and our settlement calculations, total seismically-induced settlement is expected to be less-than (<)4-inches occurring relatively uniformly in these uniform sands, where confined. Accordingly, seismically-induced differential settlement is expected to be on the order of one inch over 40 feet.

3.8 Flooding

As shown on Figure 5, *Flood Hazard Zone Map*, the site is located outside the 100- and 500-year flood zone boundaries mapped by the Federal Emergency Management Agency (FEMA, 2008). Earthquake-induced flooding can also be caused by failure of dams or other water-retaining structures as a result of an earthquake. The site is located outside of any dam inundation zone, as no such structures exist near the site. Based on the above the potential for earthquake-induced flooding at the site is considered low.

3.9 Seiches and Tsunamis

Seiches are large waves generated within enclosed bodies of water in response to ground shaking. Tsunamis are sea waves generated by large-scale disturbance of the ocean floor that induce a rapid displacement of the water column above. The most frequent causes of tsunamis are shallow underwater earthquakes or submarine landslides.

Figure 6, *Tsunami Inundation Map*, shows this campus is **not** located within the tsunami run up area as mapped on the Tsunami Inundation Map for Emergency Planning, State of California, County of Los Angeles, Venice Quadrangle (CGS, 2009). Based on the site's elevation of 100 feet above sea level, protection of the sand dune and the lack of nearby enclosed water bodies, the risks associated with tsunamis and seiches are considered negligible. Accordingly, the

City of Manhattan Beach Emergency Operations Plan (City of Manhattan Beach, 2017), identifies the school site as a designated shelter for residents living in low-lying beach and coastal community areas, during a near-shore tsunami event.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 Conclusions

This site is **not** located within a currently designated Alquist-Priolo Special Studies Zone for surface fault rupture. However, as is the case for most of southern California, strong ground shaking has and will occur at this site. Groundwater is at depths greater-than (>) 40-feet; so **liquefaction is highly unlikely** to occur at this site.

4.2 Recommendations Summary

As a generalized description, this site can be characterized as sand dunes, underlain predominantly with uniform fine sands (≤5% fines silt or clay), with subsets as undocumented fill soils, more recent and older aeolian deposits. Existing fill soils should be recompacted to support new one- to two-story structures; but otherwise, undisturbed native dune sands should provide adequate support for spread footings.

We are unaware of any fill placement documentation for this site, and existing near surface fill soils are likely to be significantly disturbed during demolition. Based upon our geotechnical exploration and analysis, all existing fill soil, and soils disturbed by demolition (e.g. modular removals), within the proposed building footprints should be excavated and recompacted to provide more uniform shallow foundation support. In any case, overexcavation should extend at-least (≥) 5-feet below existing grade. This proposed one- to two-story buildings can be founded on conventional spread footings bearing solely on a zone of newly excavated and recompacted fill soils derived from onsite soils, overlying undisturbed native dune sands.

Primary concern with all dune sands, including this site, is the propensity for ongoing sand migration near the surface. Slopes should be cut and constructed no-steeper-than 2:1 (horizontal:vertical) and will require artificial stabilization of the surface with deep-rooted, drought-resistant vegetation and augmented topsoil, geogrids, geocells, cribs and/or other imported materials and systems. In summary, these cohesionless sands must be confined to provide continued support for site improvements and to mitigate blowing sand hazards.

Detailed geotechnical recommendations for proposed campus improvements are presented in the following subsections.

4.3 Earthwork

We understand that exiting improvements will be removed prior to construction of two new buildings. Project earthwork is expected to include complete demolition/removal of existing improvements and complete overexcavation and recompaction of undocumented fill soils below proposed new building footprints as described in the following subsections. We assume finish floor (FF) surface for this new classroom building will be at or near elevation 109 feet above mean sea level (msl), with the new Multi-Purpose Room (MPR) and Administration Building at elevation 100 feet above sea level.

4.3.1. Earthwork Observation and Testing: Leighton Consulting, Inc. should observe and test all grading and earthwork, to check that the site is properly prepared, the selected fill materials are satisfactory, and that placement and compaction of fills has been performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is essential. A bulk sample of any imported soil or aggregate material should be submitted to the Leighton Consulting, Inc. geotechnical laboratory at least two working days in advance of earth material placement and compaction. Project plans and specifications should incorporate recommendations contained in the text of this report.

Variations in site conditions are possible and may be encountered during construction. To confirm correlation between soil data obtained during our field and laboratory testing and actual subsurface conditions encountered during construction, and to observe conformance with approved plans and specifications, it is essential that we be retained to perform continuous or intermittent review during earthwork, excavation and foundation construction phases. Therefore, conclusions and recommendations presented in this report are contingent upon us performing construction observation services.

4.3.2. Cut and Fill Slopes: Prior to construction, surface improvements (drainage improvements irrigations lines, fencing), underground utilities and other obstructions in the area planned for construction should be removed. Vegetation in the area of construction should be removed and hauled offsite. Cut and/or fill slope grading should be in accordance with Appendix J of the 2016 *California Building Code* (CBC), including specifically:

- **2:1 (horizontal:vertical) Gradient**: Cut slopes should be cut back no steeper than 2:1 (horizontal:vertical) in dune sands or fill slopes constructed no-steeper-than 2:1 (horizontal:vertical), in accordance with

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the Section J106.1 of the CBC. Site dune sands are inherently unstable at the surface. Fill soil surfaces should be augmented to reduce surficial erosion with deep-rooted, drought-resistant vegetation and augmented topsoil, geogrids, geocells, cribs and/or other imported materials and systems.

- **Intervening Drainage Terraces**: Intervening drainage terraces should also be provided no-more-than (≤) 30 feet in vertical slope height, in accordance with Section J109.2 of the 2016 CBC. Proposed cut slopes are anticipated to be approximately 30 feet high (see Figures 2a and 2b), so intervening terraces will not be required.
- **Overflow Berms**: Interceptor berms and overflow protection should be provided in accordance with Section J109.3 of the CBC, at the top of proposed cut slopes.
- **Erosion Control**: Slope planting and erosion control must be provided in accordance with Section J110 of the 2013 CBC. Robust erosion protection is essential for surficial slope stability, which will require maintenance. Deep-rooted and drought resistant planting is suggested.

4.3.3. Surface Drainage: Water should not be allowed to pond or accumulate anywhere except in detention basins set back at least ten feet from structures. Pad drainage should be designed to collect and direct surface water away from structures to approved drainage facilities. Hardscape drains should be installed and drain to storm water disposal systems. Drainage patterns and drainpipes approved at the time of fine grading should be maintained throughout the life of proposed structures. Stormwater infiltration should not be allowed for at least ten feet, measured horizontally around any building perimeter.

We *suggest* avoiding irrigation within five-feet of any building perimeter, when possible. However, we defer to the Architect and/or Landscape Architect for design of drought-resistant planting with controlled (e.g. drip) irrigation next to the building, with grades slope away from the building at a 3% gradient or steeper. Site soils are **not** expansive, but moisture infiltration could be a concern to be addressed by the Architect and/or Landscape Architect within their areas of professional practice.

4.3.4. Site Preparation: Based on encountered site conditions, we recommend that after removal of pavements and hardscape, and complete demolition of improvements within the proposed new building footprint, then all fill and native soils should be excavated from this proposed building footprint, down at least 2 feet below the bottoms of proposed footings or at least **5 feet** below existing grade, or deeper if required to excavate existing fill soils from within proposed buildings footprints. This overexcavation bottom should extend horizontally either the thickness of fill below spread-footings or at least 5-feet horizontally beyond the outside edges of proposed perimeter footings, whichever is greater, encompassing the whole new building footprint. Any underground obstructions encountered should be removed. Efforts should be made to locate any existing utility lines. Those lines should be removed or rerouted where interfering with proposed construction. Trees to be removed should be grubbed out.

Areas outside proposed-building footprint limits, planned for asphalt and/or concrete pavement, should be over-excavated to a minimum depth of 24-inches below existing or finish grade, or 18-inches below proposed pavement sections; whichever is deeper.

Resulting removal excavation bottom-surfaces should be observed by Leighton Consulting, Inc., prior to placement of any backfill or new construction. **It is essential that all existing fill soils be excavated from proposed new building footprints, regardless of depth.** After these over-excavations are completed, and prior to fill placement, exposed surfaces should be scarified to a minimum depth of 6 inches, moisture-conditioned to or slightly above optimum moisture content, and recompacted to a minimum 90 percent relative compaction as determined by ASTM D 1557 standard test method (modified Proctor compaction curve).

4.3.5. Reuse of Concrete and Asphalt in Fill: Pulverized demolition concrete free of rebar and other materials and demolished asphalt pavement can be pulverized to particles no-larger-than (≤) 3-inches, and mixed with site soils for use in compacted fill. Blended pulverized concrete and asphalt should be mixed with at least 25% soils by weight. Such materials must be free of and segregated from any hazardous materials and/or organic material of any kind.

4.3.6. Fill Placement and Compaction: Onsite soils **free of** organics, debris and oversized material (greater-than 3-inches in largest dimension) are suitable for use as compacted structural fill. However, any soil to be placed as fill, whether onsite or imported material, should be first viewed by Leighton Consulting, Inc., and then tested if and as necessary, prior to approval for use as compacted fill. All structural fill must be free of hazardous materials.

All fill soil should be placed in thin, loose lifts, moisture-conditioned, as necessary, to within 3 percent above optimum moisture content, and compacted to a minimum **95% relative compaction** as determined by ASTM D 1557 standard test method (modified Proctor compaction curve) within the building footprint. Aggregate base for pavement sections should be compacted to a minimum of 95% relative compaction.

4.3.7. Pipeline Backfilling: Pipeline trenches should be backfilled with compacted fill in accordance with this report, and applicable *Standard Specifications for*

Public Works Construction (Greenbook), 2015 Edition standards. Backfill in and above the pipe zone should be as follows:

- **Pipe Zone**: Pipe bedding zone should be backfilled with Controlled Low Strength Material (CLSM) consisting of at least one sack of Portland cement per cubic-yard of sand, conforming to Section 201-6 of the 2015 Edition of the *Standard Specifications for Public Works Construction* (Greenbook). Imported clean/uniform sand with a Sand Equivalent (SE) greater-than-or-equal-to (≥) 30 can also be used in the pipe zone. CLSM or uniform sand bedding should be placed to 1-foot (0.3 m) over the top of the conduit, and vibrated. CLSM should **not** be jetted but sand should be flooded and jetted.

- **Over Pipe Zone**: Above the pipe zone, trenches can be backfilled with excavated on-site soils free of debris, organic and oversized material greater-than (>) 3-inches in largest dimension. As an option, the whole trench can be backfilled with one-sack CLSM same as presented above for the pipe bedding zone. Oversized rock (cobbles and/or boulders) should either be removed from any backfill, or pulverized for use in backfill only above the pipe zone. Gravel larger than ¾-inch in diameter should be mixed with at least 80-percent soil by weight passing the No. 4 sieve. Native soil backfill over the pipe-bedding zone should be placed in thin lifts, moisture conditioned, as necessary, and mechanically compacted using a minimum standard of 90% relative compaction (relative to the laboratory modified Proctor maximum **dry** density), relative to the ASTM D 1557 laboratory maximum dry density within the building footprint and hardscape areas, or 85% under landscape areas. In any case, backfill above the pipe zone (bedding) should be observed and tested by Leighton Consulting, Inc.

4.4 Seismic Design Parameters

To accommodate effects of ground shaking produced by regional seismic events, seismic design can, at the discretion of the designing Structural Engineer, be performed in accordance with the 2016 Edition of the California Building Code (CBC). Table 2 (below), lists seismic design parameters based on the 2016 CBC and ASCE 7-10 methodologies:

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Table 3 – 2016 CBC Site-Specific Seismic Parameters

2016 CBC Site-Specific Seismic Design Parameters	Value
Site Longitude (decimal degrees) West	-118.411
Site Latitude (decimal degrees) North	33.895
Site Class Definition (2016 CBC 1613A.3.2 and ASCE 7-10)	D
Mapped Spectral Response Acceleration at 0.2s Period, S_s (Figure 1613.3.1(1))	1.622
Mapped Spectral Response Acceleration at 1s Period, S_1 (Figure 1613.3.1(2))	0.609
Short Period Site Coefficient at 0.2s Period, F_a (Table 1613A.3.3(1))	1.0
Long Period Site Coefficient at 1s Period, F_v (Table 1613A.3.3(2))	1.5
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS} (Eq. 16A-37)	1.622
Adjusted Spectral Response Acceleration at 1s Period, S_{M1} (Eq. 16A-38)	0.914
Design Spectral Response Acceleration at 0.2s Period, S_{DS} (Eq. 16A-39)	1.082
Design Spectral Response Acceleration at 1s Period, S_{D1} (Eq. 16A-40)	0.609
Seismic Design Category (1613A.3.5, S_1<0.75, Risk Category III)	D
Long Period (T_L)	8

Derived from the USGS web page: http://earthquake.usgs.gov/designmaps/us/application.php
All coefficients in units of g (spectral acceleration).
S_{D1} < 0.75g, so a site-specific ground motion evaluation is not required.

4.5 Foundations

Based on our preliminary exploration and our experience in the region, conventional shallow spread footings/mats may be used to support this proposed two new buildings. Anticipated foundation loads were not available during preparation of this report. However, we assumed maximum column dead loads up to (≤) 250 kips and bearing wall loads of 3 kips-per-lineal-foot for our preliminary foundation recommendations. Overexcavation and recompaction of footing subgrade soils should be performed as detailed in Section 4.3 of this report. Specific spread footing recommendations are presented below:

4.5.1. Minimum Embedment and Width: Based on our preliminary exploration, footings for this proposed building should have a minimum embedment of 24-inches below lowest adjacent exterior grade (to reduce the potential for dune sand migration) or 18-inches below interior finished grade; whichever is deeper/lower. Minimum footings widths should be at least 24-inches for isolated rectangular column footings or 12-inches for continuous bearing wall (strip) footings.

4.5.2. Allowable Bearing Capacity: A net allowable bearing capacity of 2,500 pounds-per-square-foot (psf) may be used for design of continuous wall footings or 3,000 pounds-per-square-foot (psf) may be used for design of isolated rectangular column footings. These values are based on the

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minimum embedment depth and width recommended in Section 4.5.1, above, and are governed by properly compacted fill settlement. These allowable bearing values may be increased by 300 psf per foot increase in embedment-depth and/or width to a maximum allowable bearing pressure of 5,000 psf, and are for total dead load and sustained live loads, which can be increased by one-third when considering short-duration wind or seismic loads. Footing reinforcement should be designed by the project Structural Engineer.

4.5.3. Lateral Load Resistance: Soil resistance available to withstand lateral loads on a shallow foundation is a function of the frictional resistance along the base of the footing and the passive resistance that may develop as the face of the structure tends to move into the soil. The frictional resistance between the base of the foundation and the subgrade soil may be computed using a coefficient of friction of 0.4. The passive resistance may be computed using an equivalent fluid pressure of 300 pounds-per-cubic-foot (pcf), assuming there is constant contact between the footing and undisturbed soil. These friction and passive values have already been reduced by a factor-of-safety of 1.5, and can be increased by one-third when considering short-duration wind or seismic loads. For spread footings and slabs-on-grade bearing on properly compacted fill over undisturbed native soils, full friction and passive resistance can be combined to resist lateral loads; although some lateral displacement is required to mobilize full passive resistance.

4.5.4. Uplift Load Resistance: If required to resist seismic uplift loads, properly compacted backfill soils over spread footings can be used, modeled with both dead weight and soil shear strength resisting short term dynamic uplift forces. Properly compacted backfill soils may be assumed to have a moist unit weight of 120 pounds-per-cubic-foot (pcf). A friction angle of 35° can be used to model properly compacted backfill soil's shear strengths. A factor-of-safety has not been applied to these values.

4.5.5. Settlement Estimates: The above recommended allowable bearing capacity is generally based on a total allowable, post-construction total settlement of 1 inch, for column loads and wall loads not exceeding 250 kips and 3 kips-per-foot, respectively, for dead plus sustained live loads. Differential settlement due to static loading is generally estimated at ½ inch over a horizontal distance of 30 feet. Once developed by the Structural Engineer, we can review total dead and sustained live loads for each column including plan location and span distance, to evaluate if differential settlements between dissimilarly loaded columns will be tolerable. Excessive differential settlement can be mitigated with the use of reduced bearing pressures, deeper footing embedment, possibly changing overexcavation schemes and using imported base material under spread footings, or possibly other methods. Assuming all existing fill soils are properly recompacted below these buildings, dynamic differential settlement in dense sands is expected to be negligible.

4.6 Retaining Wall Design

4.6.1. Design Static Lateral (Horizontal) Earth Pressures: On-site sands are considered suitable to be used as retaining wall backfill. Should import soils be used for backfilling against retaining walls they should be tested to check that the Expansion Index (EI) is less-than (<) 20. Recommended lateral earth pressures for retaining walls backfilled with dune sands with drained conditions as follows:

Table 4 – Earth Retaining Design Coefficients

Condition	Level Backfill
Active (K_a)	0.30
At-Rest (K_0)	0.48
Passive (K_p)	3.25
Coefficient of Friction	0.4
Seismic Increment (K_e)	0.28
Unit Weight (pcf)	120

These values do not contain an appreciable factor-of-safety, so the Structural Engineer should apply applicable factors-of-safety and/or load factors during design. Cantilever walls that are designed to yield at least 0.001H, where H is equal to the wall height, may be designed using the active condition. Rigid walls and walls braced at the top should be designed using the at-rest condition, which is expected to be the case for vaults and elevator shafts. Passive pressure is used to compute soil resistance to lateral structural movement.

Total depth of retained earth for design of walls and for uplift resistance should be measured as the vertical height of the stem below the ground surface at the wall face for stem design, or measured at the heel of the footing for overturning and sliding. A total unit weight of 120 pounds-per-cubic-foot (pcf) may be assumed to calculate weight of compacted fill soil over wall footings, if properly compacted and drained.

4.6.2. Retaining Wall Surcharges: In addition to the above lateral forces due to retained earth, surcharge due to above grade loads on the wall backfill, such as traffic, should be considered in design of retaining walls. Vertical surcharge loads behind a retaining wall on or in backfill within a 1:1 (horizontal:vertical) plane projection up and out from the retaining wall toe, should be considered as lateral and vertical surcharge. Unrestrained (cantilever) retaining walls should be designed to resist one-third of these surcharge loads applied as a uniform horizontal pressure on the wall. Braced walls should also be designed to resist an additional uniform horizontal-pressure equivalent to one-half of uniform vertical surcharge-loads.

In areas where autos and pickup trucks will drive, we suggest assuming a uniform vertical surcharge of 300 psf, which would result in active and at-rest horizontal surcharges of 100 psf and 150 psf, respectively. This should be doubled in areas of heavy construction traffic (such as concrete trucks, heavy equipment delivery-trucks, etc.). If crane outrigger loads or other point load sources are applied as wall surcharge, this will require additional analyses based on load source and location relative to the wall.

4.6.3. Retaining Wall Incremental Seismic Loads: Seismic incremental loads need **not** be added to retaining walls with stem heights on the order of (≤) 6-feet or less, with adjacent level backfill. However, at the discretion of the project Structural Engineer (SE), incremental seismic earth pressures of 20 pounds-per-cubic-foot (pcf) may be applied for design in addition to static active earth and surcharge pressures presented above. This is based on traditional Mononobe-Okabe (1929) equations. Traditionally, this incremental seismic earth pressure has been applied as an inverted triangle (inverted equivalent fluid pressure), with largest dynamic earth pressure occurring at the top of the wall (upper ground surface). Resultant seismic earth pressure force has traditionally been applied at approximately 0.6H from the bottom of the wall, where H is the wall (stem) height (e.g. Seed and Whitman, 1970).

However, recent studies (Sitar, et. al., 2010, U.C. Berkeley) suggest a uniform pressure distribution is likely closer to actual lateral seismic loads, so a uniform pressure of 10H (psf) applied as a uniform/rectangular pressure distribution can also be considered (based on current research and observations). It is important to consider that for level backfill and in areas without shallow groundwater, both case history reviews and centrifuge test results suggest all of these approaches above are conservative, particularly for retaining walls with modest heights.

4.7 Concrete Slab-On-Grade

Concrete slabs-on-grade should be designed by the structural engineer in accordance with 2016 CBC requirements. More stringent requirements may be required by the structural engineer and/or architect; however, slabs-on-grade should have the following minimum recommended components:

- **Subgrade:** Slab-on-grade subgrade soil should be moisture conditioned to or within 3% over optimum moisture content, to a minimum depth of 24 inches within building footprints, and compacted to 95% of the modified Proctor (ASTM D1557) laboratory maximum density prior to placing either a moisture barrier, steel and/or concrete.

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- **Moisture Barrier:** A moisture barrier consisting of at least 15-mil-thick Stego-wrap vapor barriers (see: http://www.stegoindustries.com/products/stego_wrap_vapor_barrier.php), or equivalent, should then be placed below slabs where moisture-sensitive floor coverings or equipment will be placed.

- **Reinforced Concrete**: A conventionally reinforced concrete slab-on-grade with a thickness of at least 4-inches should be placed in pedestrian areas without heavy loads. Reinforcing steel should be designed by the structural engineer, but as a minimum should be No. 4 rebar placed at 30-inches on-center, each direction (perpendicularly), mid-depth in the slab. A modulus of subgrade reaction (k) as a linear spring constant, of 200 pounds-per-square-inch per inch deflection (pci) can be used for design of heavily loaded slabs-on-grade, assuming a linear response up to deflections on the order of ¾-inch.

- **Slab-On-Grade Control Joints**: Slab-on-grade crack control joint locations and spacing should be designed by the project Structural Engineer (SE). However, consideration should be given to potential for differential-vertical-offset at control joints, due to structure settlement. Where possible, slabs-on-grade should be allowed to “float” on the subgrade to allow for differential vertical movement. Interior full-depth joints at wall and column interfaces are recommended to allow the slab-on-grade to “float” unrestrained by vertical structural components. However, doweling is recommended at other joints in open areas of rooms to avoid trip hazards.

Minor cracking of concrete after curing due to drying and shrinkage is normal and should be expected. However, cracking is often aggravated by a high water-to-cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. The use of low-slump concrete or low water/cement ratios can reduce the potential for shrinkage cracking.

4.8 Sulfate Attack and Ferrous Corrosion Protection

4.8.1. **Sulfate Exposure**: Sulfate ions in the soil can lower the soil resistivity and can be highly aggressive to Portland cement concrete by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. A potentially high sulfate content could also cause corrosion of reinforcing steel in concrete. Section 1904A of the 2016 California Building Code (CBC) defers to the American Concrete Institute’s (ACI’s) ACI 318-14 for concrete durability requirements. Table 19.3.1.1 of ACI 318-14 lists “*Exposure categories and classes*,” including sulfate exposure as follows:

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Table 5 – Sulfate Concentration and Exposure

Soluble Sulfate in Water (parts-per-million)	Water-Soluble Sulfate (SO_4) in soil (percentage by weight)	ACI 318-14 Sulfate Class
0-150	0.00 - 0.10	S0 (negligible)
150-1,500	0.10 - 0.20	S1 (moderate*)
1,500-10,000	0.20 - 2.00	S2 (severe)
>10,000	>2.00	S3 (very severe)

*or seawater

4.8.2. <u>Ferrous Corrosivity</u>: Many factors can modify corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor. Based on the findings of studies presented in ASTM STP 1013 titled "*Effects of Soil Characteristics on Corrosion*" (February 1989), the approximate relationship between soil resistivity and soil corrosiveness was developed as follows:

Table 6 – Soil Resistivity and Soil Corrosivity

Soil Resistivity (ohm-cm)	Classification of Soil Corrosiveness
0 to 900	Very Severely Corrosive
900 to 2,300	Severely Corrosive
2,300 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
10,000 to >100,000	Very Mildly Corrosive

Acidity is an important factor of soil corrosivity. The lower the pH (the more acidic the environment), the higher the soil corrosivity will be with respect to buried metallic structures and utilities. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures, due to protective surface films, which form on steel in high pH environments. A pH between 5 and 8.5 is generally considered relatively passive from a corrosion standpoint. Chloride and sulfate ion concentrations, and pH appear to play secondary roles in modifying corrosion potential. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried steel or reinforced concrete structures.

4.8.3. <u>Corrosivity Test Results</u>: To evaluate corrosion potential of soils sampled from this site, we tested a bulk soil sample for soluble sulfate content, soluble chloride content, pH and resistivity. Results of these tests are summarized below:

Table 7 – Results of Corrosivity Testing

Boring Number	Sample Depth (feet)	Sulfate (mg/kg)	Chloride (mg/kg)	pH	Minimum Resistivity (ohm-cm)
LB-3	0 to 5	**50**	61	6.7	**6,400**
HA-3	2½ to 5	**41**	10	7.9	**17,000**

Note: mg/kg = milligrams per kilogram, or parts-per-million (ppm)

These results are discussed as follows:

- **Sulfate Exposure**: Based on our previous experience and Table 19.3.1.1 of ACI 318-14, in our opinion, sulfate exposure should be considered "**negligible**" with an **Exposure Class S0** for native silty sands sampled at the site. Based on Table 19.3.2.1 of ACI 318-14, for this **Exposure Category S0**, there would be **no restrictions** on cement type ("cementitious material") nor water/cement ratio; an f_c' (28-day compressive strength) of at-least (≥) 2,500 pounds-per-square-inch (psi) is required at a minimum for structural concrete.
- **Ferrous Corrosivity**: As shown above, minimum soil resistivity of 6,400 ohm-centimeters was measured in our laboratory test. In our opinion, based on resistivity correlation presented in Table 6, it appears for site soils that corrosion potential to buried steel may be characterized as "**mildly corrosive**" at the site. As standard design concepts, ferrous pipe buried in moist to wet site earth materials should be avoided by using high-density polyethylene (HDPE) or other non-ferrous pipe when possible. Or ferrous pipe can be protected by polyethylene bags, tap or coatings, di-electric fittings or other means to separate the pipe from on-site earth materials.

4.9 Pavement Section Design

Based on design procedures outlined in the current Caltrans *Highway Design Manual* and an assumed design R-value of 30 for silty sand subgrade variations, preliminary flexible pavement sections were calculated for the Traffic Indices (TIs) tabulated, and are listed below:

Table 8 – Hot Mixed Asphalt (HMA) Pavement Sections

Assumed Traffic Index	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
4.0 (automobile parking)	3	4
5.0 (driveways and truck traffic)	3	6
6.0 (roadways and heavy truck traffic)	3½	8

In areas of past pavement failure to be rehabilitated, placement of geogrids below the aggregate base layer is recommended, to reduce pavement distress and enhance pavement life cycle.

For fire truck (60,000-pound "apparatus") lanes, asphalt pavements designed for a TI=6.0 are recommended. However, note that undistributed apparatus outrigger loads could cause local asphalt pavement punching damage. When possible, outrigger loads should be distributed over asphalt pavements with planks and plywood. Otherwise, areas where outrigger loads are anticipated could be paved with 8-inch-thick concrete as described below.

Portland cement concrete pavement sections were calculated in accordance with procedures developed by the Portland Cement Association. Concrete paving sections for three Traffic Indices (TIs) are presented below:

Table 9 – Portland Cement Concrete Pavement Sections

Assumed Traffic Index	PC Concrete (inches)	Base Course (inches)
4.0 (automobile parking)	6	4
5.0 (driveways and truck traffic)	7	
6.0 (roadways and heavy truck traffic)	8	

We have assumed that this Portland cement concrete will have a compressive strength of at least 3,000 pounds-per-square-inch (psi). Prior to placement of aggregate base, subgrade soils should be scarified to a minimum depth of 8-inches, moisture-conditioned, as necessary, and recompacted to a minimum of 90 percent relative compaction, determined in accordance with ASTM D 1557 modified Proctor laboratory maximum density. Aggregate base should be placed in thin lifts; moisture conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction. Field observation and periodic testing, as needed during placement of base course materials, should be undertaken to ensure that requirements of Caltrans' *Standard Specifications* (2015) and Special Provisions are fulfilled. Consideration should be given to reinforce concrete pavements where large outrigger point loads are anticipated.

Adequate drainage (both surface and subsurface) should be provided such that the subgrade soils and aggregate base materials are not allowed to become wet. All pavement construction should be performed in accordance with the Caltrans *Standard Specifications* (2015). Recommended structural pavement materials should conform to the specified provisions in the Caltrans *Standard Specifications* (2015) including grading and quality requirements, shown below:

- **Asphalt Concrete (Hot Mixed Asphalt)** for pavement should be Type A and should conform to Section 39 of the *Standard Specifications*. Asphalt concrete specimens should be tested for surface abrasion in accordance with CT-360.
- **Portland Cement Concrete (PCC)** pavement should conform to Section 40 of the *Standard Specifications*. PCC pavement materials (pavement, structures, minor concrete) should conform to Section 90 of the *Standard Specifications*.
- **Class II Aggregate Base (AB)** should conform to Section 26 of the *Standard Specifications*.

Traffic Indices (TIs) used in our pavement design are considered reasonable values for typical parking lot areas, and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. Irrigation adjacent to pavements, without a deep curb or other cutoff to separate landscaping from the paving, will result in premature pavement failure. Traffic parameters used for design were selected based on engineering judgment and not on information furnished to us such as an equivalent wheel-load analysis or a traffic study.

5.0 CONSTRUCTION CONSIDERATIONS

5.1 Trench Excavations

Based on our field observations, caving of cohesionless and loose fill soils will likely be encountered in unshored trench excavations. To protect workers entering excavations, excavations should be performed in accordance with OSHA and Cal-OSHA requirements, and the current edition of the California Construction Safety Orders, see:

http://www.dir.ca.gov/title8/sb4a6.html

Contractors should be advised that sand and fill soils should be considered Type C soils as defined in the California Construction Safety Orders. As indicated in Table B-1 of Article 6, Section 1541.1, Appendix B, of the California Construction Safety Orders, excavations less-than (<) 20 feet deep within Type C soils should be sloped back no steeper than 1½:1 (horizontal:vertical), where workers are to enter the excavation. This may be impractical near adjacent existing utilities and structures; so shoring may be required depending on trench locations. Stiff undisturbed native clays will stand steeper.

During construction, soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor is responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Close coordination between the competent person and Leighton Consulting, Inc. should be maintained to facilitate construction while providing safe excavations.

5.2 Temporary Shoring

Temporary cantilever shoring can be designed based on the active equivalent fluid pressure of 30 pounds-per-cubic-foot (pcf) in alluvium. If excavations are braced at the top and at specific depth intervals, then braced earth pressure may be approximated by a uniform rectangular soil pressure distribution. This uniform pressure expressed in pounds-per-square-foot (psf), may be assumed to be 20 multiplied by H for design, where H is equal to the depth of the excavation being shored, in feet. These recommendations are valid only for trenches not exceeding 15 feet in depth at this site.

5.3 Geotechnical Services During Construction

Our geotechnical recommendations provided in this report are based on information available at the time the report was prepared and may change as plans are developed. Additional geotechnical exploration, testing and/or analysis may be required based on final plans. Leighton Consulting, Inc. should review site grading, foundation and shoring (if any) plans when available, to comment further on geotechnical aspects of this project and check to see general conformance of final project plans to recommendations presented in this report.

Leighton Consulting, Inc. should be retained to provide geotechnical observation and testing during excavation and all phases of earthwork. Our conclusions and recommendations should be reviewed and verified by us during construction and revised accordingly if geotechnical conditions encountered vary from our findings and interpretations. Geotechnical observation and testing should be provided:

- During all excavation,
- During compaction of all fill materials,
- After excavation of all footings and prior to placement of concrete,
- During utility trench backfilling and compaction,
- During pavement subgrade and base preparation, and/or

- If and when any unusual geotechnical conditions are encountered.

6.0 LIMITATIONS

This report was necessarily based in part upon data obtained from a limited number of observances, site visits, soil samples, tests, analyses, histories of occurrences, spaced subsurface explorations and limited information on historical events and observations. Such information is necessarily incomplete. The nature of many sites is such that differing characteristics can be experienced within small distances and under various climatic conditions. Changes in subsurface conditions can and do occur over time. This exploration was performed with the understanding that this subject site is proposed for development as described in Section 1.2 of this report. Please also refer to Appendix C, GBA's *Important Information About This Geotechnical-Engineering Report*, presenting additional information and limitations regarding geotechnical engineering studies and reports.

Until reviewed and accepted by the California Geological Survey (CGS), this report may be subject to change. Changes may be required as part of the CGS review process. Leighton Consulting, Inc. assumes no risk or liability for consequential damages that may arise due to design work progressing before this report is reviewed and accepted by CGS.

This report has been prepared for the express use of Manhattan Beach Unified School District and its design consultants, and only as related expressly to the assessment of the geotechnical constraints of developing the subject site and for construction purposes, in accordance with generally accepted geotechnical engineering practices at this time in California for public schools. This report is not authorized for use by, and is not to be relied upon by, any party except the District and their design and construction management team, with whom Leighton Consulting, Inc. has contracted for this work. Use of or reliance on this report by any other party is at that party's risk. Unauthorized use of or reliance on this report constitutes an agreement to defend and indemnify Leighton Consulting, Inc. from and against any liability which may arise as a result of such use or reliance, regardless of any fault, negligence, and/or strict liability of Leighton Consulting, Inc..

Anyone using this report for bidding or construction purposes should perform such independent studies as they deem necessary to satisfy themselves as to the surface and/or subsurface conditions to be encountered and means and methods of construction to be used in performance of work on this campus.

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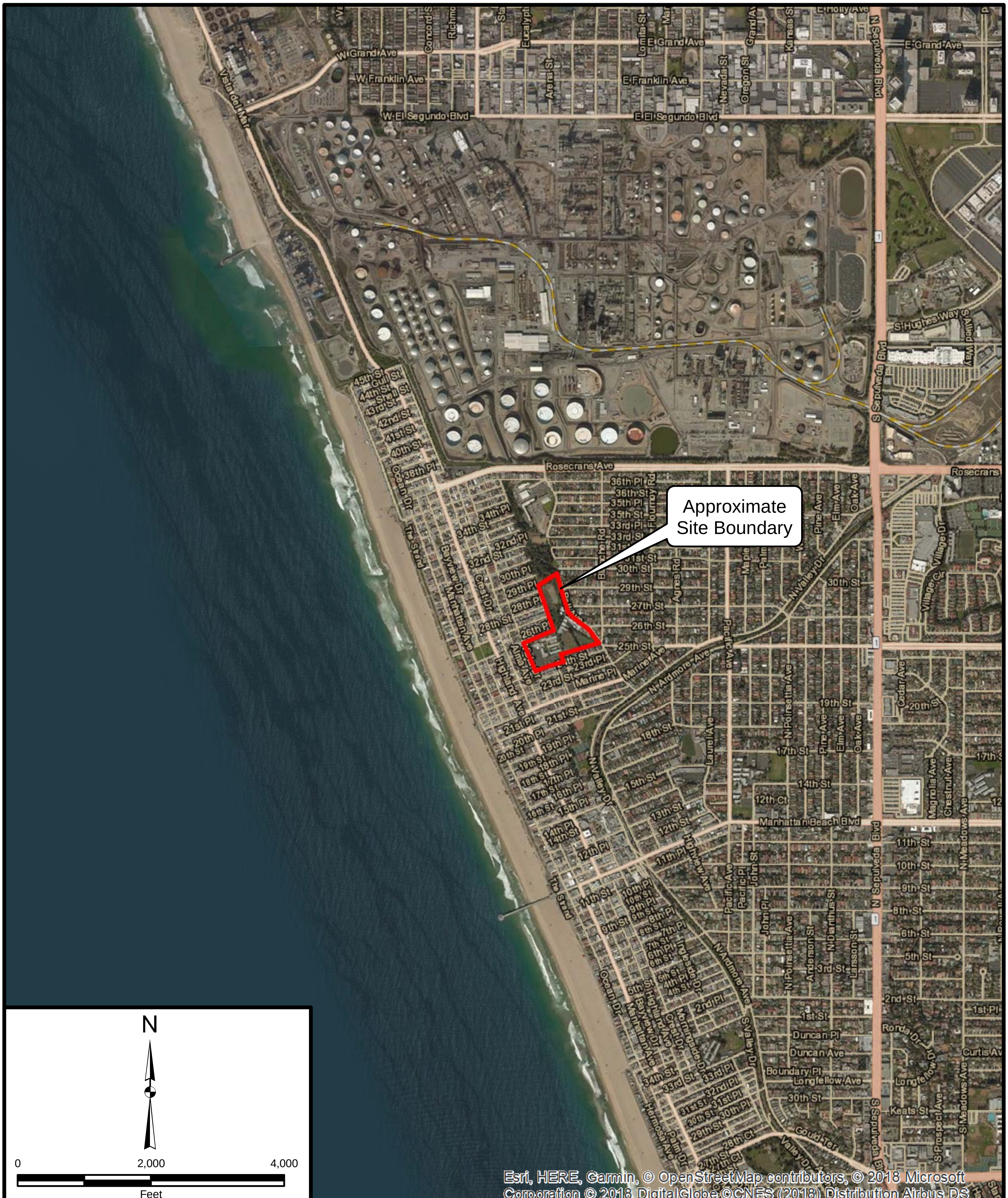

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Scale:1 " = 2,000 '	Date: September 2018

Base Map: ESRI ArcGIS Online 2018
Thematic Information: Leighton
Author: Leighton Geomatics (btran)

SITE LOCATION MAP

Grandview Elementary School
455 24th. Street
Manhattan Beach, Los Angeles County, California

Figure 1

Leighton

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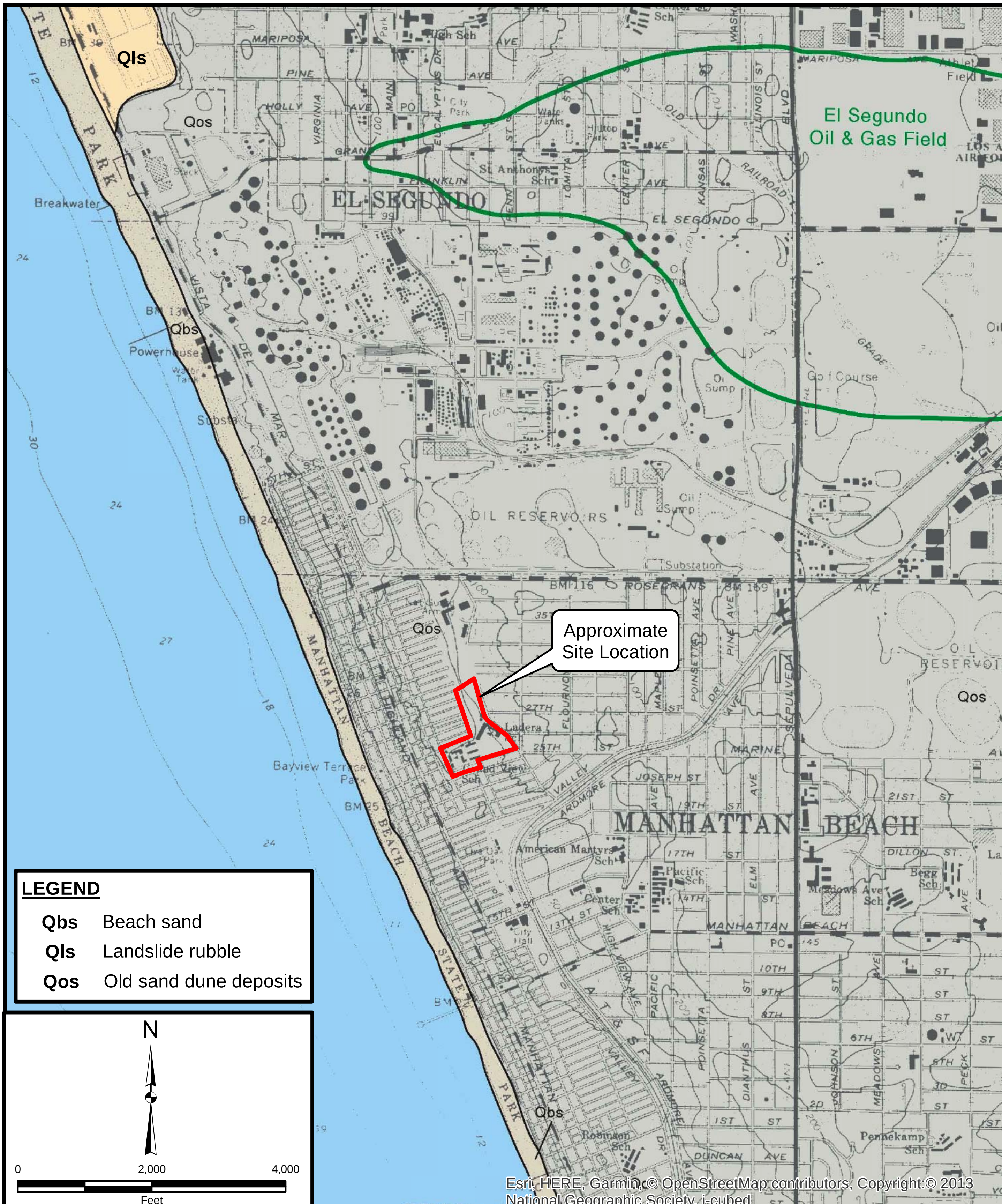

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Venice and Inglewood Quadrangles
Author: Leighton Geomatics (btran)

REGIONAL GEOLOGY MAP

Grandview Elementary School
455 24th. Street
Manhattan Beach, Los Angeles County, California

Figure 2

Leighton

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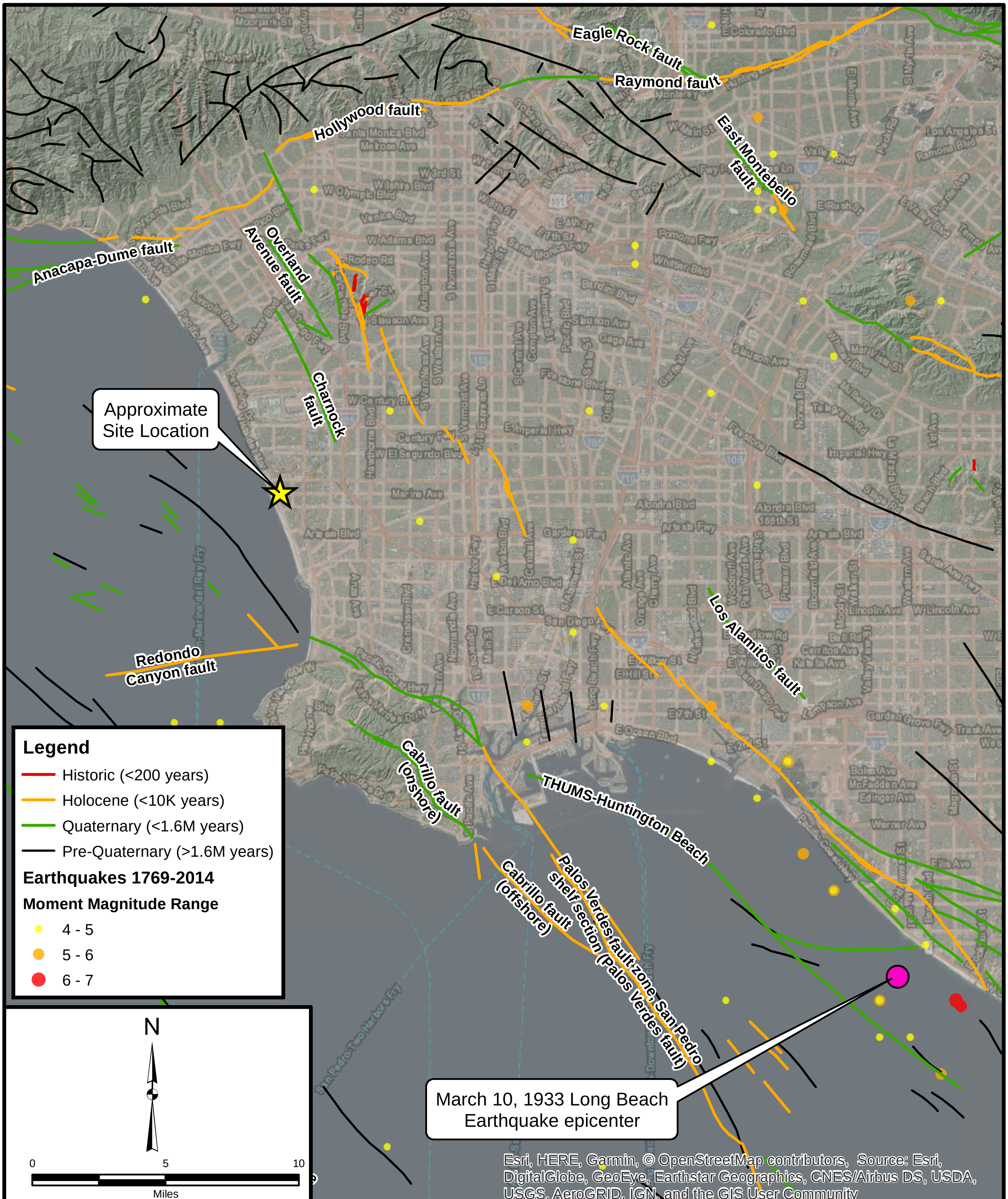

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Author: Leighton Geomatics (btran)

REGIONAL SURFACE FAULT AND HISTORICAL SEISMICITY MAP

Grandview Elementary School
455 24th. Street
Manhattan Beach, Los Angeles County, California

Figure 3

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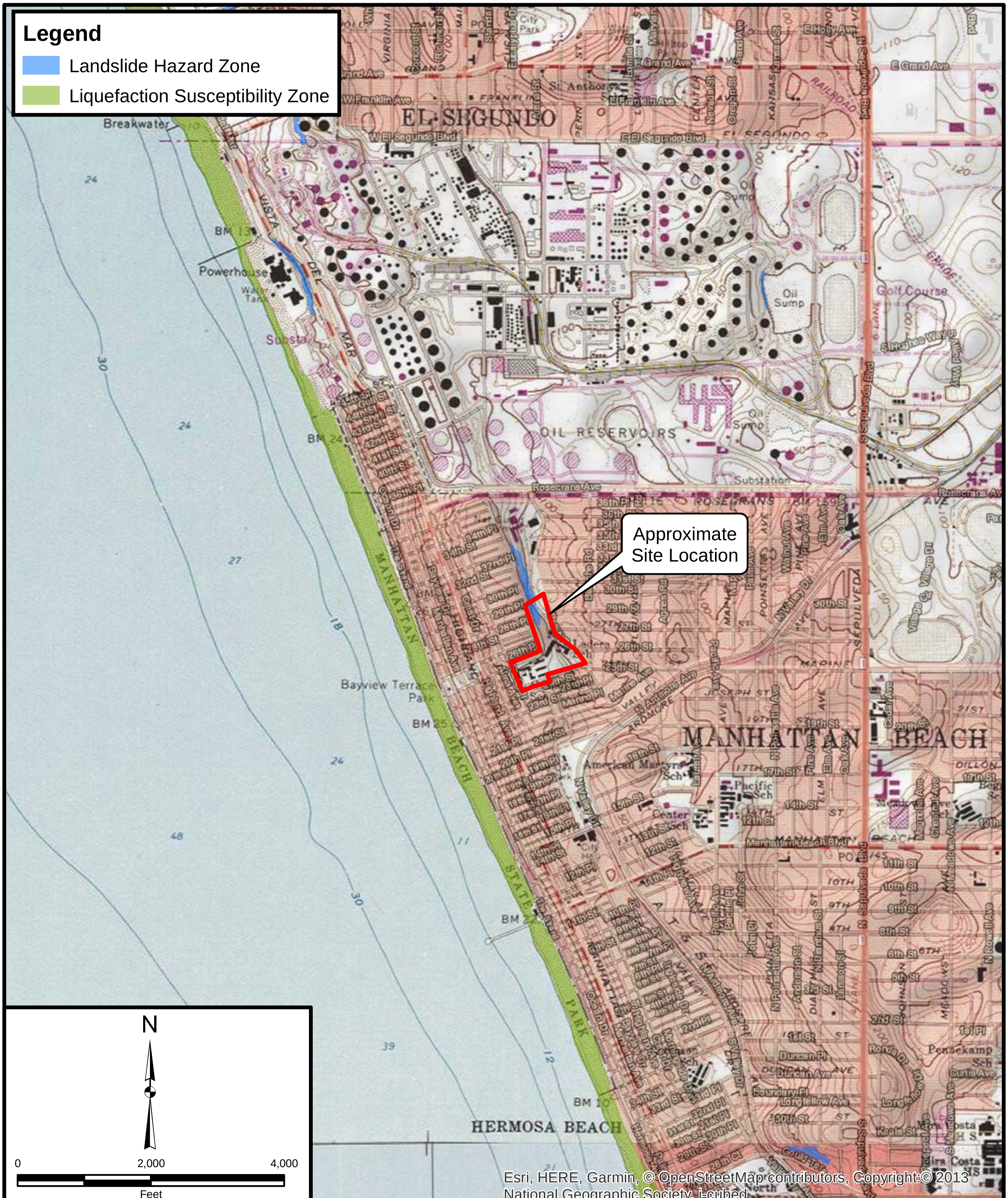

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Thematic Information: Leighton, CGS
Author: Leighton Geomatics (btran)

SEISMIC HAZARD ZONE MAP

Grandview Elementary School
455 24th. Street
Manhattan Beach, Los Angeles County, California

Figure 4

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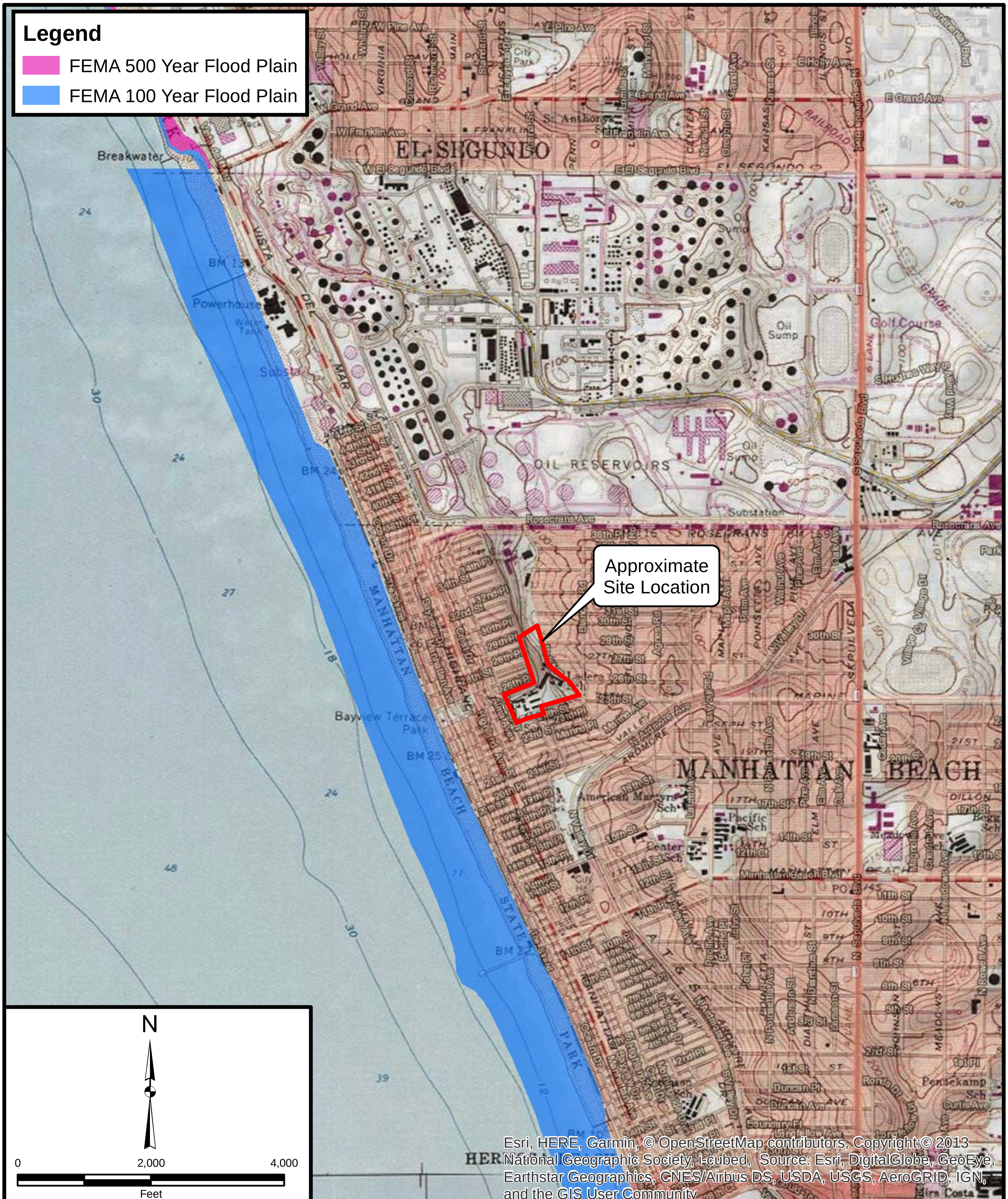

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Base Map: ESRI ArcGIS Online 2018
Thematic Information: Leighton, CA DWR, FEMA
Author: Leighton Geomatics (btran)

FLOOD HAZARD ZONE MAP

Grandview Elementary School
455 24th. Street
Manhattan Beach, Los Angeles County, California

Figure 5

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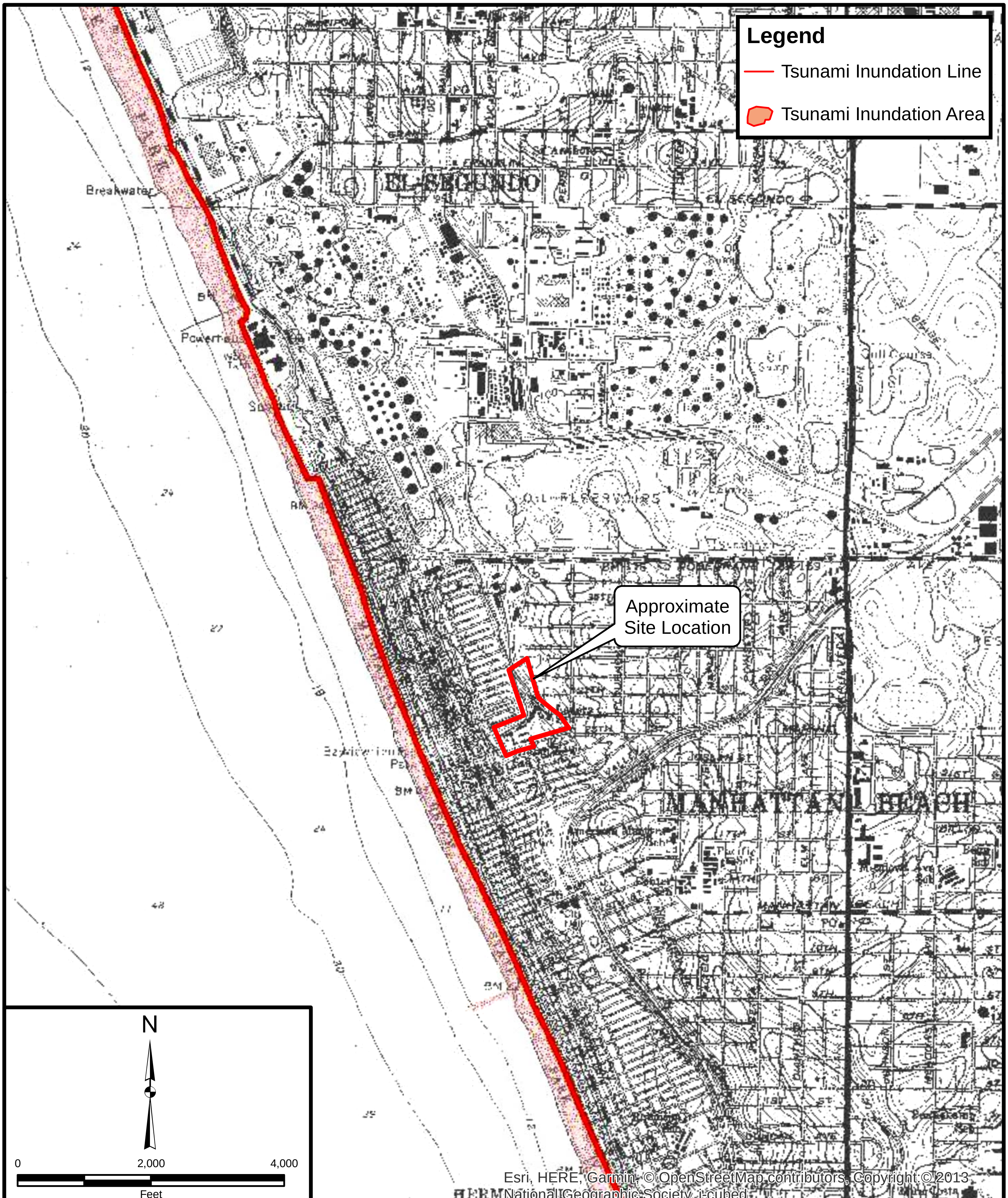

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Thematic Information: Tsunami Inundation Map, Venice Quadrangle, March 1, 2009
Author: Leighton Geomatics (btran)

TSUNAMI INUNDATION MAP

Grandview Elementary School
455 24th. Street
Manhattan Beach, Los Angeles County, California

Figure 6

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Plate 1

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GEOTECHNICAL EXPLORATION MAP

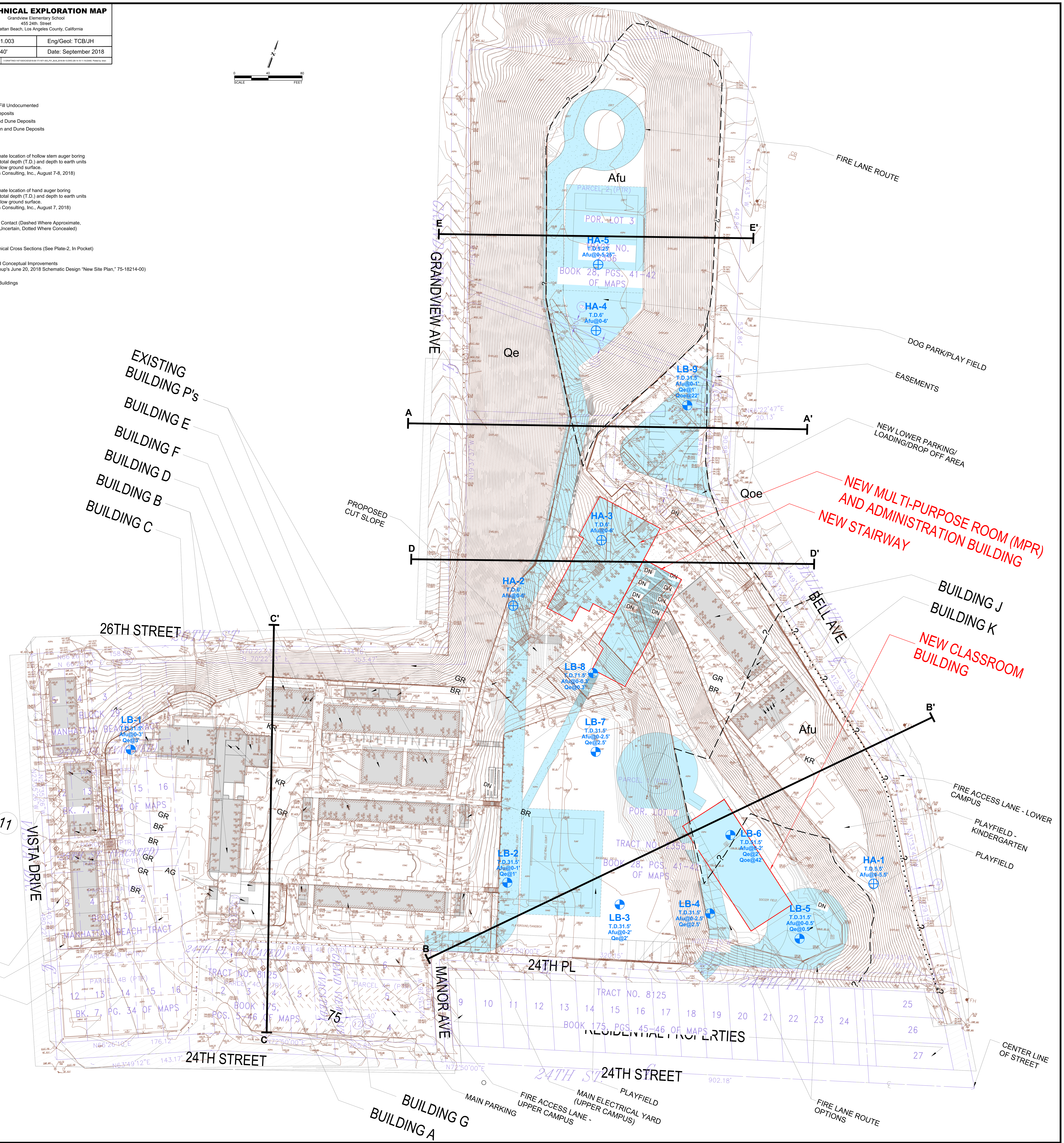
Grandview Elementary School
455 24th. Street
Manhattan Beach, Los Angeles County, California

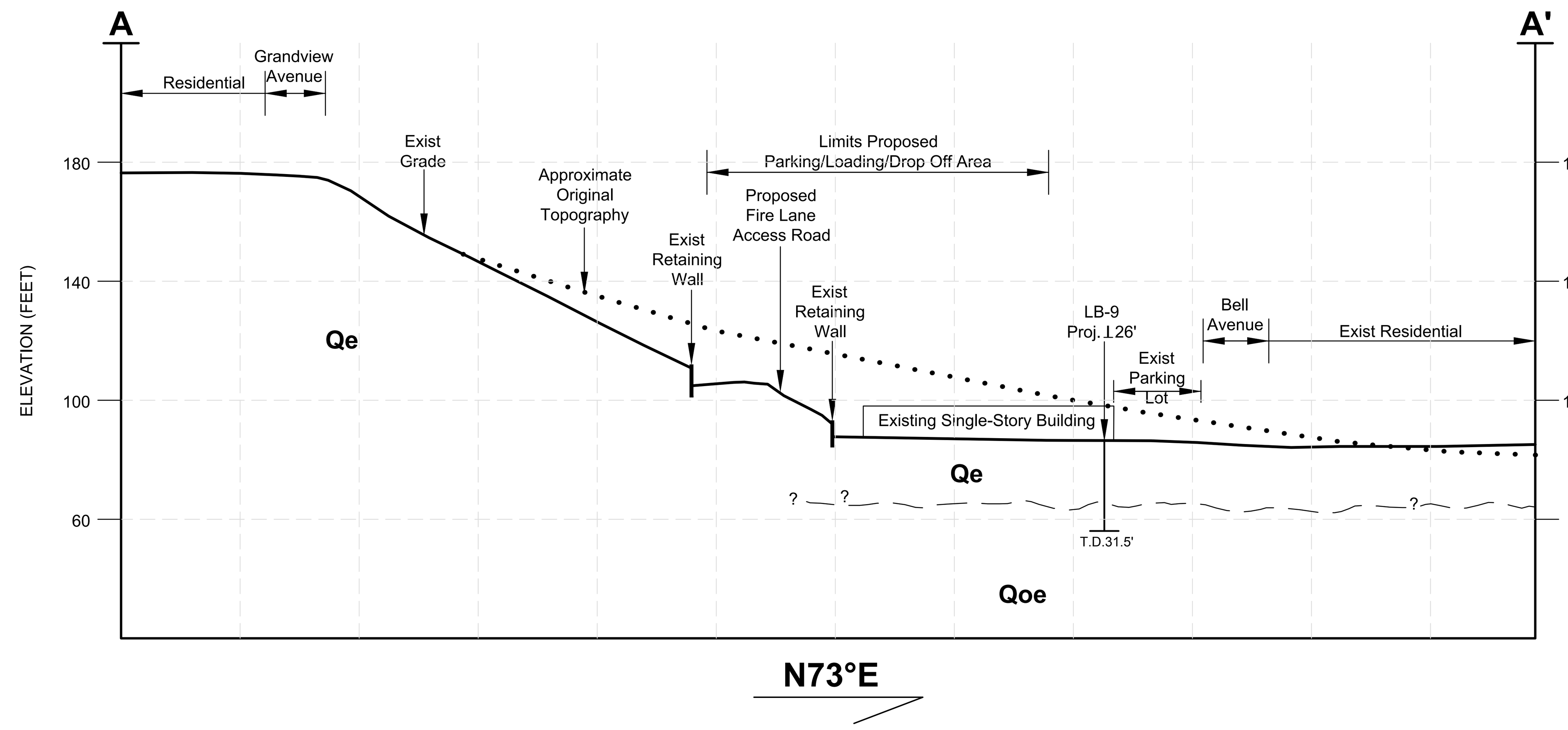
Proj: 11671.003	Eng/Geol: TCB/JH
Scale: 1"=40'	Date: September 2018

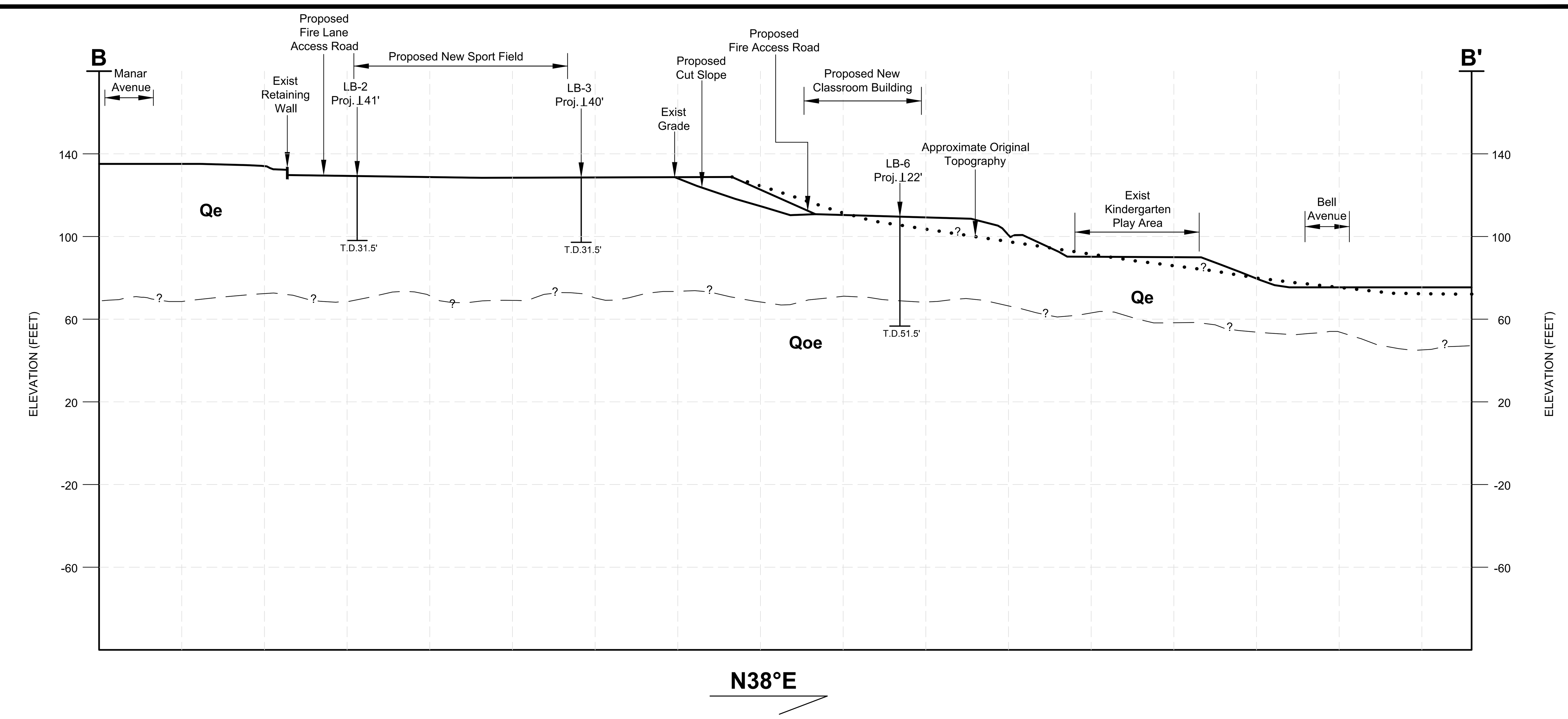
Drafted By: BQT | Checked By: BQT | V:\DRAFTING\11671\003\CAD\2018-08-17\11671-003_P01_BLM_2018-09-13.DWG (09-14-18 11:19:23AM) Plotted by: btran

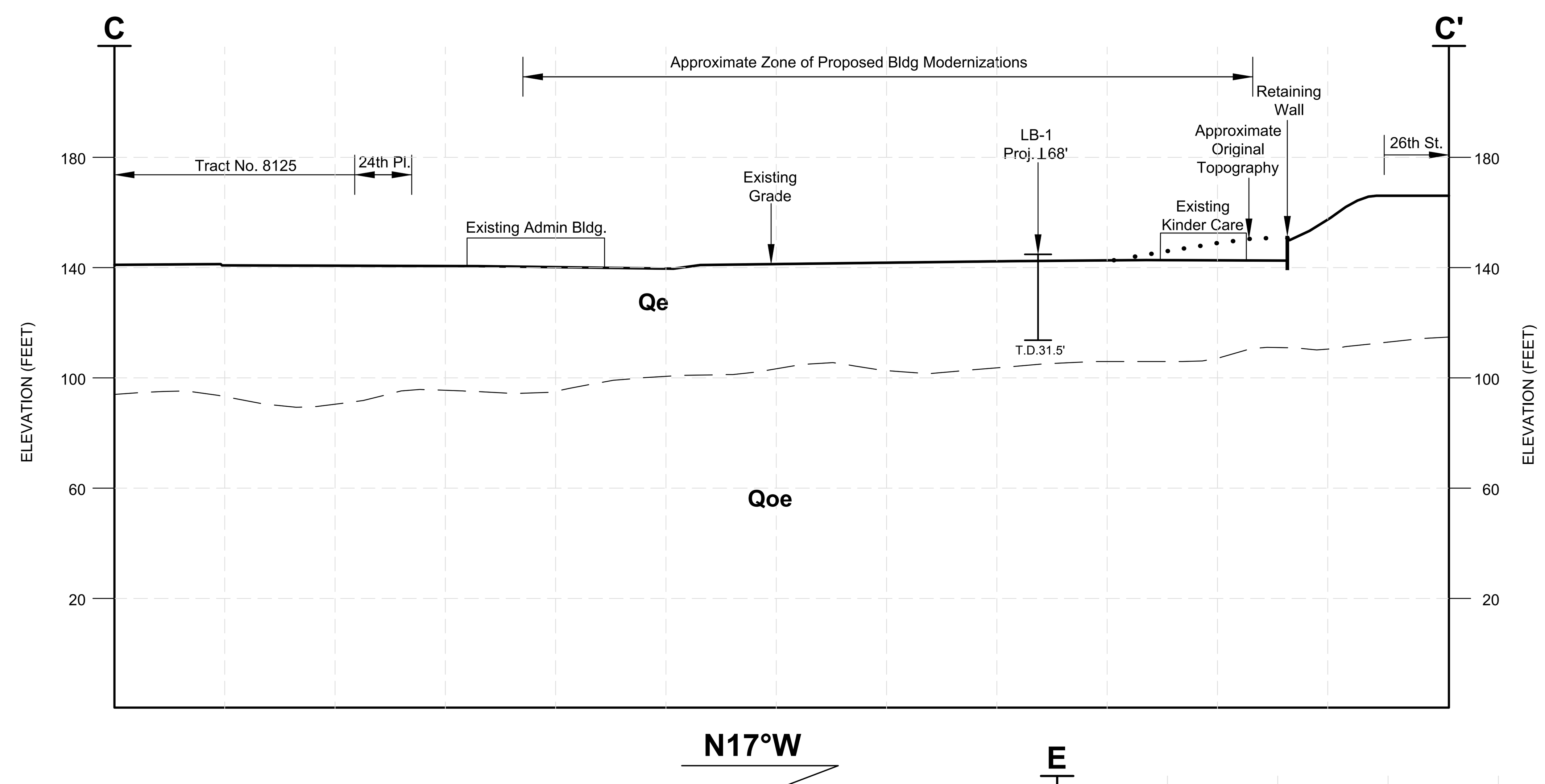
LEGEND

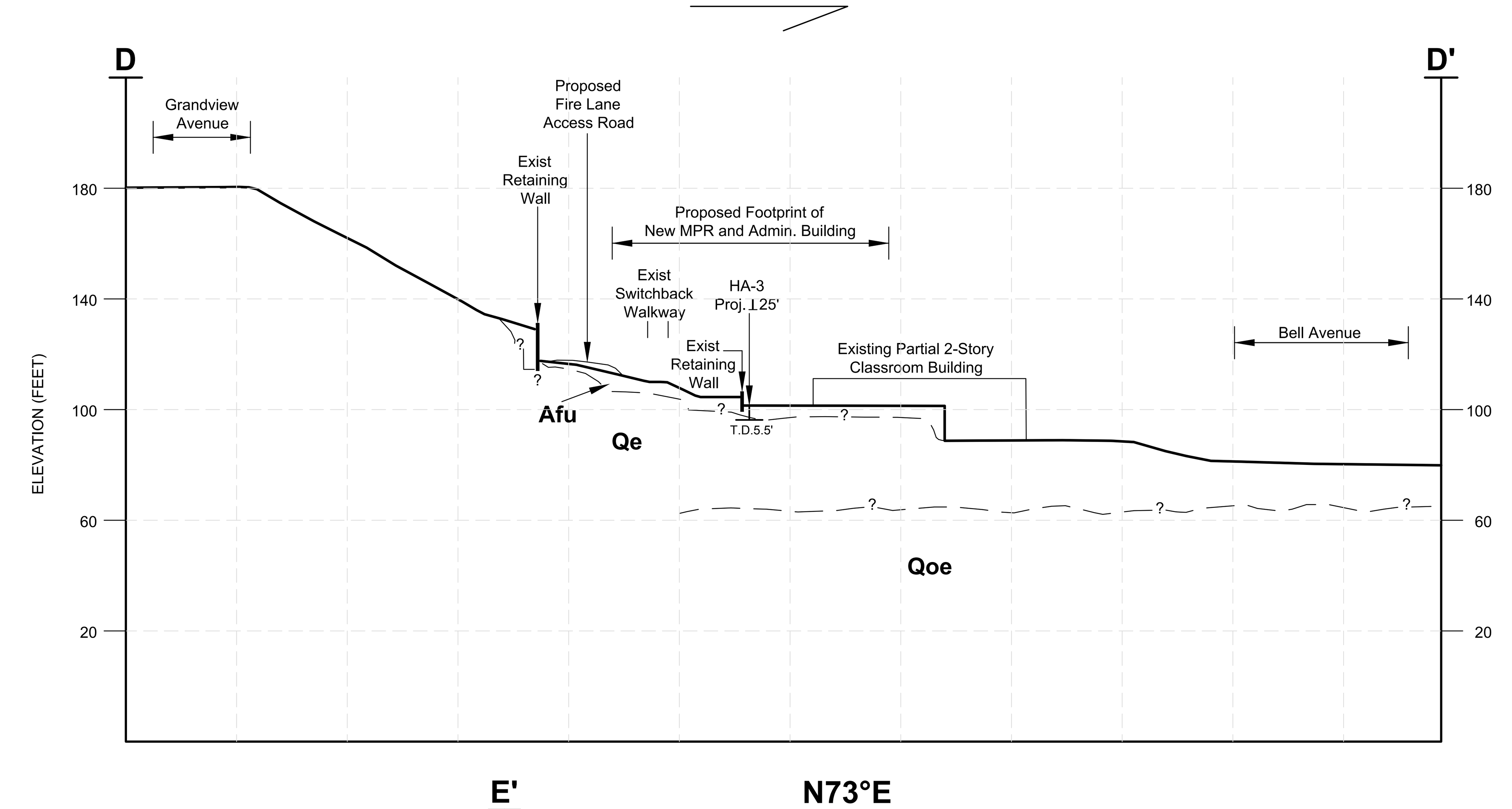
Afu	Artificial Fill Undocumented
Qb	Beach Deposits
Qe	Eolian and Dune Deposits
Qoe	Old Eolian and Dune Deposits
LB-9 T.D.31.5' Afu@0-1' Qoe@1'	Approximate location of hollow stem auger boring showing total depth (T.D.) and depth to earth units in feet below ground surface. (Leighton Consulting, Inc., August 7-8, 2018)
HA-5 T.D.6' Afu@0-6'	Approximate location of hand auger boring showing total depth (T.D.) and depth to earth units in feet below ground surface. (Leighton Consulting, Inc., August 7, 2018)
—?— —• • • •	Geologic Contact (Dashed Where Approximate, Queried Uncertain, Dotted Where Concealed)
E —— E'	Geotechnical Cross Sections (See Plate-2, In Pocket)
(light blue)	Proposed Conceptual Improvements (DLR Group's June 20, 2018 Schematic Design "New Site Plan," 75-18214-00)
(gray)	Existing Buildings











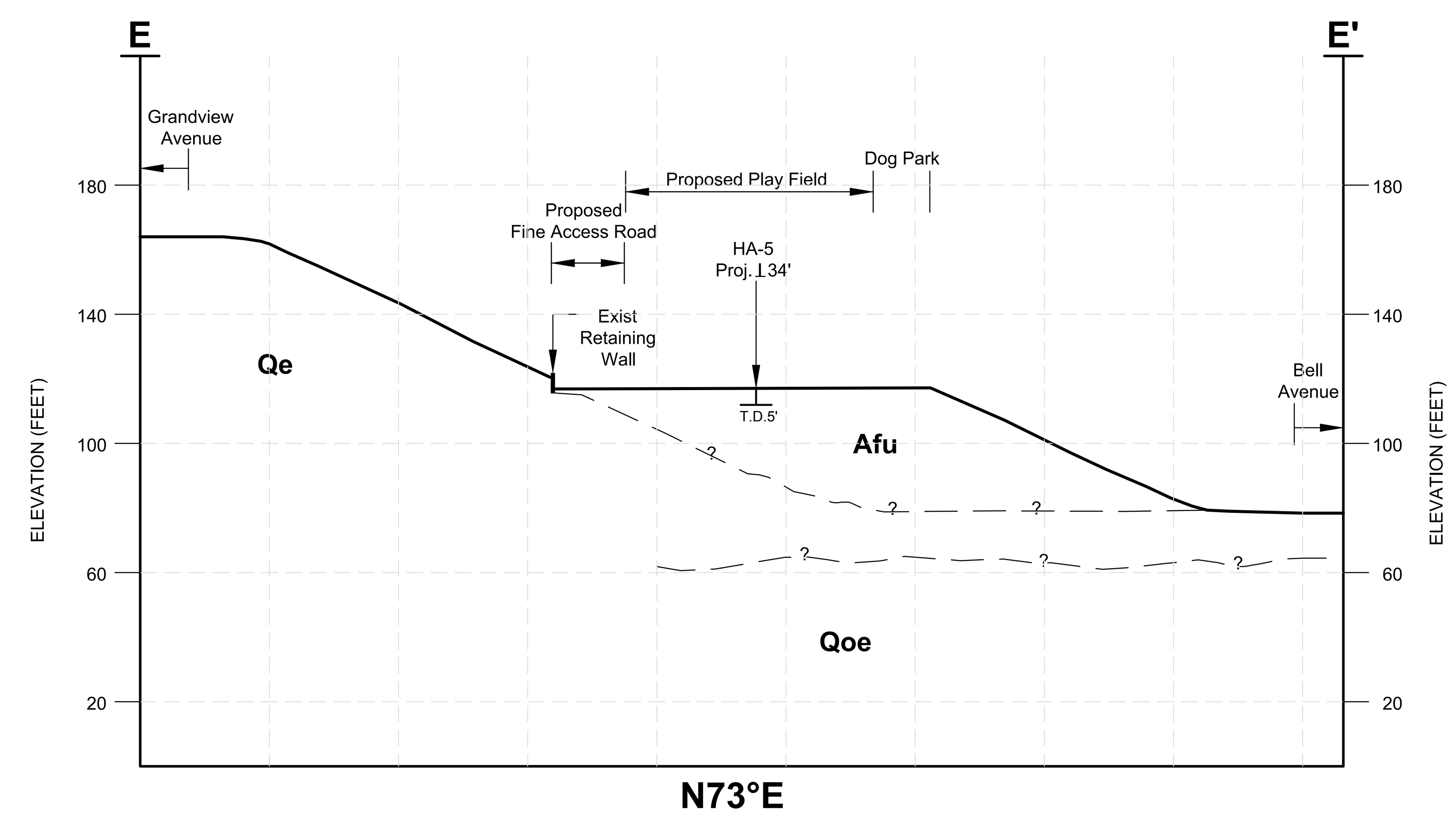


Plate 2

GEOTECHNICAL CROSS SECTIONS
A-A', B-B', C-C', D-D' AND E-E'

Grandview Elementary School
455 24th. Street
Manhattan Beach, Los Angeles County, California

Leighton

Proj: 11671.003	Eng/Geol: TCB/JH
Scale: 1"=40'	Date: September 2018

SEE PLATE 1 FOR SECTION LINE LOCATIONS

APPENDIX A

FIELD EXPLORATION

Our field exploration consisted of a surface reconnaissance, and subsurface exploration by drilling small-diameter truck-mounted hollow-stem-auger borings and hand-auger borings. Prior to beginning fieldwork, we marked proposed boring locations on site and contacted Underground Service Alert (USA) to mark utilities at proposed subsurface exploration locations. These subsurface exploration locations are plotted on Plate 1, *Geotechnical Exploration Map* (in pocket), and describe in more detail below:

On August 7th and 8^{th}, 2018, nine hollow-stem-auger and five hand-auger borings were drilled at this site to depths ranging from five to 51½ feet below existing grades. Boring locations were chosen based on DLR Group's conceptual design plan we received in July 2018. Encountered soils were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D 2488). Relatively undisturbed California ring-lined soil drive-samples were obtained at selected depth intervals within these borings. Standard Penetration Tests (SPTs) were also driven at selected intervals within the hollow-stem auger borings. Both drive samplers in the hollow-stem borings were driven with a 140-pound hammer falling 30-inches. Near surface bulk soil samples were also collected from these borings. Boring logs are included as part of this appendix. Our borings were backfilled immediately after drilling, logging and sampling the same day. Soil samples were transported to our Irvine geotechnical laboratory (DSA LEA-63) for geotechnical testing.

Attached boring logs depict subsurface conditions at the specific approximate locations noted on our Plate 1, *Geotechnical Exploration Map* (in pocket), during designated date(s) of exploration. Subsurface conditions at other site locations may differ from those encountered in our borings. It is also possible that passage of time could alter conditions due to environmental changes, fluctuations in groundwater, or occurrence of a significant geological event such as an earthquake. In addition, boundaries between soil or geologic units noted on our logs are based on field observations and results of geotechnical laboratory testing. Vertical location of these "contacts" is not precise, given vertical sampling intervals, and possible transitional/gradual changes in soil types.

GEOTECHNICAL BORING LOG HA-1

Project No.	11671.003	**Date Drilled**	8-7-18
Project	Grand View	**Logged By**	JLH
Drilling Co.	Earthworks Techniques	**Hole Diameter**	3.5"
Drilling Method	Hand Auger	**Ground Elevation**	90'
Location	*See Plate 1- Geotechnical Exploration Map*	**Sampled By**	JLH

Elevation Feet	Depth Feet	Graphic Log (N S)	PID	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
										This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
90	0			BB-1	Bulk				SP	@0': **Artificial Fill, undocumented (Afu):** SAND, medium yellow, loose, dry, fine to medium sand, cohesionless, silty matrix, gritty to poorly graded, caving to angle of repose	CR
85	5			R1	Driven		114	1		@5' - 6': Increasingly dense	
										Total Depth of Boring: 6 feet bgs **Groundwater not encountered during drilling** **Boring backfilled with soil cuttings upon completion of drilling on 8/7/18** **Excess soil cuttings spread onsite**	
80	10										
75	15										
70	20										
65	25										
60	30										

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG HA-2

Project No.	11671.003	**Date Drilled**	8-7-18
Project	Grand View	**Logged By**	JLH
Drilling Co.	Earthworks Techniques	**Hole Diameter**	3.5"
Drilling Method	Hand Auger	**Ground Elevation**	125'
Location	*See Plate 1- Geotechnical Exploration Map*	**Sampled By**	JLH

Elevation Feet	Depth Feet	Graphic Log (N S)	PID	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
125	0			BB-1	Bulk				SP	@0': **Artificial Fill, undocumented (Afu):** SAND, medium to light yellow, loose, dry to slightly moist, minor roots, moderately well rounded, predominantly medium sand, noncohesive	
				R1	Driven		97	1		@2.5' - 6': SAND, medium yellow brown, loose to medium dense, slightly moist, fine sand, predominantly quartz, well rounded, slightly micaceous	
120	5			R2	Driven		95	1			
										Total Depth of Boring: 6 feet bgs **Groundwater not encountered during drilling** **Boring backfilled with soil cuttings upon completion of drilling on 8/7/18** **Excess soil cuttings spread onsite**	
115	10										
110	15										
105	20										
100	25										
95	30										

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG HA-3

Project No.	11671.003	**Date Drilled**	8-7-18
Project	Grand View	**Logged By**	JLH
Drilling Co.	Earthworks Techniques	**Hole Diameter**	3.5"
Drilling Method	Hand Auger	**Ground Elevation**	103'
Location	*See Plate 1- Geotechnical Exploration Map*	**Sampled By**	JLH

Elevation Feet	Depth Feet	Graphic Log (N S)	PID	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			BB-1					SP	@0' - 5': **Artificial Fill, undocumented (Afu):** SAND, light yellow to white, dry, loose, fine to medium sand, predominantly quartz, noncohesive	
100											
	5			R1			100	2		@5' - 6': Increased density.	
95										**Total Depth of Boring: 6 feet bgs** **Groundwater not encountered during drilling** **Boring backfilled with soil cuttings upon completion of drilling on 8/7/18** **Excess soil cuttings spread onsite**	
	10										
90											
	15										
85											
	20										
80											
	25										
75											
	30										

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG HA-4

Project No.	11671.003	**Date Drilled**	8-7-18
Project	Grand View	**Logged By**	JLH
Drilling Co.	Earthworks Techniques	**Hole Diameter**	3.5"
Drilling Method	Hand Auger	**Ground Elevation**	117'
Location	*See Plate 1- Geotechnical Exploration Map*	**Sampled By**	JLH

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	PID	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			BB-1	Bulk				SP-SM	@0': **Artificial Fill, undocumented (Afu):** Silty SAND to SAND, medium to light yellow, loose, dry to slightly moist, predominantly medium sand, minor roots, moderately well rounded, noncohesive	
115				R1	Driven		100	1			
	5			R2	Driven		97	5	SP	SAND, light reddish brown, medium dense, dry, fine to medium sand, well rounded, local shell fragments	
110										**Total Depth of Boring: 6 feet bgs** **Groundwater not encountered during drilling** **Boring backfilled with soil cuttings upon completion of drilling on 8/7/18** **Excess soil cuttings spread onsite**	
	10										
105											
	15										
100											
	20										
95											
	25										
90											
	30										

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:

-200	% FINES PASSING	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

GEOTECHNICAL BORING LOG HA-5

Project No.	11671.003		**Date Drilled**	8-7-18
Project	Grand View		**Logged By**	JLH
Drilling Co.	Earthworks Techniques		**Hole Diameter**	3.5"
Drilling Method	Hand Auger		**Ground Elevation**	117'
Location	*See Plate 1- Geotechnical Exploration Map*		**Sampled By**	JLH

Elevation Feet	Depth Feet	Graphic Log (N / S)	PID	Sample No. (Bulk / Driven)	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	0			BB-1				SP	@0': **Artificial Fill, undocumented (Afu):** SAND with silt, olive, loose, dry to slightly moist, fine sand, local rock fragments 3% gravel, 92% sand, 5% fines	SA
115										
									@3.25': Refusal, cobble sized rock fragment encountered @3.3': SAND, medium brown, slightly moist, minor asphalt concrete fragments, some rounded igneous gravels	
	5								@5': Refusal on cobble-sized concrete clast.	
									Total Depth of Boring: 5.25 feet bgs **Groundwater not encountered during drilling** **Boring backfilled with soil cuttings upon completion of drilling on 8/7/18** **Excess soil cuttings spread onsite**	
110										
	10									
105										
	15									
100										
	20									
95										
	25									
90										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:

-200	% FINES PASSING	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-1

Project No.	11671.003	**Date Drilled**	8-8-18
Project	Grand View	**Logged By**	SAM
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	142'
Location	*See Plate 1- Geotechnical Exploration Map*	**Sampled By**	SAM

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Graphic Log: N S

Elevation Feet	Depth Feet	PID	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0							@0': 4.5-inches Asphalt Concrete over 8-inches Aggregate Base	
140			BB-1				SM	@1': **Artificial Fill, undocumented (Afu):** SAND with silt, greyish brown, slightly moist, fine sand, some fine subangular gravel, trace asphalt; 5% gravel, 90% sand, 5% fines	SA
			R1	13 19 22		3	SP-SM	@2.5': Medium dense	
								@3': **Quaternary Eolian and Dune Deposits (Qe):** SAND with silt to Silty SAND, tan, medium dense, slightly moist, fine sand	
	5		R2	3 5 7		1	SP	@5': SAND, tan, loose, slightly moist, fine sand	
135			S1	2 4 5		1			
	10		R3	4 9 15	98	1	SP-SM	@10': SAND with silt to Silty SAND, tan, medium dense, moist, fine sand	
130									
	15		S2	7 12 16		2			
125									
	20		R4	14 30 50/4		2			
120									
	25		S3	5 10 15		2	SM	@25': Silty SAND, tan, medium dense, slightly moist, fine sand, with one 1-inch lamination of caliche	
115									
	30								

Bulk / Driven

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-1

Project No.	11671.003	**Date Drilled**	8-8-18
Project	Grand View	**Logged By**	SAM
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	142'
Location	*See Plate 1- Geotechnical Exploration Map*	**Sampled By**	SAM

Elevation Feet	Depth Feet	Graphic Log (N S)	PID	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
										This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	30			R5		9 20 36	102	2		@30': Trace coarse sand	
110										**Total Depth of Boring: 31.5 feet bgs** **Groundwater not encountered during drilling** **Boring backfilled with soil cuttings upon completion of drilling on 8/8/18** **Excess soil cuttings spread onsite**	
	35										
105											
	40										
100											
	45										
95											
	50										
90											
	55										
85											
	60										

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-2

Project No.	11671.003	**Date Drilled**	8-8-18
Project	Grand View	**Logged By**	SAM
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	129'
Location	*See Plate 1- Geotechnical Exploration Map*	**Sampled By**	SAM

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log	PID	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0								@0': 10-inches Asphalt Concrete over 0-inches Aggregate Base	
				BB-1				SP	@1': <u>Quaternary Eolian and Dune Deposits (Qe):</u> SAND, pale brown, slightly moist, fine sand 98% sand, 2% fines	SA
				R1	5 14 19	100	2		@2.5': Medium dense	
125	5			S1	1 1 1		2		@5': Very loose	
				R2	1 2 3	93	2		@7.5': Loose, trace shell fragments	
120	10			S2	1 2 3		3	SP-SM	@10': SAND with Silt to Silty SAND, tan, medium dense, moist, fine sand	
115	15			R3	5 10 17	103	2	SP	@15':SAND, orangish brown, medium dense, slightly moist, fine sand	
110	20			S3	4 6 9		3			
									@23': Harder drilling	
105	25			R4	8 10 29		3			
100	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-2

Project No.	11671.003	**Date Drilled**	8-8-18
Project	Grand View	**Logged By**	SAM
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	129'
Location	*See Plate 1- Geotechnical Exploration Map*	**Sampled By**	SAM

Elevation Feet	Depth Feet	Graphic Log (N S)	PID	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	30			S4		5 6 9		4	SP	@30': Few laminations of medium sand, one 1-inch lamination of caliche	
										Total Depth of Boring: 31.5 feet bgs Groundwater not encountered during drilling Boring backfilled with soil cuttings upon completion of drilling on 8/8/18 Excess soil cuttings spread onsite	
95											
	35										
90											
	40										
85											
	45										
80											
	50										
75											
	55										
70											
	60										

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
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- MD MAXIMUM DENSITY
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- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-3

Project No.	11671.003	**Date Drilled**	8-7-18
Project	Grand View	**Logged By**	SAM
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	128'
Location	*See Plate 1- Geotechnical Exploration Map*	**Sampled By**	SAM

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	PID	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			BB-1					SM	@0': Grass @0.1': **Artificial Fill, undocumented (Afu):** SAND with silt, grayish brown to orangish brown, moist, fine sand, some rootlets and organics	CR
125				R1		7 16 26	104	6	SP	@2': **Quaternary Eolian and Dune Deposits (Qe):** SAND, orangish brown, loose, slightly moist, fine sand	
	5			R2		4 6 6		7		@5': Very loose	
120				S1		1 2 2	98	4		@7.5': Loose, trace shell fragments	
	10			R3		4 8 12		5		@10': Moist to very moist	
115											
	15			S2		4 7 9	104	4			
110											
	20			R4		5 11 12		6		@20': Slightly moist, very fine to fine sand	
105											
	25			S3		2 3 4	100	4			
100											
	30										

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG LB-3

Project No.	11671.003	**Date Drilled**	8-7-18
Project	Grand View	**Logged By**	SAM
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	128'
Location	*See Plate 1- Geotechnical Exploration Map*	**Sampled By**	SAM

Elevation Feet	Depth Feet	Graphic Log (N S)	PID	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
										This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	30			R5		5 10 16			SP	@30': SAND, orangish brown, medium dense, slightly moist, fine sand	
										Total Depth of Boring: 31.5 feet bgs **Groundwater not encountered during drilling** **Boring backfilled with soil cuttings upon completion of drilling on 8/7/18** **Excess soil cuttings spread onsite**	
95											
	35										
90											
	40										
85											
	45										
80											
	50										
75											
	55										
70											
	60										

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG LB-4

Project No.	11671.003	**Date Drilled**	8-7-18
Project	Grand View	**Logged By**	SAM
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	111'
Location	*See Plate 1- Geotechnical Exploration Map*	**Sampled By**	SAM

Elevation Feet	Depth Feet	Graphic Log (N S)	PID	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0									@0': 4-inches Asphalt Concrete over 10-inches Aggregate Base	
110				BB-1					SP	@1': **Artificial Fill, undocumented (Afu):** SAND, olive brown, dry, fine sand	MD
				R1		8 19 31		1	SP	@2.5': **Quaternary Eolian and Dune Deposits (Qe):** SAND, orangish brown, dense, dry to slightly moist, fine sand	
	5			R2		4 7 11	115	1	SP-SM	@5': SAND with silt, orange brown, medium dense, slightly moist, fine sand, with laminations of medium sand	
105				S1		2 5 7		1		@7.5': Trace shell fragments	
	10			R3		5 15 23	89	1		@10': Zones of fine to medium sand	
100											
	15			S2		5 9 11		2		@15': With one 1-inch lamination of caliche	
95											
	20			R4		18 31 50/4		2		@20': Fine sand	
90											
	25			S3		8 18 25		3			
85											
	30										

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-4

Project No.	11671.003	**Date Drilled**	8-7-18
Project	Grand View	**Logged By**	SAM
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	111'
Location	*See Plate 1- Geotechnical Exploration Map*	**Sampled By**	SAM

Elevation Feet	Depth Feet	Graphic Log (N S)	PID	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
80	30			R5		17 50/5	100	2	SP-SM	@30': SAND with silt, orange brown, medium dense, slightly moist, fine sand, with laminations of medium sand	
										Total Depth of Boring: 31.5 feet bgs **Groundwater not encountered during drilling** **Boring backfilled with soil cuttings upon completion of drilling on 8/7/18** **Excess soil cuttings spread onsite**	
75	35										
70	40										
65	45										
60	50										
55	55										
	60										

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG LB-5

Project No.	11671.003	**Date Drilled**	8-7-18
Project	Grand View	**Logged By**	SAM
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	110'
Location	*See Plate 1- Geotechnical Exploration Map*	**Sampled By**	SAM

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	PID	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
110	0									@0':Synthetic turf over 4-inches Aggregate Base	
				BB-1					SP	@0.5': **<u>Quaternary Eolian and Dune Deposits (Qe):</u>** SAND, pale brown, slightly moist, fine sand, trace coarse sand 2% gravel, 96% sand, 2% fines	SA
				R1		10 12 13				@2.5': Medium dense	
105	5			S1		2 4 4		4		@5': Loose, one 1-inch lamination of coarse sand	
				R2		6 11 18	101	3	SP	@7.5': SAND, orangish brown, medium dense, slightly moist, fine to medium sand, trace coarse sand, mildly oxidized 99.2% sand, 0.8% fines	
100	10			S2		3 5 7		4	SP	@10': SAND , orangish brown, medium dense, slightly moist, very fine to fine sand	-200
95	15			R3		4 12 20	100	3		@15': With zones of heavy oxidation	
90	20			S3		5 10 15		3		@20': With one 1-inch lamination of caliche	
85	25			R4		9 24 50/5		2	SM	@25': Silty SAND, orangish brown, very dense, slightly moist, fine sand, trace medium to coarse sand	
80	30										

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG LB-5

Project No.	11671.003	**Date Drilled**	8-7-18
Project	Grand View	**Logged By**	SAM
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	110'
Location	*See Plate 1- Geotechnical Exploration Map*	**Sampled By**	SAM

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	PID	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			S4		7 15 17		2	SM	@30': Silty SAND, orangish brown, dense, slightly moist, fine sand, trace medium to coarse sand	
										Total Depth of Boring: 31.5 feet bgs **Groundwater not encountered during drilling** **Boring backfilled with soil cuttings upon completion of drilling on 8/7/18** **Excess soil cuttings spread onsite**	
75	35										
70	40										
65	45										
60	50										
55	55										
50	60										

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG LB-6

Project No.	11671.003	**Date Drilled**	8-7-18
Project	Grand View	**Logged By**	SAM
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	109'
Location	*See Plate 1- Geotechnical Exploration Map*	**Sampled By**	SAM

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	PID	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0									@0': 5-inches Asphalt Concrete over 4-inches Aggregate Base	
				BB-1	Bulk				SP-SM	@0.5': **Artificial Fill, undocumented (Afu):** SAND with silt, orangish brown, slightly moist, fine sand, trace coarse sand, trace fine subangular gravel	RV
				R1	Driven	6 9 11	102	3	SM	@2': **Quaternary Eolian and Dune Deposits (Qe):** Silty SAND, orangish brown, medium dense, slightly moist, fine sand, trace coarse sand	
105	5			R2	Driven	5 9 15	104	3		@5': Loose, one 1-inch lamination of coarse sand	
				S1	Driven	4 7 9		4		@7.5': Mild gradation from very fine to fine sand, returning to very fine sand at depth	
100	10			R3	Driven	7 13 23	99	3		@10': Slightly moist	
95	15			S2	Driven	5 7 9		2			
90	20			R4	Driven	8 19 36	103	2		@20': Moist	
85	25			S3	Driven	7 15 16				@25': Some fine sand sized shell fragments	
80	30										

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG LB-6

Project No.	11671.003	**Date Drilled**	8-7-18
Project	Grand View	**Logged By**	SAM
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	109'
Location	*See Plate 1- Geotechnical Exploration Map*	**Sampled By**	SAM

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	PID	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			R5		14 27 50/5		1	SP-SM	@30': Trace shell fragments, grades to SAND with silt, orangish brown, very dense, slightly moist, fine sand, trace coarse sand	
75	35			S4		4 6 9		2		@35': Very fine sand	
70	40			R6		8 19 47	98	1			
									SM	@42': **Quaternary Old Eolian and Dune Deposits (Qoe):**	
65	45			S5		4 7 7		7		@45': Silty SAND, dark reddish brown, medium dense, slightly moist, fine sand, trace medium to coarse sand	
60	50			R7		8 16 24	122	8		@50': Few zones of grayish brown sand, some clay	
55	55									**Total Depth of Boring: 51.5 feet bgs** **Groundwater not encountered during drilling** **Boring backfilled with soil cuttings upon completion of drilling on 8/7/18** **Excess soil cuttings spread onsite**	
50	60										

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-7

Project No.	11671.003	**Date Drilled**	8-7-18
Project	Grand View	**Logged By**	SAM
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	129'
Location	*See Plate 1- Geotechnical Exploration Map*	**Sampled By**	SAM

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	PID	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0									@0': Grass	
				BB-1	Bulk				SP-SM	@0.1': **Artificial Fill, undocumented (Afu):** SAND with silt, grayish brown to pale brown, moist, fine sand, some rootlets 95% sand, 5% fines	SA
				R1	Driven	6 13 21	102	4	SM	@2.5': **Quaternary Eolian and Dune Deposits (Qe):** Silty SAND, orangish brown, medium dense, moist, fine sand, trace coarse sand	
125											
	5			S1	Driven	2 3 5	102	4		@5': Loose	
				R2	Driven	6 10 17		6		@7.5': Medium dense	
120											
	10			S2	Driven	3 6 9	97	4		@10': Very fine to fine sand	
115											
	15			R3	Driven	4 7 10		5		@15': Moist, fine sand	
110											
	20			S3	Driven	3 4 4	97	3		@20': Loose	
105											
	25			R4	Driven	7 17 29		5		@25': With laminations of silty sand, grayish brown	
100											
	30										

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG LB-7

Project No.	11671.003	**Date Drilled**	8-7-18
Project	Grand View	**Logged By**	SAM
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	129'
Location	*See Plate 1- Geotechnical Exploration Map*	**Sampled By**	SAM

Elevation Feet	Depth Feet	Graphic Log (N S)	PID	Sample No. (Bulk / Driven)	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	30			S4	5 8 12				@30': Oxidized	
									Total Depth of Boring: 31.5 feet bgs **Groundwater not encountered during drilling** **Boring backfilled with soil cuttings upon completion of drilling on 8/7/18** **Excess soil cuttings spread onsite**	
95										
	35									
90										
	40									
85										
	45									
80										
	50									
75										
	55									
70										
	60									

SAMPLE TYPES:
B BULK SAMPLE
C CORE SAMPLE
G GRAB SAMPLE
R RING SAMPLE
S SPLIT SPOON SAMPLE
T TUBE SAMPLE

TYPE OF TESTS:
-200 % FINES PASSING
AL ATTERBERG LIMITS
CN CONSOLIDATION
CO COLLAPSE
CR CORROSION
CU UNDRAINED TRIAXIAL
DS DIRECT SHEAR
EI EXPANSION INDEX
H HYDROMETER
MD MAXIMUM DENSITY
PP POCKET PENETROMETER
RV R VALUE
SA SIEVE ANALYSIS
SE SAND EQUIVALENT
SG SPECIFIC GRAVITY
UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG LB-8

Project No.	11671.003	**Date Drilled**	8-8-18
Project	Grand View	**Logged By**	SAM
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	124'
Location	*See Plate 1- Geotechnical Exploration Map*	**Sampled By**	SAM

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	PID	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0									@0': 4-inches Concrete over 0-inches Aggregate Base	
				BB-1	Bulk				SP	@0.3': **Quaternary Eolian and Dune Deposits (Qe):** SAND, pale brown, slightly moist, fine sand 2% gravel, 96% sand, 2% fines	SA
				R1	Driven	6 9 12		2		@2.5': Medium dense, moist	
120	5			R2	Driven	4 9 15	99	2		@5': Increase in silt content	
115				S1	Driven	4 7 9		2		@7.5': With laminations of medium to coarse sand, trace shell fragments	
	10			R3	Driven	5 9 17	102	3	SM	@10': Silty SAND, orangish brown, medium dense, slightly moist, fine sand	
110	15			S2	Driven	5 7 10		2		@15': Some medium to coarse sand, grades to very fine to fine sand at depth	
105	20			R4	Driven	15 25 50/5	105	2	SP-SM	@20': SAND with silt, orangish brown, very dense, slightly moist, fine sand, trace shell fragments	
100	25			S3	Driven	6 7 11		2	SM	@25': Silty SAND, orangish brown, medium dense, slightly moist, fine sand, with one 1-inch lamination of caliche	
95	30										

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG LB-8

Project No.	11671.003	**Date Drilled**	8-8-18
Project	Grand View	**Logged By**	SAM
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	124'
Location	*See Plate 1- Geotechnical Exploration Map*	**Sampled By**	SAM

Elevation Feet	Depth Feet	Graphic Log (N S)	PID	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	30			R5	Driven	8 11 12	97	2	SM	@30': Silty SAND, orangish brown, medium dense, slightly moist, fine sand	
										Total Depth of Boring: 31.5 feet bgs **Groundwater not encountered during drilling** **Boring backfilled with soil cuttings upon completion of drilling on 8/8/18** **Excess soil cuttings spread onsite**	
90	35										
85	40										
80	45										
75	50										
70	55										
65	60										

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-9

Project No.	11671.003	**Date Drilled**	8-8-18
Project	Grand View	**Logged By**	SAM
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	86'
Location	*See Plate 1- Geotechnical Exploration Map*	**Sampled By**	SAM

Elevation Feet	Depth Feet	Graphic Log (N S)	PID	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0									@0': 9-inches Asphalt Concrete over 2-inches Aggregate Base	
85				BB-1					SP	@1': **Quaternary Eolian and Dune Deposits (Qe):** SAND, dark brown, slightly moist, fine sand, some fine subangular gravel, oxidized 1% gravel, 95% sand, 4% fines	SA
				R1		8 11 16	101	1		@2.5': Medium dense	
	5			S1		3 4 6		3		@5': Some medium to coarse sand at top, grading to fine sand with depth	
80											
				R2		6 13 19	107	4			
	10			S2		4 6 7		4		@10': Slightly moist	
75											
	15			R3		10 15 17	101	2		@15': SAND, orangish brown, medium dense, slightly moist, fine sand, trace coarse sand	
70											
	20			S3		4 6 7		8		@20': Sharp increase in silt content, trace clay	
65											
									SP-SM	@22': **Quaternary Old Eolian and Dune Deposits (Qoe):**	
	25			R4		10 19 42	116	7		@25': Interlaminated SAND and Silty SAND, orange brown, dense, slightly moist, fine sand	
60											
	30										

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-9

Project No.	11671.003	**Date Drilled**	8-8-18
Project	Grand View	**Logged By**	SAM
Drilling Co.	Martini Drilling Co.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	86'
Location	*See Plate 1- Geotechnical Exploration Map*	**Sampled By**	SAM

Elevation Feet	Depth Feet	Graphic Log (N S)	PID	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
55	30			S4		7 9 13		5		@30': Medium dense	
										Total Depth of Boring: 31.5 feet bgs **Groundwater not encountered during drilling** **Boring backfilled with soil cuttings upon completion of drilling on 8/8/18** **Excess soil cuttings spread onsite**	
50	35										
45	40										
40	45										
35	50										
30	55										
	60										

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

***** This log is a part of a report by Leighton and should not be used as a stand-alone document. *****

APPENDIX B

GEOTECHNICAL LABORATORY TESTING

Our geotechnical laboratory testing program was directed toward a quantitative and qualitative evaluation of physical and mechanical properties of soils underlying this site, and to aid in verifying soil classification. This geotechnical testing was performed at our Irvine laboratory (DSA LEA 63).

Percent Fines (Percentage Passing No. 200 Sieve, -200): Selected soil samples were wet-washed through a No. 200 U.S. Standard brass sieve in accordance with ASTM Test Methods D1140 to measure percent fines (silts and clays). This data was used to refine the Unified Soil Classification for tested soil samples. Test results are tabulated in this appendix and listed on boring logs in Appendix A.

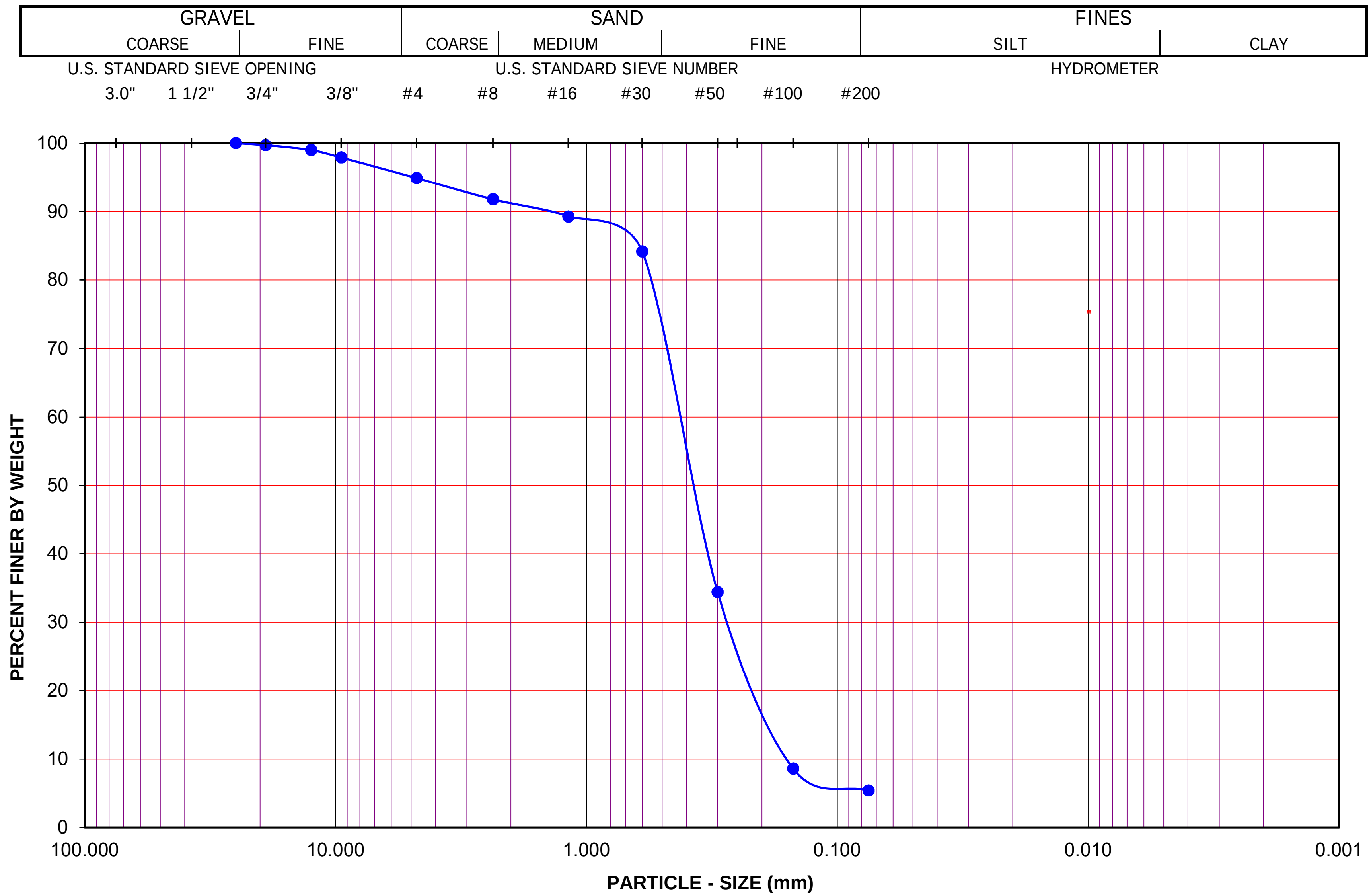
Sieve Analysis (SA): Selected bulk soil samples were tested for grain size distribution by sieving in accordance with ASTM Test Methods D6913. This data was used to refine the Unified Soil Classification for tested soil samples and to correlate engineering properties for similar soil types. Test results are plotted as particle-size distribution curves in this appendix.

Modified Proctor Compaction Curve (MD): In accordance with ASTM Test Methods D1557, laboratory modified Proctor compaction curves were established for bulk soil-samples to determine sample-specific modified Proctor laboratory maximum dry density and optimum moisture content. Results of these tests are plotted on the following "*Modified Proctor Compaction Test*" sheets in this appendix.

Direct Shear (DS): Direct shear testing was performed on one relatively undisturbed drive-soil sample. Three different rings were cut from each relatively undisturbed soil sample then sheared separately at three different normal loads to establish soil friction and cohesion parameters. Test results are presented on the Direct Shear Test Results figures in this appendix.

Corrosivity Tests (CR): To evaluate corrosion potential of subsurface soils at the site, we tested bulk soil samples collected during our subsurface exploration for pH, minimum electrical resistivity (CTM 532/643), soluble sulfate content (CTM 417 Part II) and soluble chloride content (CTM 422). Results of these tests are enclosed at the end of this appendix.

R-Value Tests (RV): A shallow bulk soil sample was tested in accordance with California (Caltrans) Test Method (CTM) 301, to model pavement subgrades for pavement section thickness design. Results are included in this appendix on the *R-value Test Results* sheet.



Project Name: Manhattan Beach USD/Grand View Elementary School

Project No.: 11671.003

Boring No.: LB-1

Sample No.: BB-1

Depth (feet): 0-5

Soil Type : SP-SM

Soil Identification: Grayish brown poorly-graded sand with silt (SP-SM)

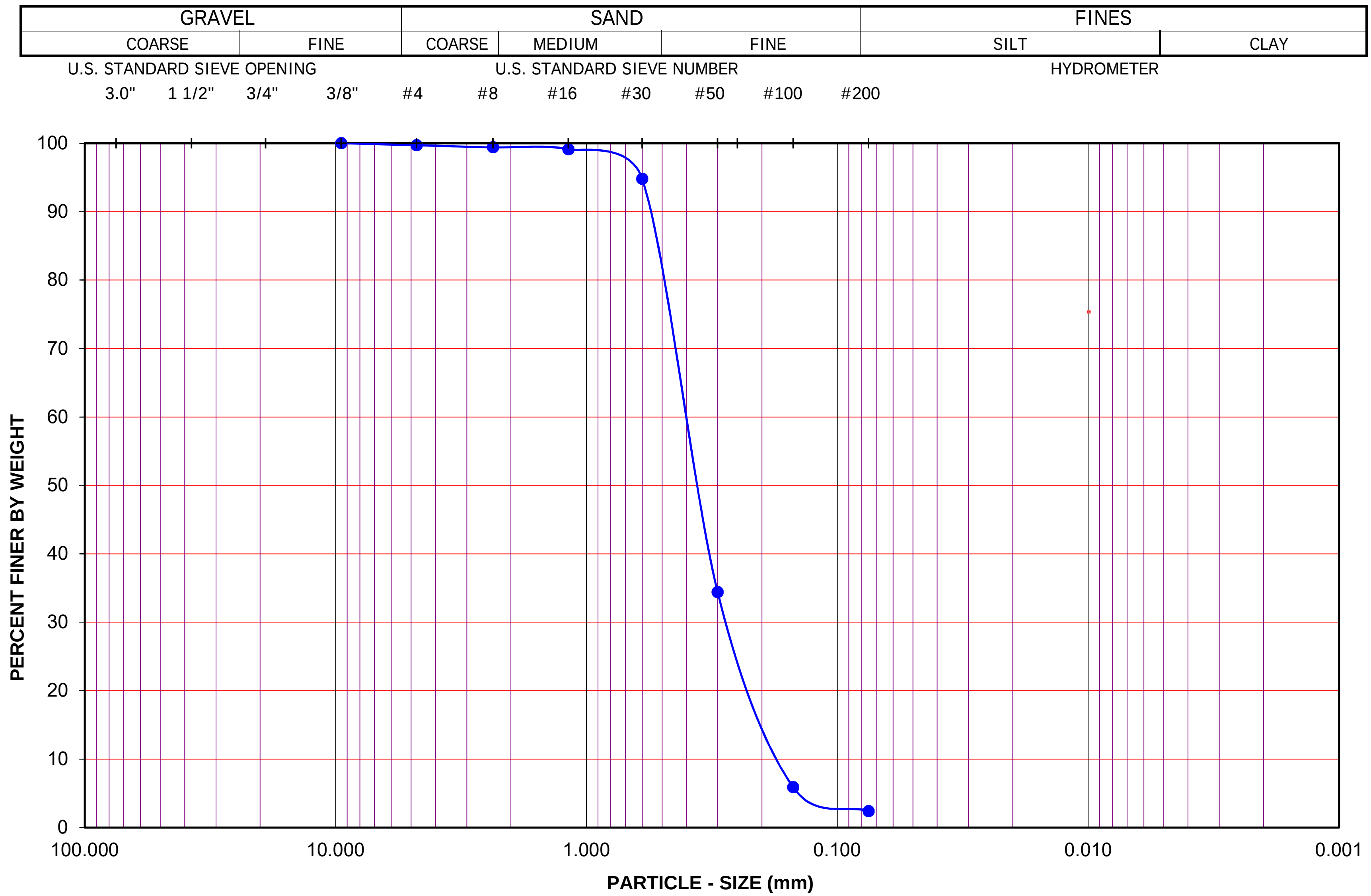
GR:SA:FI : (%) 5 : 90 : 5

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Aug-18

SA LB-1, BB-1 @ 0-5



Project Name:	Manhattan Beach USD/Grand View Elementary School	Boring No.: LB-2		Sample No.: BB-1
Project No.:	11671.003	Depth (feet): 0-5		Soil Type : SP
		Soil Identification:	Pale brown poorly-graded sand (SP)	

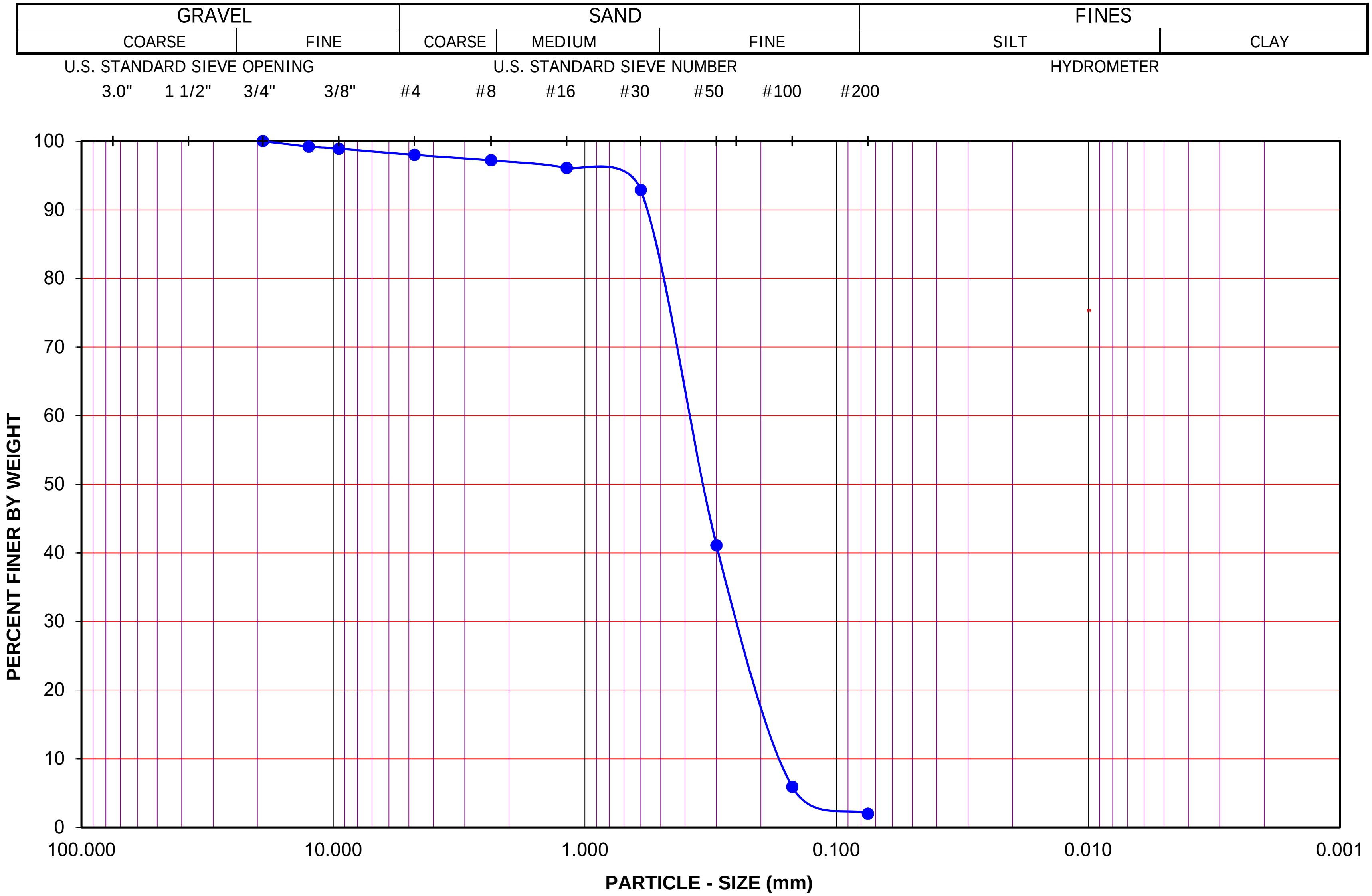
Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) **0 : 98 : 2**

Aug-18

SA LB-2, BB-1 @ 0-5



Project Name: Manhattan Beach USD/Grand View Elementary School

Project No.: 11671.003

Boring No.: LB-5

Sample No.: BB-1

Depth (feet): 0-5

Soil Type : SP

Soil Identification: Pale brown poorly-graded sand (SP)

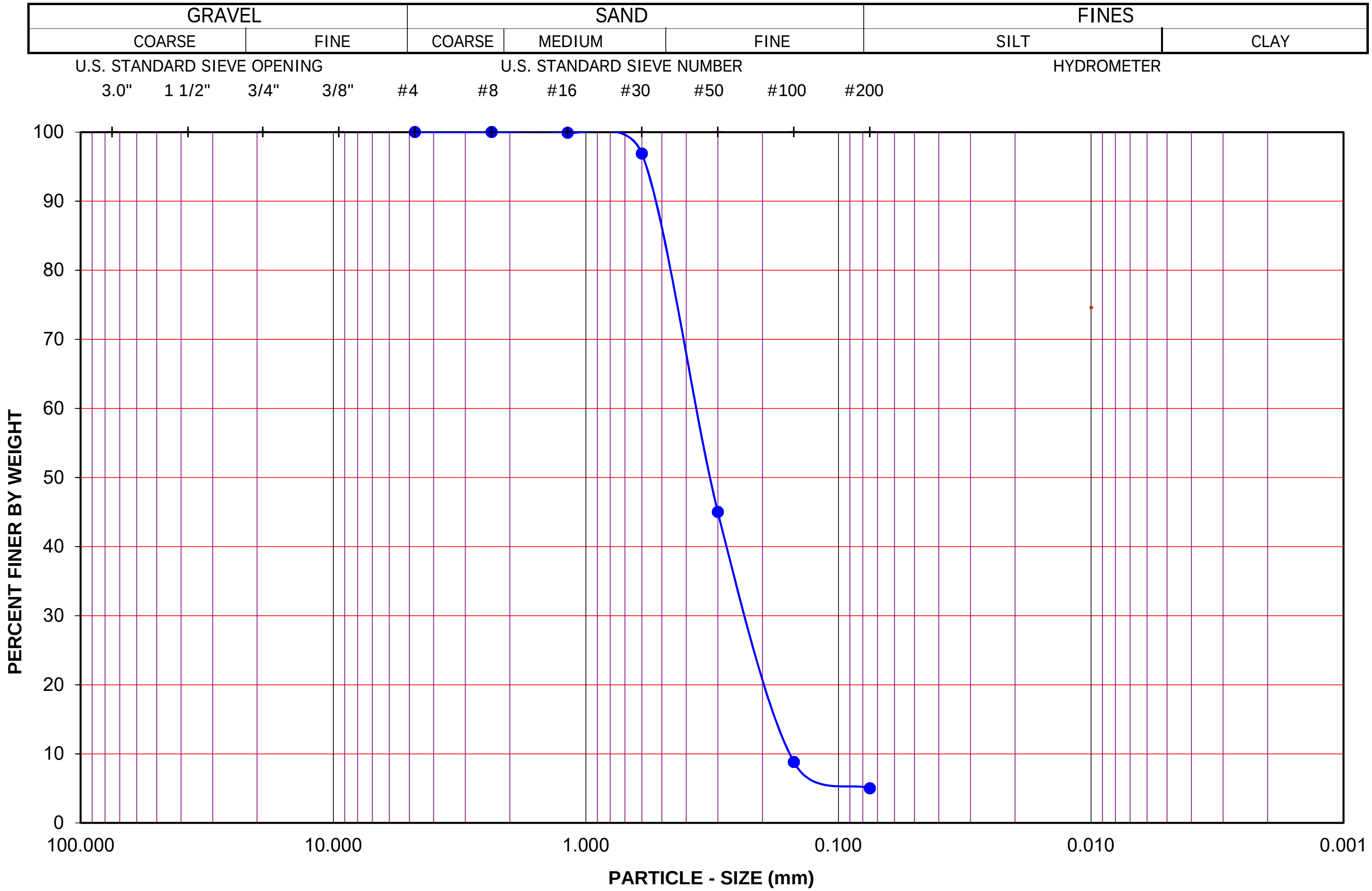
GR:SA:FI : (%) 2 : 96 : 2

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Aug-18

SA LB-5, BB-1 @ 0-5



Project Name: Manhattan Beach USD/Grand View Elementary School

Project No.: 11671.003

Boring No.: LB-7

Sample No.: BB-1

Depth (feet): 0-5

Soil Type : SP-SM

Soil Identification: Pale brown poorly-graded sand with silt (SP-SM)

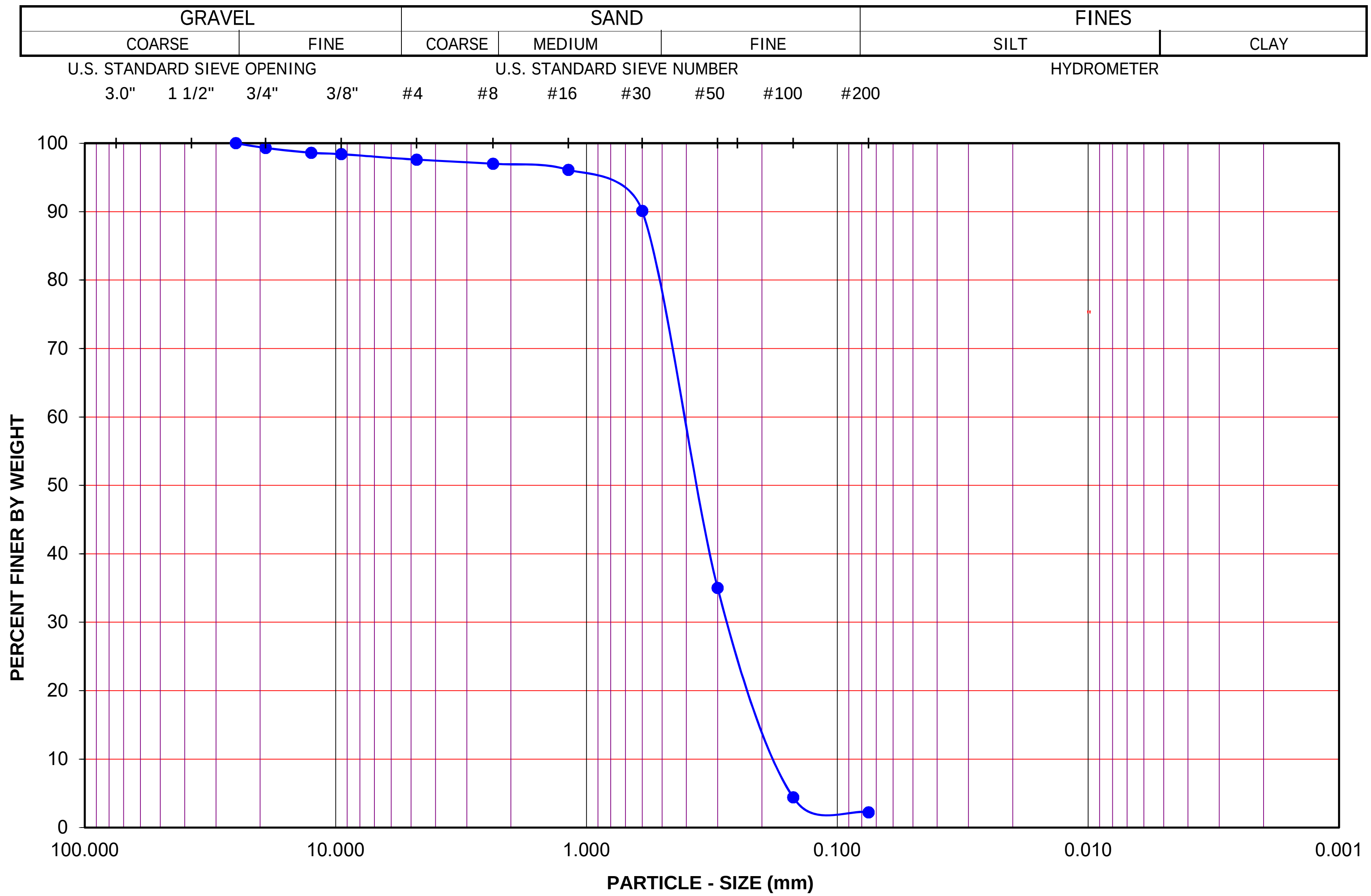
Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 0 : 95 : 5

Aug-18

SA LB-7, BB-1 @ 0-5



Project Name: Manhattan Beach USD/Grand View Elementary School

Boring No.: LB-8

Boring No.: LB-8 | Sample No.: BB-1

Depth (feet): 0-5 | Soil Type : SP

Soil Identification: Pale brown poorly-graded sand (SP)

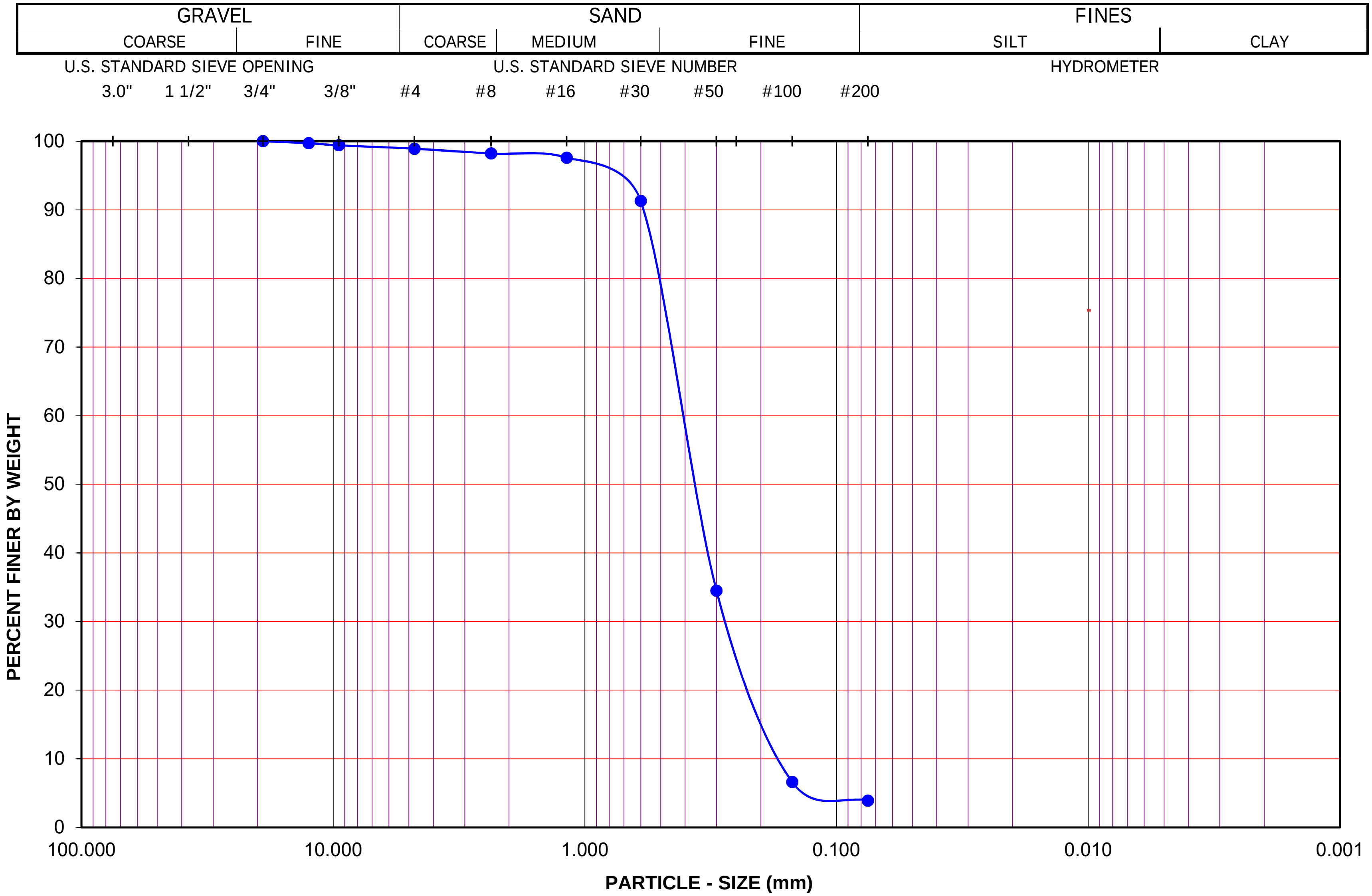
Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 2 : 96 : 2

Aug-18

SA LB-8, BB-1 @ 0-5



Project Name: Manhattan Beach USD/Grand View Elementary School

Project No.: 11671.003

Boring No.: LB-9

Sample No.: BB-1

Depth (feet): 0-5

Soil Type : SP

Soil Identification: Dark brown poorly-graded sand (SP)

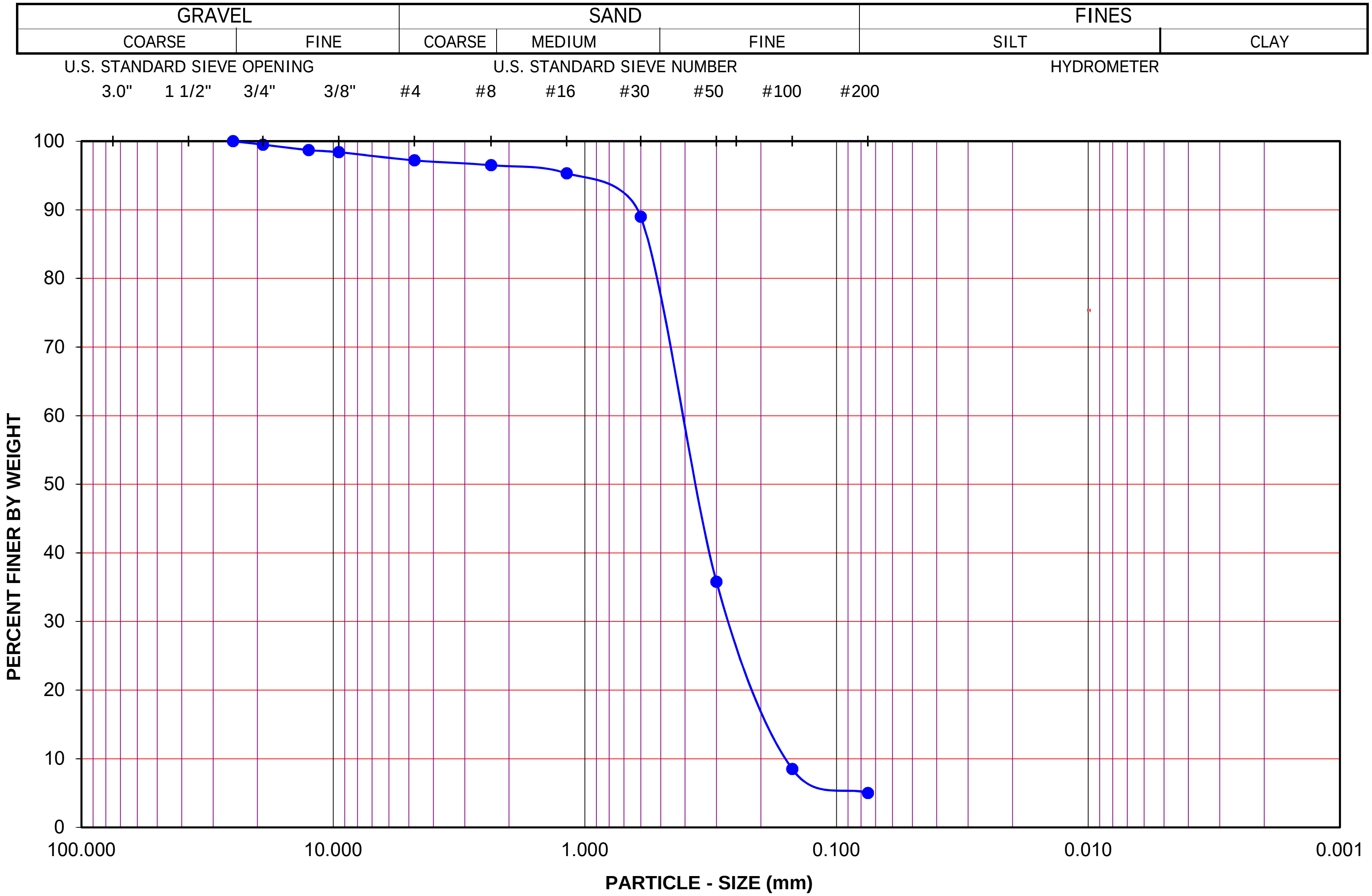
Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 1 : 95 : 4

Aug-18

SA LB-9, BB-1 @ 0-5



Project Name: Manhattan USD/Grand View Elementary School

Project No.: 11671.003

Boring No.: HA5

Sample No.: HA5-1

Depth (feet): 2.5-5

Soil Type : SP-SM

Soil Identification: Olive poorly-graded sand with silt (SP-SM)

GR:SA:FI : (%) **3 : 92 : 5**

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Aug-18

SA HA5, HA5-1 @ 2.5-5

Boring No.	LB-5								
Sample No.	S-2								
Depth (ft.)	10.0								
Sample Type	SPT								
Soil Identification	Brown poorly-graded sand (SP)								
Moisture Correction									
Wet Weight of Soil + Container (g)	0.0								
Dry Weight of Soil + Container (g)	0.0								
Weight of Container (g)	1.0								
Moisture Content (%)	0.0								
Sample Dry Weight Determination									
Weight of Sample + Container (g)	1007.2								
Weight of Container (g)	248.5								
Weight of Dry Sample (g)	758.7								
Container No.:									
After Wash									
Method (A or B)	A								
Dry Weight of Sample + Cont. (g)	1000.9								
Weight of Container (g)	248.5								
Dry Weight of Sample (g)	752.4								
% Passing No. 200 Sieve	**0.8**								
% Retained No. 200 Sieve	99.2								

Leighton

PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name: Manhattan Beach USD/Grand View Elementary School
Project No.: 11671.003
Tested By: S. Felter Date: 08/15/18

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	Manhattan Beach USD/Grand View Elementary School	Tested By:	G. Berdy	Date:	08/23/18
Project No.:	11671.003	Input By:	J. Ward	Date:	08/27/18
Boring No.:	LB-4	Depth (ft.):	0-5		
Sample No.:	BB-1				
Soil Identification:	Olive brown poorly-graded sand (SP)				

Preparation Method: [X] Moist [ ] Dry; [X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft³) 0.03330

Ram Weight = 10 lb.; Drop = 18 in.

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3517	3593	3642	3660		
Weight of Mold (g)	1848	1848	1848	1848		
Net Weight of Soil (g)	1669	1745	1794	1812		
Wet Weight of Soil + Cont. (g)	750.6	784.7	995.2	934.1		
Dry Weight of Soil + Cont. (g)	713.3	732.2	907.1	840.4		
Weight of Container (g)	231.5	223.8	223.6	231.2		
Moisture Content (%)	7.74	10.33	12.89	15.38		
Wet Density (pcf)	110.5	115.5	118.8	120.0		
Dry Density (pcf)	102.6	104.7	105.2	104.0		

Maximum Dry Density (pcf) 105.0 **Optimum Moisture Content (%)** 12.5

PROCEDURE USED

[X] **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

[ ] **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

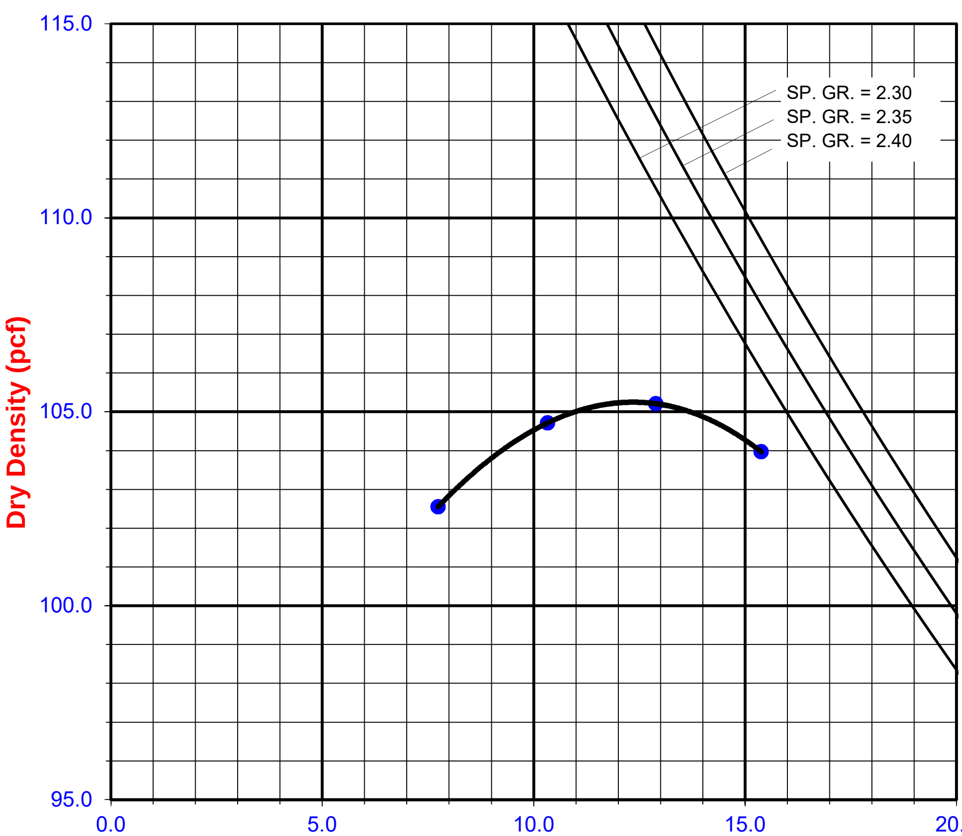
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



DIRECT SHEAR TEST

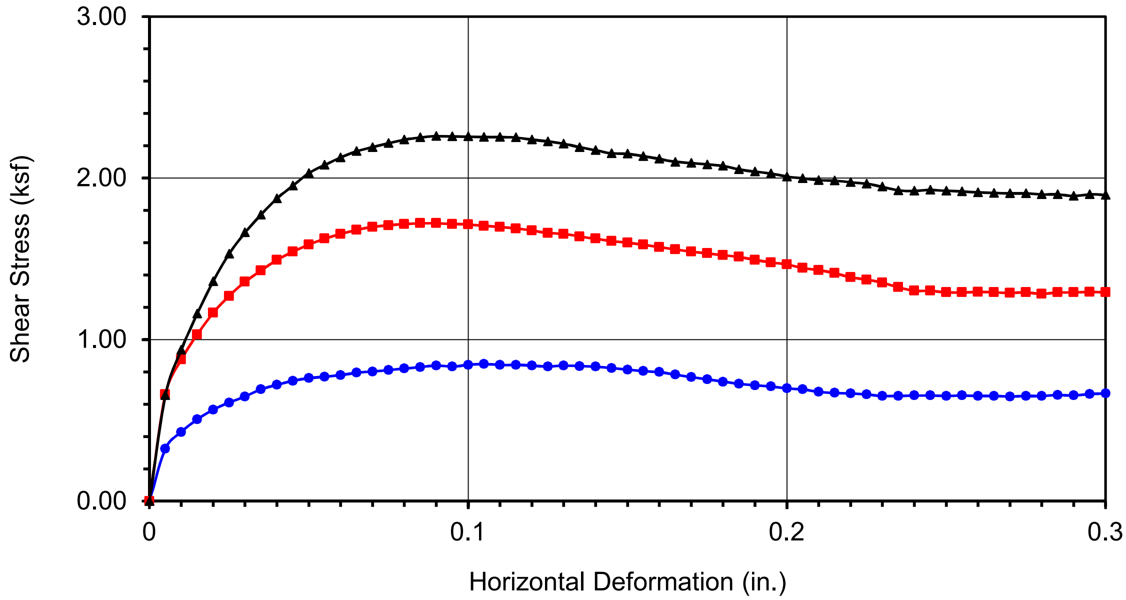
Consolidated Drained - ASTM D 3080

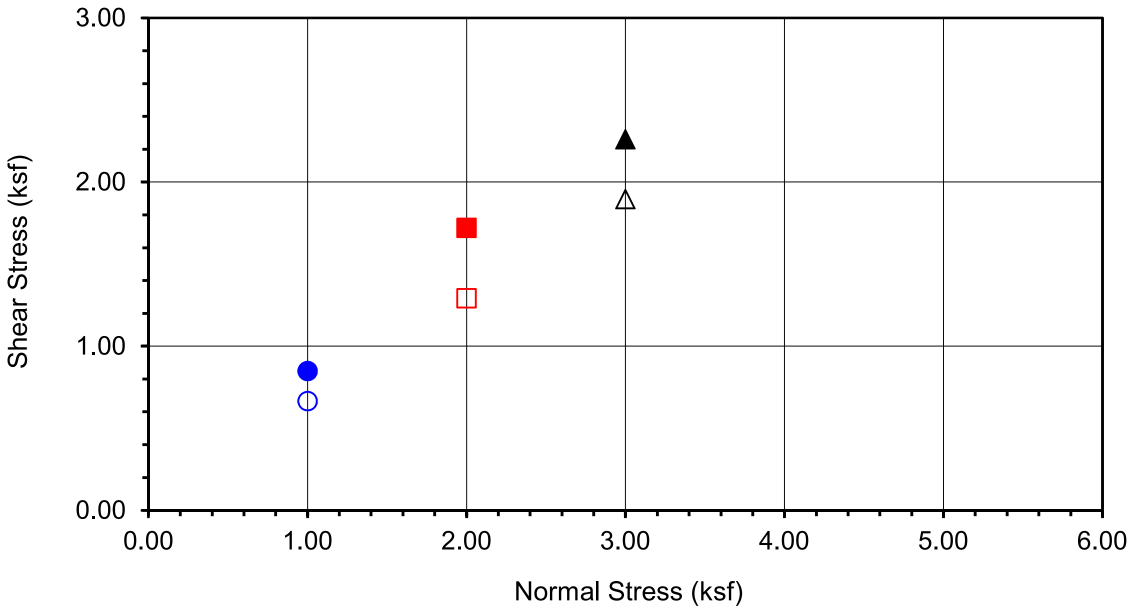
Leighton

Project Name:	Manhattan Beach USD/Grand View Elementary School	Tested By:	G. Bathala	Date:	09/13/18
Project No.:	11671.003	Checked By:	J. Ward	Date:	09/14/18
Boring No.:	LB-9	Sample Type:	Ring		
Sample No.:	R-2	Depth (ft.):	7.0		

Soil Identification: Dark yellowish brown poorly-graded sand (SP)

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	175.51	175.78	179.39
Weight of Ring(gm):	41.84	41.90	43.62
Before Shearing			
Weight of Wet Sample+Cont.(gm):	181.42	181.42	181.42
Weight of Dry Sample+Cont.(gm):	177.34	177.34	177.34
Weight of Container(gm):	66.95	66.95	66.95
Vertical Rdg.(in): Initial	0.2448	0.2389	0.2626
Vertical Rdg.(in): Final	0.2523	0.2541	0.2770
After Shearing			
Weight of Wet Sample+Cont.(gm):	183.97	182.31	225.93
Weight of Dry Sample+Cont.(gm):	162.99	162.90	205.81
Weight of Container(gm):	38.10	37.09	77.41
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-9**
Sample No.	**R-2**
Depth (ft)	**7**

Sample Type:

Ring

Soil Identification:
Dark yellowish brown poorly-graded sand (SP)

Normal Stress (kip/ft²)	1.000	2.000	3.000
Peak Shear Stress (kip/ft²)	● 0.849	■ 1.720	▲ 2.260
Shear Stress @ End of Test (ksf)	○ 0.666	□ 1.292	△ 1.896
Deformation Rate (in./min.)	0.0050	0.0050	0.0050
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	3.70	3.70	3.70
Dry Density (pcf)	107.2	107.4	108.9
Saturation (%)	17.4	17.5	18.2
Soil Height Before Shearing (in.)	0.9925	0.9848	0.9856
Final Moisture Content (%)	16.8	15.4	15.7

Leighton

DIRECT SHEAR TEST RESULTS

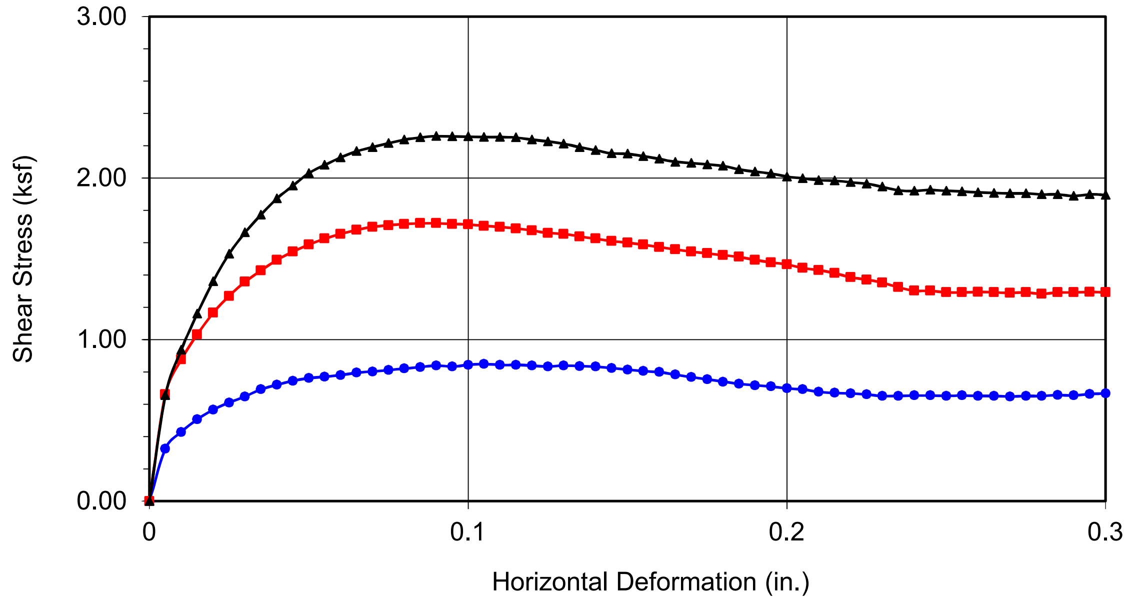
Consolidated Drained - ASTM D 3080

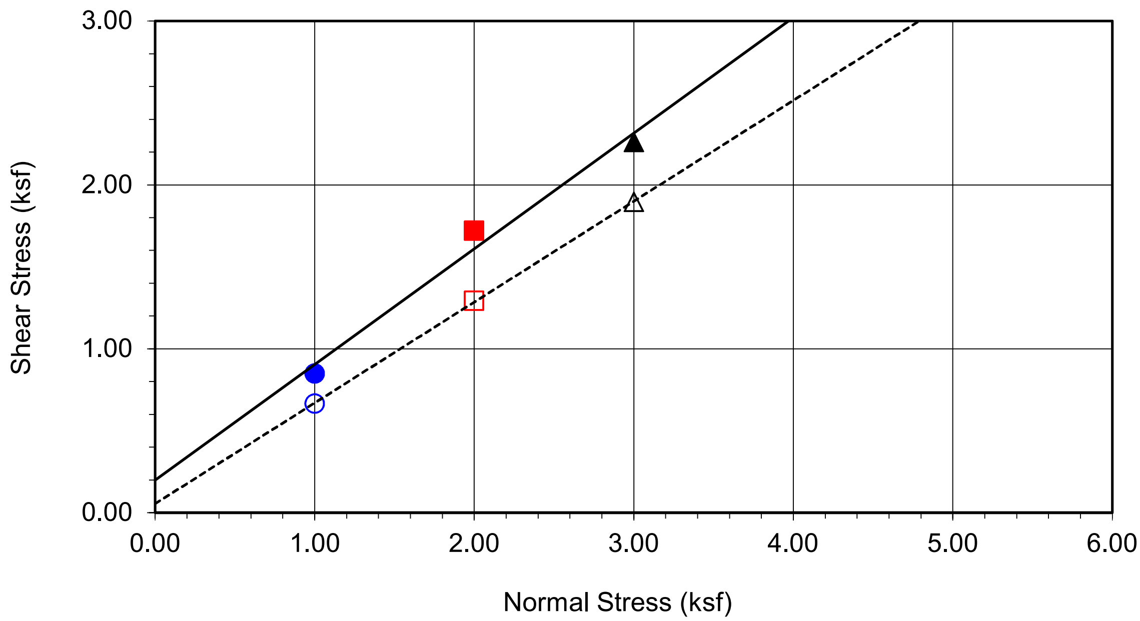
Project No.: 11671.003

Manhattan Beach USD/Grand View Elementary School

09-18

DS LB-9, R-2 @ 7





Boring No.	**LB-9**
Sample No.	**R-2**
Depth (ft)	**7**
Sample Type:	Ring

Soil Identification:
Dark yellowish brown poorly-graded sand (SP)

Strength Parameters

	C (psf)	ϕ (°)
Peak	199	35
Ultimate	55	32

Normal Stress (kip/ft²)	1.000	2.000	3.000
Peak Shear Stress (kip/ft²)	● 0.849	■ 1.720	▲ 2.260
Shear Stress @ End of Test (ksf)	○ 0.666	□ 1.292	△ 1.896
Deformation Rate (in./min.)	0.0050	0.0050	0.0050
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	3.70	3.70	3.70
Dry Density (pcf)	107.2	107.4	108.9
Saturation (%)	17.4	17.5	18.2
Soil Height Before Shearing (in.)	0.9925	0.9848	0.9856
Final Moisture Content (%)	16.8	15.4	15.7

Leighton

DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 11671.003
Manhattan Beach USD/Grand View Elementary School
09-18

DS LB-9, R-2 @ 7

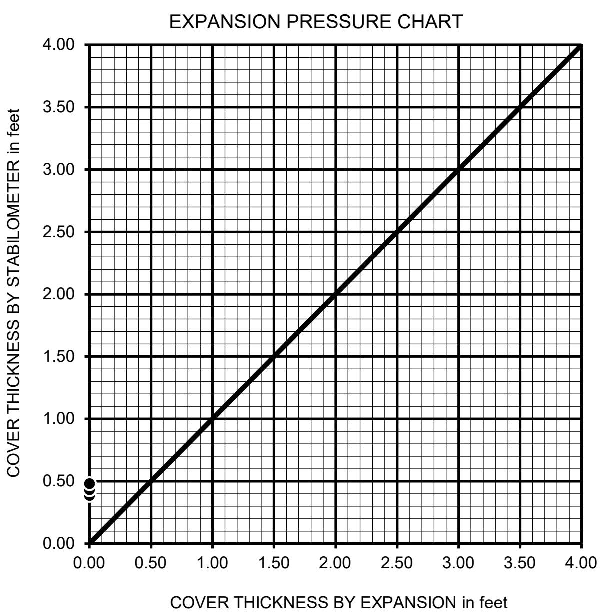
R-VALUE TEST RESULTS

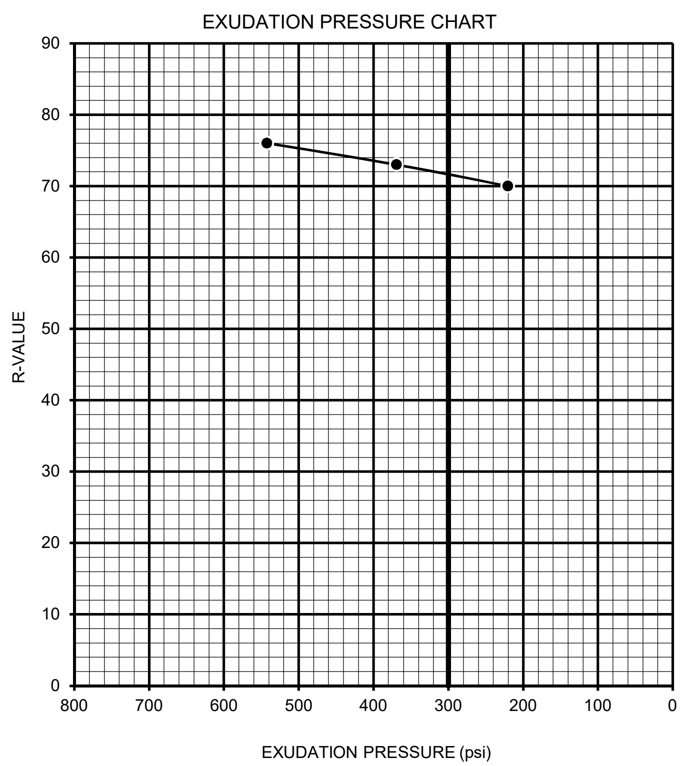
DOT CA Test 301

PROJECT NAME:	Manhattan Beach USD/Grand View Elementary School	PROJECT NUMBER:	11671.003
BORING NUMBER:	LB-6	DEPTH (FT.):	0-5
SAMPLE NUMBER:	BB-1	TECHNICIAN:	S. Felter
SAMPLE DESCRIPTION:	Brown SP-SM	DATE COMPLETED:	8/24/2018

TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	10.3	10.9	11.3
HEIGHT OF SAMPLE, Inches	2.61	2.62	2.56
DRY DENSITY, pcf	99.9	100.8	102.2
COMPACTOR PRESSURE, psi	100	50	50
EXUDATION PRESSURE, psi	543	369	220
EXPANSION, Inches x 10exp-4	0	0	0
STABILITY Ph 2,000 lbs (160 psi)	22	26	29
TURNS DISPLACEMENT	5.42	5.30	5.11
R-VALUE UNCORRECTED	74	71	69
R-VALUE CORRECTED	76	73	70

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.38	0.43	0.48
EXPANSION PRESSURE THICKNESS, ft.	0.00	0.00	0.00





R-VALUE BY EXPANSION:	N/A
R-VALUE BY EXUDATION:	71
EQUILIBRIUM R-VALUE:	71

Free water drain noted

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TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name: Manhattan Beach USD/Grand View Elementary School　Tested By : G. Berdy　Date: 08/21/18

Project No. : 11671.003　Data Input By: J. Ward　Date: 08/27/18

Boring No.	LB-3			
Sample No.	BB-1			
Sample Depth (ft)	0-5			
Soil Identification:	Olive SP-SM			
Wet Weight of Soil + Container (g)	175.17			
Dry Weight of Soil + Container (g)	173.78			
Weight of Container (g)	58.54			
Moisture Content (%)	1.21			
Weight of Soaked Soil (g)	100.75			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	307			
Crucible No.	11			
Furnace Temperature (°C)	860			
Time In / Time Out	10:00/10:45			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	22.1475			
Wt. of Crucible (g)	22.1463			
Wt. of Residue (g) (A)	0.0012			
PPM of Sulfate (A) x 41150	49.38			
PPM of Sulfate, Dry Weight Basis	**50**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15			
ml of AgNO3 Soln. Used in Titration (C)	0.5			
PPM of Chloride (C -0.2) * 100 * 30 / B	60			
PPM of Chloride, Dry Wt. Basis	**61**			

pH TEST, DOT California Test 643

pH Value	**6.71**			
Temperature °C	23.2			

SOIL RESISTIVITY TEST

DOT CA TEST 643

Leighton

Project Name:	Manhattan Beach USD/Grand View Elementary School	Tested By :	G. Berdy	Date:	08/24/18
Project No. :	11671.003	Data Input By:	J. Ward	Date:	08/27/18
Boring No.:	LB-3	Depth (ft.) :	0-5		
Sample No. :	BB-1				
Soil Identification:*	Olive SP-SM				

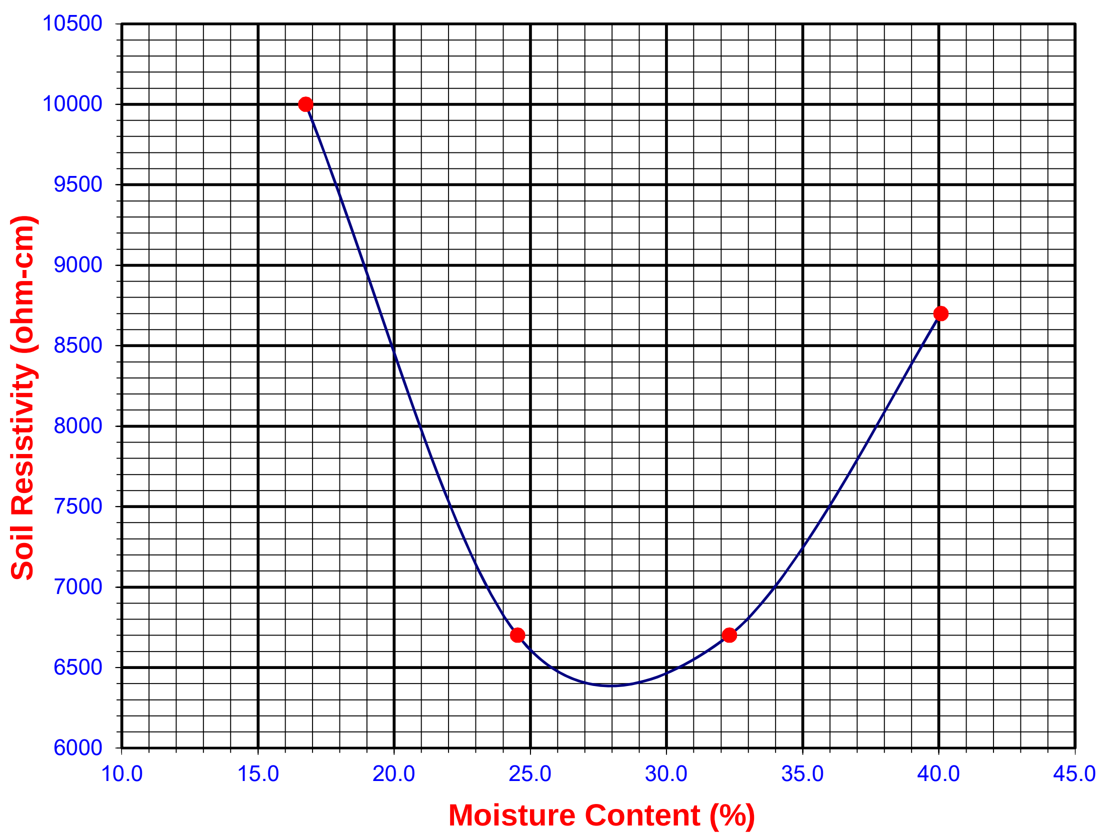
*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	20	16.76	10000	10000
2	30	24.53	6700	6700
3	40	32.31	6700	6700
4	50	40.08	8700	8700
5				

Moisture Content (%) (MCi)	1.21
Wet Wt. of Soil + Cont. (g)	175.17
Dry Wt. of Soil + Cont. (g)	173.78
Wt. of Container (g)	58.54
Container No.	
Initial Soil Wt. (g) (Wt)	130.17
Box Constant	1.000

MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
6400	**28.0**	**50**	**61**	**6.71**	**23.2**



TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Leighton

Project Name: Manhattan USD/Grand View Elementary School Tested By : G. Berdy Date: 08/15/18

Project No. : 11671.003 Data Input By: J. Ward Date: 08/23/18

Boring No.	HA3			
Sample No.	HA3-1			
Sample Depth (ft)	2.5-5			
Soil Identification:	Olive SP			
Wet Weight of Soil + Container (g)	195.31			
Dry Weight of Soil + Container (g)	194.18			
Weight of Container (g)	57.00			
Moisture Content (%)	0.82			
Weight of Soaked Soil (g)	100.38			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	153			
Crucible No.	17			
Furnace Temperature (°C)	860			
Time In / Time Out	10:00/10:45			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	22.2111			
Wt. of Crucible (g)	22.2101			
Wt. of Residue (g) (A)	0.0010			
PPM of Sulfate (A) x 41150	41.15			
PPM of Sulfate, Dry Weight Basis	**41**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	0.3			
PPM of Chloride (C -0.2) * 100 * 30 / B	10			
PPM of Chloride, Dry Wt. Basis	**10**			

pH TEST, DOT California Test 643

pH Value	**7.87**			
Temperature °C	23.5			

SOIL RESISTIVITY TEST

DOT CA TEST 643

Leighton

Project Name:	Manhattan USD/Grand View Elementary School	Tested By :	G. Berdy	Date:	08/16/18
Project No. :	11671.003	Data Input By:	J. Ward	Date:	08/23/18
Boring No.:	HA3	Depth (ft.) :	2.5-5		
Sample No. :	HA3-1				

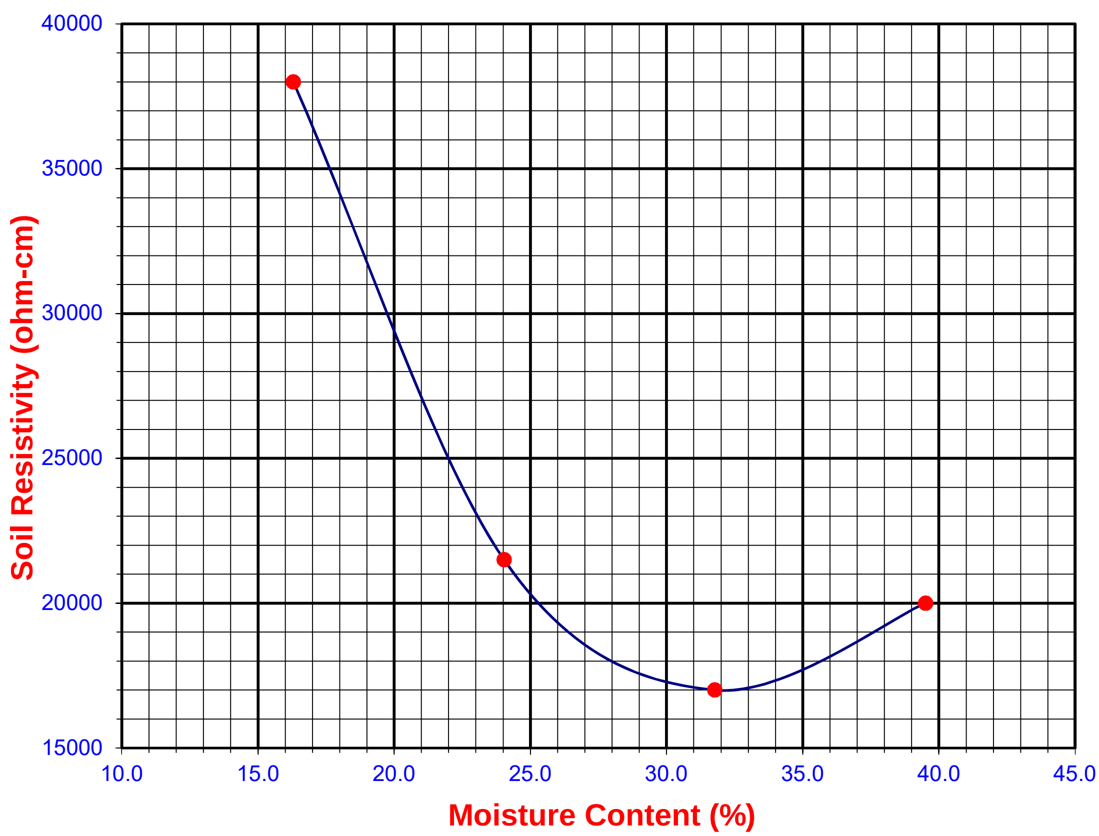
Soil Identification:* Olive SP

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	20	16.30	38000	38000
2	30	24.04	21500	21500
3	40	31.77	17000	17000
4	50	39.51	20000	20000
5				

Moisture Content (%) (MCi)	0.82
Wet Wt. of Soil + Cont. (g)	195.31
Dry Wt. of Soil + Cont. (g)	194.18
Wt. of Container (g)	57.00
Container No.	
Initial Soil Wt. (g) (Wt)	130.30
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
17000	**32.0**	**41**	**10**	**7.87**	**23.5**



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APPENDIX C

GBA'S IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL-ENGINEERING REPORT

Leighton

Important Information about This

Geotechnical-Engineering Report

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

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

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

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

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

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

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

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