

SUPPLEMENTAL GEOTECHNICAL EXPLORATION
PROPOSED WAREHOUSING FACILITY
3041± DEL AMO BOULEVARD
TORRANCE, LOS ANGELES COUNTY, CALIFORNIA
APN: 7352-015-004, 5 AND 6
PROLOGIS PROJECT ID: LAX12490

Prepared For:

PROLOGIS, INC.
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Project No. 12452.002

October 4, 2019



Leighton Consulting, Inc.

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Friday, October 4, 2019

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Prologis, Inc.
17777 Center Court Drive North, Suite 100
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Attention: Mr. Blake Kelley
Vice President, Construction & Development Manager

**Subject: Supplemental Geotechnical Exploration
Proposed Warehousing Facility
3041± Del Amo Boulevard
Torrance, Los Angeles County, California
APN: 7352-015-004, 5 and 6
Prologis Project ID: LAX 12490**

In accordance with our July 5, 2019 proposal, authorized on August 13, 2019 (Prologis Contract Change Order 1), Leighton Consulting, Inc. has completed this *Supplemental Geotechnical Exploration* for your use in final planning and design of a warehousing and semi-trailer staging facility to be constructed north of Del Amo Boulevard and east of Prairie Avenue, within the City of Torrance, Los Angeles County, California. This report completely replaces our September 17, 2019 “client draft.” We have relied on geotechnical and geoenvironmental exploration and testing by others, augmented by our current shallow subsurface exploration. Based on our current supplemental subsurface exploration and prior work by others, this report summarizes our findings, conclusions and recommendations regarding geotechnical aspects of this site and proposed warehousing facility.

This 31.3-acre site is located southwest of the Torrance Refinery (previously ExxonMobil Refinery), separated by BNSF’s five-track railway right-of-way. Reportedly, there is no known evidence of past oil production and/or refining activities having occurred on this site, but this site has reportedly been an earth borrow source for berms and other earthwork on the refinery, and ceramic catalyst-beads and other debris has been disposed of in the northwestern portion of this site (during our July 10, 2019 site visit, catalyst-beads were visible at the surface in the northwestern portion of this site).

This site is **not** located within a currently-designated Alquist-Priolo Earthquake Fault Zone of potential surface fault rupture, nor zone of potential seismically-induced liquefaction. Groundwater has been relatively deep at this site, presumably on the order of 60- to 100-feet below existing site surface. This site is geologically and geotechnically characterized as older stabilized dune sands (Dibblee's "Qos" and CGS's "Qoe"), and site topography is consistent with undulating dunes. The 1934 USGS "Torrance" Quadrangle shows a depression at and below the 75-foot elevation contour (NGVD29), filled by the railway at the northeast-center portion of the site; with the eastern, southern and western portions of the site roughly at elevation 100-feet. Psomas's March 16, 2018 (flown November 18, 2015) ALTA topographic survey for this site shows the lowest surveyed point in the borrow pit area at elevation 59.6 feet above mean sea level (presumably NAVD88; note that NAVD88 elevations are 2.4 feet higher at this site compared to the NGVD29 datum).

Reportedly, agreements were made, which limit earthwork and excavation activity to the zone on site above elevation 60 feet (above mean sea level, presumably NAVD88 datum). Psomas's March 16, 2018 (flown November 18, 2015) ALTA topographic survey for this site shows the lowest survey point within the borrow pit area at elevation 59.6 feet above mean sea level. Extrapolation and interpolation of prior reported subsurface conditions, in addition to conditions exposed in our five current test pits, shows undocumented fill extending below elevation 60 feet in the central area of this borrow pit as delineated with **red dots** on Plate 1, "*Locations of Explorations*" (in pocket in this report). Deepest reported undocumented fill extended down to elevation 54 feet, which was six feet below the limits of allowable excavation at elevation 60 feet above mean sea level. Encountered fill was highly variable but consisted predominantly of a black "fat" (high plasticity, CH) clay that was expansive with a fuel hydrocarbon odor. Some small amounts of encountered fill contained trace organics, which measured no-higher-than ($\leq$) 3.1% by loss-on-ignition weight.

Best standard geotechnical practice is to remove all undocumented fill under proposed building footprints, which would extend down to elevation 54 feet above mean sea level in the central portion of the existing borrow pit. However, this would be six feet below the allowable excavation threshold at elevation 60 feet. Existing fill was found to be predominantly expansive clay, which might expand, but not consolidate much. Conservatively assuming an unlikely worst-case 5% consolidation in this undocumented old fill, 6-feet of this fill could presumably settle 3½-inches in the center worst case fill depth; which would be differential settlement relative to the dense native sands with negligible long-term settlement. Therefore, **if** excavation below elevation 60 feet is not

allowed, **then** (and only then) alternative geotechnical mitigations are recommended, as follows:

- **Adjust Warehouse Footprint (Westerly Wall):** While central columns supporting a lightly-loaded roof for this warehouse will be susceptible to excessive differential settlement, perimeter tilt-up-concrete bearing walls will be the most sensitive to differential settlement. Therefore, we recommend that the currently proposed westerly wall of the warehouse be moved 20 feet west either by moving the whole warehouse footprint or perhaps increasing the warehouse size. This would move the west wall continuous footing outside of the influence of any subsiding remaining existing fill soils. Note that as an option if the footprint cannot be moved west, then the westerly wall continuous footing could be stepped down to elevation 90 feet adjacent to the borrow pit area, to provide a ½:1 (horizontal:vertical) setback of the interior edge of this new footing relative to the closest perimeter of existing undocumented fill left in place.
- **Earth Surcharge (Northerly Wall):** Based on our design-team discussions, we understand it is impractical to move the northern wall of the proposed warehouse to avoid overlying remaining undocumented fill. Pile foundations are prohibited, since drilling through the remaining undocumented fill is prohibited. Therefore, we recommend that the remaining undocumented fill be consolidated by earth-surcharging to mitigate total and differential settlement of existing fill along the western portion of the warehouse northern perimeter wall. An earth berm 25-feet-high above finish grade is proposed as surcharge; encompassing the portion of the warehouse footprint that will overlie remaining undocumented fill, extending at-least 25-feet beyond the warehouse footprint in areas of remaining undocumented fill. Settlement of finish grade under this surcharge should be monitored bi-weekly (by CTM 112 pipe-riser settlement devices), for **not-less-than (>) six months** and until diminished asymptotic settlement occurs, with less-than (<) one-tenth-of-an-inch (0.1) differential-settlement measured over a 30-day period.
- **Interior Tubular Steel Roof Columns:** While roof columns would be inevitable over this remaining undocumented fill zone, we would recommend that any heavy roof HVAC units or other heavy roof installations be avoided over this remaining undocumented fill zone. Tubular steel columns over this zone could be fitted with screw-jacks that can be adjusted if needed from time to time to compensate for differential settlement ranging from zero at the **red dot** perimeter to conceivably 3½-inches in the interior of this remaining undocumented fill zone (at SoCal Geo's B-9). Newly-placed compacted fill overlying this zone of undocumented fill would be geogrid reinforced to reduce differential-settlement. Concrete slabs-on-grade, only over this area of remaining undocumented fill, should be structurally isolated from other column bays; yet inter-doweled to reduce differential vertical movement between column bay joints.

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Site sands are non-expansive and have desirable consolidation characteristics (compared to clay) when properly compacted. Therefore, we recommend that site finish grades be lowered to provide more balanced earthwork requiring less import. Torrance is known to have undesirable highly expansive clays not far from this site. Such clays are undesirable due to adverse expansion and consolidation characteristics, and will consolidate over time relative to sand. If it is absolutely unavoidable to import clays to the site, then clays should be placed within the pit at least 15 feet below finish grade. Even with this design, clay consolidation will need to be evaluated specific to the grading scheme and structural requirements for differential settlement.

Although conventional tilt-up structures with conventional shallow spread footings are the likely structure alternative, differential settlement of new fill soils (presumably 35- to 40-feet-deep) needs to be evaluated relative to cut zones in older eolian (stabilized) sands. Compaction to 95% relative compaction in deeper fills is recommended (in more detail) in this report.

We appreciate the opportunity to be of service to Prologis and hope to see this project through construction. If you have any questions, or if we can be of further service, please call us at your convenience at **866-LEIGHTON**, directly at the phone extension and/or e-mail address listed below.



Respectfully submitted,

LEIGHTON CONSULTING, INC.

A handwritten signature in blue ink, appearing to read "Thomas C. Benson, Jr.", written over a horizontal line.

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TCB:tcb

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Appendix C – Earthwork and Grading Guide Specifications

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1.0 INTRODUCTION

1.1 Site Description

As depicted on Figure 1, *Site Location Map*, this relatively flat, gentle rolling dunes, site is roughly a triangular parcel (Los Angeles County APN: 7352-015-004, 5 and 6) covering on the order of 31.3 acres, bound by a “Santa Fe Railroad” (BNSF) right-of-way and the Torrance Refinery to the northeast, Del Amo Boulevard to the south and Prairie Avenue to the west; generally within north-central Torrance; within the Los Angeles Basin. This is an industrial area with the five-track active BNSF railway and Torrance Refinery oil tank farm to the northeast, vacant land and warehousing facility to the south, and offices and hotels to the west.

1.2 Proposed Warehouse

Our understanding of this project was derived solely from your June through September 2019 e-mails. Grading concepts are depicted on Plate 1, *Location of Explorations* (in pocket). As depicted on Plate 1 (in pocket), we understand that a 338,970-square-foot warehouse is proposed, located in the southeastern portion of this site, with semi-trailer parking located in the northwest portion of this site. A roughly 42-foot-high relatively-lightweight tilt-up warehouse structure is proposed, with a conventional concrete slab-on-grade designed for a stacked-product live-load of 1,000 pounds-per-square-foot (psf). Interior column and perimeter bearing wall loads are expected to be as follows:

Table 1. Proposed Warehouse Column and Bearing-Wall Loads

Load Location	Load Type (kips)		
	Dead	Live	Seismic
Interior Columns	57	41	120
Perimeter Bearing Walls (per foot)	8	0.6	-0-

1.3 Purpose and Scope of Work

Purpose of our exploration was to: (1) further evaluate geotechnical conditions in the vicinity of proposed warehouse footprint, (2) identify significant geotechnical issues that would impact this proposed warehouse construction, and (3) provide geotechnical recommendations for design and construction of this proposed warehouse and appurtenant site improvements. In accordance with our July 5, 2019 proposal, authorized on August 13, 2019 (Prologis Contract Change Order

1), Leighton Consulting, Inc. has completed this *Supplemental Geotechnical Exploration* specific to this site for your use in final planning and initial design of warehousing facilities. This report completely replaces our September 17, 2019 “client draft.” Scope of our evaluation has included the following tasks:

- **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.
- **Subsurface Exploration:** Our Certified Engineering Geologist (CEG) visited this site to observe exposed geologic conditions. On August 22, 2019, five backhoe-excavated test pits were excavated, logged and sampled to depths ranging from 5- to 14-feet below existing grades within the borrow pit area. After sampling and logging, all five test pits were immediately backfilled with excavated soils. Approximate test pit locations are depicted in blue on Plate 1, *Location of Explorations* (in pocket). A description of our field exploration and test pit logs are presented in Appendix A, *Field Exploration*.
- **Geotechnical Laboratory Testing:** Geotechnical laboratory tests were conducted on recovered bulk soil samples obtained from our test pits. 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 test pit logs in Appendix A, *Field Exploration*.
- **Engineering Analysis:** Subsurface exploration and geotechnical laboratory testing data were analyzed to develop geotechnical conclusions and provide preliminary geotechnical recommendations. A geotechnical cross-section was drawn through the western portion of the proposed warehouse footprint, as depicted on Figure 2, *Geotechnical Cross Sections A-A'*.
- **Report Preparation:** Results of our 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. Overall *Earthwork and Grading Guide Specifications* are presented in Appendix C.

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 D, *GBA's Important Information About This Geotechnical-Engineering Report*.

2.0 FINDINGS

2.1 **Regional Geology**

This site is located within the Peninsular Ranges geomorphic province of California, at the western margin of the Los Angeles Basin (Basin), a broad gently seaward-sloping coastal plain. Dibblee (DF-70, 1999) and others map this site at the eastern margin of “*older, stabilized dune and drift sand – mostly unconsolidated fine-grained sand*” (Qos). There are **no** known geologic discontinuities on this site and there are no significant natural slopes. As is the case for most of southern California, strong ground shaking has and will impact this site.

2.2 **Past and Existing Surface Conditions**

During our fieldwork in the summer of 2019, there were horse stables covering about an acre or two in the southeastern portion of this site. Otherwise, this site is undeveloped at the surface. As can be seen on the 1934 “Torrance” USGS Quadrangle, the northeast central portion of the site adjacent to the railroad was the lowest point on this parcel, roughly at 75-feet above mean sea level (MSL). Since then, a borrow pit had been excavated down to roughly 60-feet above mean sea level, while fill had been placed in the northwestern zone, as deep as reportedly (SCS, 2019) 30 feet. Current (2018) plans are presumably based on the NAVD88 vertical datum while 1934 USGS topographic quadrangles were based on the NGVD29 elevation datum. NOAA reports that the NAVD88 elevations are roughly 2½ feet higher than the NGVD29 elevations at this location; see:

http://www.ngs.noaa.gov/cgi-bin/VERTCON/vert_con.pr1

2.3 **Prior Geotechnical Exploration (2018)**

Southern California Geotechnical (SoCalGeo) prepared a June 25, 2018 “*Geotechnical Feasibility Study*” for this site. They reported drilling 20 borings up to 30-feet-deep and excavating six “trenches” up to 17½ feet deep (at locations depicted on Plate 1, in pocket). Reportedly, fill soils were encountered in seven borings and all six “trenches” to depths up to 22 feet below existing grade, with the deepest fill located in the northwestern corner of the site. SoCalGeo did not report encountering fill in the eastern and southern portion of the site adjacent to Del Amo Boulevard.

2.4 **Five Current Test Pits (2019)**

Undocumented fill soils were encountered in all five of our test pits excavated on August 22, 2019 within the existing borrow pit at locations depicted on Plate 1 (in pocket). Encountered fill was highly variable but consisted predominantly of a black “fat” (high plasticity, CH with Liquid Limit as high as 57) clay that was expansive (EI=74) with a fuel hydrocarbon odor. Some small amounts of this fill contained trace organics, which measured no-higher-than ($\leq$) 3.1% by loss-on-ignition weight. Mapped existing undocumented fill limits within the borrow pit are depicted in plan-view on Plate 1 (in pocket) and in section-view on Figure 2.

2.5 **Site-Specific Seismic Hazards**

Seismic hazards pertinent for review at this site include faulting, liquefaction, dynamic differential settlement and lateral spreading, as discussed in the following subsections.

- 2.5.1. Faulting:** This site is **not** within a currently-designated Alquist-Priolo Earthquake Fault Zone:

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

- 2.5.2. Liquefaction:** Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soils most susceptible to liquefaction are clean, loose, saturated, uniformly-graded, fine-grained sands. As shown on the March 25, 1999 *Official Map of Seismic Hazard Zones* for the Torrance Quadrangle, this site is **not** within a mapped potential liquefaction hazard zone; see:

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

Since there is no shallow groundwater (deeper than 60 feet) at this site, liquefaction can be ruled out.

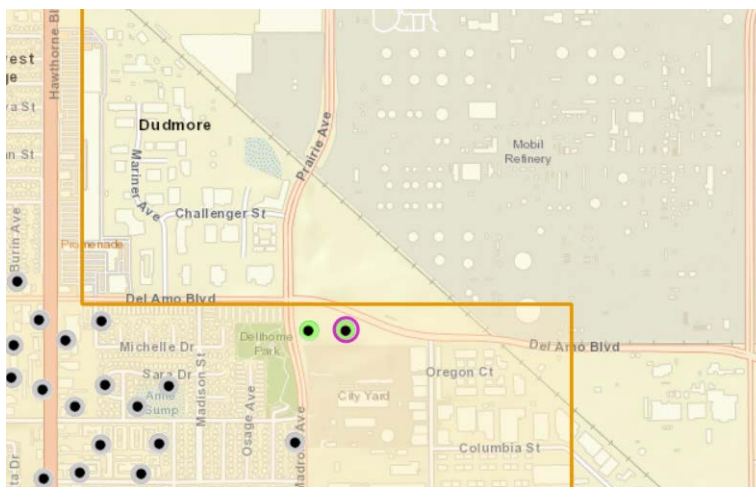
- 2.5.3. Differential Compaction:** If near-surface soils vary in composition both vertically and laterally, strong earthquake shaking can cause non-uniform compaction of soil strata, resulting in movement of near-surface soils. Local strong ground shaking will cause differential dynamic compaction. Differential compaction of fill soils relative to native sands should be considered in final design.

- 2.5.4. Lateral Spreading:** Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvial material toward an open or “free” face such as an open body of water, channel, or excavation. Generally in soils, this movement is due to failure along a weak plane, and may often be associated with liquefaction. As cracks develop within the weakened material, blocks of soil displace laterally towards the open face. Cracking and lateral

movement may gradually propagate away from the face as blocks continue to break free. Generally, failure in this mode is analytically unpredictable, since it is difficult to determine where the first tension crack will occur.

Since liquefaction has been ruled out for this site, which will also be graded relatively flat, lateral spreading can also be ruled out.

- 2.5.5. Oil Wells:** Reviewing the California Division of Oil, Gas and Geothermal Resources (DOGGR) well finder, we do not see wells on site, although there were numerous wells south and west of this site:



<http://maps.conservation.ca.gov/doggr/WellFinder/Index.html?api=0403716805&config=WellFinder#/-118.34054/33.84932/16>

However, Psomas's March 16, 2018 ALTA survey depicts several "oil well risers," which should be researched to find out what these are (or perhaps Psomas can explain).

3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 Conclusions and Findings Summary

This site is **not** located within a currently-designated Alquist-Priolo Earthquake Fault Zone of potential surface fault rupture, nor zone of potential seismically-induced liquefaction. Groundwater has been relatively deep at this site, presumably on the order of 60- to 100-feet below existing site surface. This site is geologically and geotechnically characterized as older stabilized dune sands (Dibblee's "Qos" and CGS's "Qoe"), and site topography is consistent with undulating dunes. The 1934 USGS "Torrance" Quadrangle shows a depression at and below the 75-foot elevation contour (NGVD29), filled by the railway at the

northeast-center portion of the site; with the eastern, southern and western portions of the site roughly at elevation 100-feet. Psomas's March 16, 2018 (flown November 18, 2015) ALTA topographic survey for this site shows the lowest surveyed point in the borrow pit area at elevation 59.6 feet above mean sea level (presumably NAVD88; note that NAVD88 elevations are 2.4 feet higher at this site compared to the NGVD29 datum).

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3.2 **Warehouse Foundation Recommendations Summary**

Best standard geotechnical practice is to remove all undocumented fill under proposed building footprints, which would extend down to elevation 54 feet above mean sea level in the central portion of the existing borrow pit. However, this would be six feet below the allowable excavation threshold at elevation 60 feet. Existing fill was found to be predominantly expansive clay, which might expand, but not consolidate much. Conservatively assuming an unlikely worst-case 5% consolidation in this undocumented old fill, 6-feet of this fill could presumably settle 3½-inches in the center worst case fill depth; 3½-inches in the center worst case fill depth; which would be differential settlement relative to the dense native sands with negligible long-term settlement. Therefore, **if** excavation below elevation 60 feet is not allowed, **then** (and only then) alternative geotechnical mitigations are recommended, as follows:

- **Adjust Warehouse Footprint (Westerly Wall):** While central columns, supporting a lightly-loaded roof (see Table 1), for this warehouse will be



susceptible to excessive differential settlement, perimeter tilt-up-concrete bearing walls will be the most sensitive to differential settlement. Therefore, we recommend that the currently proposed westerly wall of the warehouse be moved 20 feet west either by moving the whole warehouse footprint or perhaps increasing the warehouse size. This would move the west wall continuous footing outside of the influence of any subsiding remaining existing fill soils. Note that as an option if the footprint cannot be moved west, then the westerly wall continuous footing could be stepped down to elevation 90 feet adjacent to the borrow pit area, to provide a ½:1 (horizontal:vertical) setback of the interior edge of this new footing relative to the closest perimeter of existing undocumented fill left in place.

- **Earth Surcharge (Northerly Wall):** Based on our design-team discussions, we understand it is impractical to move the northern wall of the proposed warehouse to avoid overlying remaining undocumented fill. Pile foundations are prohibited, since drilling through the remaining undocumented fill is prohibited. Therefore, we recommend that the remaining undocumented fill be consolidated by earth-surcharging to mitigate total and differential settlement of existing fill along the western portion of the warehouse northern perimeter wall. An earth berm 25-feet-high above finish grade is proposed as surcharge; encompassing the portion of the warehouse footprint that will overlie remaining undocumented fill, extending at-least 25-feet beyond the warehouse footprint in areas of remaining undocumented fill. Settlement of finish grade under this surcharge should be monitored bi-weekly (by CTM 112 pipe-riser settlement devices), for **not-less-than (>) six months** and until diminished asymptotic settlement occurs, with less-than (<) one-tenth-of-an-inch (0.1) differential-settlement measured over a 30-day period.
- **Interior Tubular Steel Roof Columns:** While roof columns would be inevitable over this remaining undocumented fill zone, we would recommend that any heavy roof HVAC units or other heavy roof installations be avoided over this remaining undocumented fill zone. Tubular steel columns over this zone could be fitted with screw-jacks that can be adjusted if needed from time to time to compensate for differential settlement ranging from zero at the **red dot** perimeter to conceivably 3½-inches in the interior of this remaining undocumented fill zone (at SoCal Geo's B-9). Newly-placed compacted fill overlying this zone of undocumented fill would be geogrid reinforced to reduce differential-settlement. Concrete slabs-on-grade, only over this area of remaining undocumented fill, should be structurally isolated from other column bays; yet inter-doweled to reduce differential vertical movement between column bay joints.

Site sands are non-expansive and have desirable consolidation characteristics (compared to clay) when properly compacted. Therefore, we recommend that site finish grades be lowered to provide more balanced earthwork requiring less import.

Torrance is known to have undesirable highly expansive clays not far from this site. Such clays are undesirable due to adverse expansion and consolidation characteristics, and will consolidate over time relative to sand. If it is absolutely unavoidable to import clays to the site, then clays should be placed within the pit at least 15 feet below finish grade. Even with this design, clay consolidation will need to be evaluated specific to the grading scheme and structural requirements for differential settlement.

Although conventional tilt-up structures with conventional shallow spread footings are the likely structure alternative, differential settlement of new fill soils (presumably 35- to 40-feet-deep) needs to be evaluated relative to cut zones in older eolian (stabilized) sands. Compaction to 95% relative compaction (relative to the ASTM D1557 modified Proctor laboratory maximum density) in deeper fills is recommended (in more detail) in this report.

More detailed geotechnical recommendations are presented in the following sections of this report.

3.3 **Earthwork**

Overall *Earthwork and Grading Guide Specifications* are presented in Appendix C. Site-specific earthwork recommendations are presented in the following subsections.

3.3.1. Clearing and Site Preparation: Portions of the site to receive fill for support of buildings, retaining walls and/or pavements, should be cleared of all surface and subsurface deleterious materials including debris, brush and associated roots. Minimum recommended overexcavation is as follows:

- **Pavements:** Existing soils should be excavated to **at least 2 feet** below existing or finish grade, whichever is lower, in areas of proposed pavements.
- **Buildings and Retaining Walls:** Existing soils should be excavated to **at least 5 feet** below existing or finish grade, whichever is lower, in areas of proposed warehouse building(s) and/or retaining walls.

Any existing undocumented fill soils would also need to be completely removed from under buildings and retaining wall footings (only), for support of shallow spread footings. Rubble piles should be legally disposed of off-site or screened and used in properly compacted fill. Shallow zones of organic soils must also be removed from building and retaining wall footprints (if and where encountered). Trees and brush will need to be removed from building, retaining

wall and pavement areas, including complete removal of root-balls and associated organic soils.

However, as previously indicated in Section 3.2 of this report, these recommended overexcavation might be required to be limited to not extend below elevation 60-feet. In those areas, the exposed surface should be scarified, moisture conditioned and compacted as described in the following section, followed by laying-out a geogrid as described in the following section.

- 3.3.2. Subgrade Preparation:** After the site has been properly cleared, stripped, and necessary excavations have been made as indicated above, the exposed surface soils in those areas to support pavements should be scarified to a depth of 8 inches, moisture conditioned to near optimum, and compacted in accordance with the recommendations for fill presented in Section 3.3.4 “Compaction,” later in this report.

In areas where fill or pavements may be constructed over undocumented fill soils, exposed fill subgrades should be stabilized with a S2 biaxial geogrid (such as a Tensar® BX 1100 bi-axial geogrid or equivalent; see Table 213-5.2 (D) “BIAXIAL” of the *Standard Specifications for Public Works Construction*, 2018 Edition) in combination with crushed rock, as specified in Section 200-1.2 of the *Standard Specifications for Public Works Construction*, (“Greenbook”), 2018 Edition. A 4-inch-thick layer of ¾-inch crushed rock should then be placed atop the geogrid. Thicker gravel layers may be required in more heavily loaded areas, or in zones of very soft subgrades. Depending on the time of year and rainfall, pavement subgrades may also need to be stabilized with crushed rock and/or geogrids, to facilitate pavement subgrade and base compaction. Finished compacted subgrade should be firm and non-yielding under weight of compaction equipment.

- 3.3.3. Material for Fill:** All on-site soils having an organic content less-than (<) 3 percent by weight are suitable for use as fill at the site. In general, fill material should not contain rocks or lumps larger than 3-inches (7.6 cm) in greatest dimension, with no more than (≤) 15 percent larger than 2½ inches (6.4 cm). Imported and non-expansive fill material should be predominantly granular with a Plasticity Index of 15 or less, and/or an Expansion Index (EI) of 30 or less.

If desired to recycle imported asphalt or Portland cement as engineered fill, we recommend that these materials be thoroughly broken down and mixed with on-site or import soil. In general, recycled asphalt or concrete should be ground or pulverized to particles less than 3 inches (7.6 cm) in greatest dimension, with no more than 25 percent larger than 2½ inches (6.4 cm). Recycled material should be thoroughly mixed with a sufficient amount of soil, such that there is no more than 40 percent by weight of recycled material in the final mix.

- 3.3.4. Compaction:** All fill, as well as scarified surface soils in those areas to receive fill, should be compacted to at least **90 percent** relative compaction as determined by the ASTM D1557 Test Method (modified Proctor). Fill should be placed in lifts no greater than 8-inches in uncompacted thickness at a moisture content near the laboratory optimum. Each successive lift should be firm and non-yielding under the weight of the construction equipment.

Fills placed more-than (>) 15-feet below finish grade should be compacted to at least 95 percent relative compaction as determined by the ASTM D1557 Test Method (modified Proctor).

The upper 6 inches of subgrade in pavement areas and all aggregate base should also be compacted to at least **95 percent** relative compaction (ASTM D1557. Aggregate base and all import soils should be compacted at a moisture content near the laboratory optimum.

- 3.3.5. Pipeline Backfilling:** Pipeline trenches should be backfilled with compacted fill in accordance with this report, and applicable *Standard Specifications for Public Works Construction* (Greenbook), 2018 Edition standards. Backfill in and above the pipe zone should be as follows:

- **Pipe Zone:** Pipe bedding zone within warehousing facility 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 2018 Edition of the *Standard Specifications for Public Works Construction* (Greenbook). Due to expansive clays, sand bedding for conduits should **not** be allowed on this site within warehouse building footprints and at least 10-feet beyond building footprints. CLSM bedding should be placed to 1-foot (0.3 m) over the top of the conduit and vibrated. CLSM should **not** be 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 (>) 2-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 $\frac{3}{4}$ -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. Backfill above the pipe zone should

not be jetted. In any case, backfill above the pipe zone (bedding) should be observed and tested by Leighton Consulting, Inc.

3.3.6. Warehouse Footprint (Over Undocumented Fill) Surcharging: We recommend that the remaining undocumented fill (under the warehouse footprint) be consolidated by earth-surcharging to mitigate total and differential settlement of existing fill along the western portion of the warehouse northern perimeter wall. This western portion of the warehouse footprint should be surcharged with an earth berm and monitored for settlement during no-less-than ($\geq$) six months. Settlement monitoring should be performed by a California-licensed Professional Land Surveyor (PLS), with settlement monuments installed by Leighton Consulting, Inc. or another competent geotechnical instrumentation firm. Proposed settlement monitoring systems (CTM 112 pipe-riser settlement devices) are depicted at the end of Appendix C. Surcharges can be removed once demonstrated by settlement monitoring that asymptotic reduction in settlement is occurring with time, based on Leighton Consulting, Inc. settlement monitoring review and evaluation. At least 25 feet of fill at a moist density no-less-than ($\geq$) 125 pounds-per-cubic-foot (pcf) should be placed covering the whole proposed warehouse footprint over undocumented fill, extending at least 25-feet horizontally beyond the warehouse perimeter in the zone of remaining undocumented fill.

3.3.7. Construction Observation: Leighton Consulting, Inc. should observe all grading and test all earthwork, to check that subgrades are properly prepared, selected fill materials are satisfactory, and that placement and compaction of fill has been performed in accordance with our recommendations and project specifications. Sufficient notification to us prior to earthwork is essential. Project plans and specifications should incorporate our recommendations contained in this report.

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

3.4 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 either the 2016 or 2019 Editions of the California

Building Code (CBC). Seismic design parameters based on the 2016 and 2019 CBC methodology are tabulated below:

Table 2. CBC Site-Specific Seismic Parameters

CBC Site-Specific Seismic Design Parameters	2019	2016
Site Longitude (decimal degrees) West	-118.3427	
Site Latitude (decimal degrees) North	33.8497	
Site Class Definition (Chapter 20 of ASCE 7-16)	D	
Mapped Spectral Response Acceleration at 0.2s Period, S_s (Figure 1613.3.1(1))	1.786	1.572
Mapped Spectral Response Acceleration at 1s Period, S_1 (Figure 1613.3.1(2))	0.638	0.586
Short Period Site Coefficient at 0.2s Period, F_a (Table 1613.2.3(1))	1.0	
Long Period Site Coefficient at 1s Period, F_v (Table 1613.2.3(2))	1.7*	1.5
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS} (Eq. 16-36)	1.786	1.572
Adjusted Spectral Response Acceleration at 1s Period, S_{M1} (Eq. 16-37)	1.085*	0.880
Design Spectral Response Acceleration at 0.2s Period, S_{DS} (Eq. 16-38)	1.191	1.048
Design Spectral Response Acceleration at 1s Period, S_{D1} (Eq. 16-39)	0.723*	0.586
Long Period, T_L (seconds)	8	
Seismic Design Category (2016 CBC Section 1613.3.5)	D	

*Requires a site-specific probabilistic and deterministic seismic hazard analysis in accordance with Section 11.4.8 of ASCE 7-16.

3.5 Shallow Spread Footing Foundations (Overlying Compacted Fill)

For tilt-up warehousing facility and retaining walls constructed over at least ($\geq$) 5-feet of compacted fill, shallow spread footings can be designed using the following geotechnical design parameters and recommendations:

- 3.5.1. Spread Footings Minimum Embedment and Width:** New conventional shallow spread footings may be used to support warehousing facility and/or retaining walls, bearing solely on at least ($\geq$) a 5-foot-thick layer of properly compacted fill over undisturbed native soils. These recommendations are for dead plus sustained loads less-than ($<$) 10 kips-per-foot of wall for continuous footing, or 75-kips for isolated column loads. Footings should be embedded at least 18-inches below lowest adjacent grade, with a minimum width of 12-inches wide for buildings or 30-inches for retaining walls (possibly wider for overturning resistance).
- 3.5.2. Spread Footings Allowable Bearing Pressure:** For footings bearing solely on at least ($\geq$) a 5-foot-thick layer of properly compacted fill over undisturbed native soils, a net allowable bearing pressure of 2,500 pounds-per-square-foot (psf) may be used for static and sustained live loads, based on minimum

embedment depth and widths described above. These allowable bearing pressures are for total dead loads and frequently applied live loads, and can be increased by one-third for short duration wind loads. Our recommended maximum allowable bearing pressures are net values; so weight of the portion of footings below lowest adjacent finish grade need not be added to design loads for structural analyses.

3.5.3. Spread Footing Lateral Load Resistance: Lateral (horizontal) loads on foundations embedded in properly compacted fill may be resisted by both frictional resistance along the base of the footing and passive resistance in properly compacted fill adjacent to the sides of footings. Frictional resistance between base-of-footings poured (cast) on properly compacted silty sand fill may be computed using a coefficient of friction of 0.45. Passive resistance may be computed using an equivalent fluid pressure of 300 pounds-per-square-foot per foot of embedment below lowest adjacent grade (pcf), assuming there is constant contact between these footings and properly compacted fill soils. Passive pressure should not exceed 2,500 psf. These values may be increased by one third when considering wind and seismic forces. Both friction and passive values have already been reduced by a factor-of-safety of 1½, and can be used in combination.

3.5.4. 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 30° can be used along with cohesion of 300 pounds-per-square-foot (psf), to model properly compacted backfill soil's shear strengths. A factor-of-safety has not been applied to these values, although cohesion was reduced due to potential for shrinkage and desiccation-cracking, in addition to potential for backfill soil-type variability.

3.5.5. Spread Footing Settlement Estimates: We will need column loads and maximum fill depths under warehouses to provide settlement estimates. Since settlement is a function of footing sustained load, size and contact bearing pressure, differential settlement can be expected between adjacent dissimilarly loaded walls where a large differential loading condition exists. These settlement estimates can be evaluated by Leighton Consulting, Inc. when foundation plans, grading plans and actual loads for these proposed improvements become available.

3.6 Retaining Walls

Specific design recommendations for earth retaining structures are presented in the following subsections.

- 3.6.1. Lateral Earth Pressures:** We recommend that retaining walls be backfilled with non-expansive ($EI \leq 30$) soil, and constructed with a backdrain as described in Section 3.5.4, below. Based on these recommendations, the following parameters may be used for retaining wall design:

Table 3. Retaining Wall Design Earth Pressures

Retaining Wall Condition	Equivalent Fluid Pressure (pounds-per-cubic-foot)	
	Level Backfill	2:1 Backfill
Active (cantilever)*	35	50
At-Rest (braced)*	55	65
Passive (resistance)	300**	

*Only in properly compacted non-expansive ($EI \leq 30$) backfill.

**For level toe grades only in properly compacted fill, not to exceed 2,000 psf

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 (e.g. valve vaults and manholes) should be designed using the at-rest condition. Passive pressure is used to compute soil resistance to lateral structural movement.

- 3.6.2. 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 an adjacent structure or traffic, should be considered in design of the retaining wall. Vertical surcharge loads behind the retaining wall on or in the backfill within a 1:1 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.
- 3.6.3. Sliding and Overturning:** Total depth of retained earth for design of walls 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 soil unit weight of 120 pcf may be assumed for calculating the actual weight of the soil over the wall footing, assuming drained conditions, for properly compacted backfill.

- 3.6.4. Retaining Wall Drainage:** Adequate drainage may be provided by a subdrain system positioned behind the walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with pervious backfill material described in Section 300-3.5.2 of the *Standard Specifications for Public Works Construction* (Greenbook), 2018 Edition. This pervious backfill should extend at least 2 feet out from the wall and to within 2 feet of the outside finished grade. This pervious backfill and pipe should be wrapped in filter fabric, such as Mirafi 140N or equivalent, placed as described in Section 300-8 of the *Standard Specifications for Public Works Construction* (Greenbook), 2018 Edition. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain or Enkadrain drainage geocomposites, or similar, may be used for wall drainage as an alternative to pervious backfill or drain rock backfill, particularly where horizontal space is limited adjacent to shoring or near vertical cuts. Drainage geocomposites should be connected to a perforated drainpipe at the base of the wall.

- 3.6.5. Retaining Wall Incremental Seismic Loads:** Seismic incremental loads need **not** be added to retaining walls with stem heights on the order of ($\leq$) 6-feet or less, with adjacent level backfill. However, at the discretion of the project Structural Engineer (SE), incremental seismic earth pressures of 25 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). This incremental load should be added to active static earth pressures, and need not be added to at-rest earth pressures.

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.

3.7 **Concrete Slab-On-Grade**

Depending on rack or other point loading, slabs-on-grade should be designed by the project Structural Engineer. In general, slabs-on-grade should have the following minimum recommended components:

- **Subgrade:** Slab-on-grade sandy subgrade soil should be moisture conditioned to or within 3% of optimum moisture content, to a minimum depth of 24 inches within building footprints, then compacted to 90% of the modified Proctor (ASTM D1557) laboratory maximum density prior to placing either a moisture barrier, steel and/or concrete.
- **Moisture Barrier:** **Only in areas where moisture-sensitive floor coverings are used or equipment is installed**, 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.
- **Reinforced Concrete:** A conventionally reinforced concrete slab-on-grade with a thickness of at least 5-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 24-inches on-center, each direction (perpendicularly), mid-depth in the slab. A modulus of subgrade reaction (k) as a linear spring constant, of 125 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 potential subgrade expansion/shrinkage. Where possible, slabs-on-grade should be allowed to “float” on the subgrade to allow for differential vertical expansion/shrinkage of the subgrade. However, monolithic pours with spread footings have been recommended for liquefaction mitigation.

Minor cracking of concrete after curing due to expansion, 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.

3.8 Pavements

All pavement areas should be supported on a zone of newly placed compacted fill extending at least 3 feet below existing or finish grade, whichever is lower. A zone of properly compacted fill should then be backfilled in lifts. Pavement subgrades and base should be compacted to **95 percent** of the ASTM D1557 (modified Proctor) laboratory maximum density for these materials.

- 3.8.1. Asphalt Concrete Pavements:** Considering the effects of mixing of onsite soils, a representative R-value of 30 and Traffic Indexes (TI) of 5 through 8 were used to calculate pavement sections as listed below:

Table 4. Asphalt Pavement Section Thickness

General Traffic Condition	Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Total Thickness (inches)
Automobile Parking	5.0	3.0	6.0	9
	5½	3.0	7.0	10
Truck Access & Parking Areas	6.0	3½	8.0	11½
	6½	4.0	9.0	13
	7.0	4.0	10.0	14
	7½	4½	11.0	15½
	8.0	5.0	12.0	17

Final pavement thickness should be based on subgrade R-value test results performed on representative finished sub-grade soils. Appropriate Traffic Index (TI) data should be selected by the project Civil Engineer or traffic engineering consultant for final design of pavement sections. These Caltrans pavement-section design calculations were based on a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. Portland cement concrete should be used, rather than asphalt, in point and impact load areas such as loading docks and trash truck bin loading areas.

Asphalt concrete and aggregate base should conform to *Caltrans Standard Specifications* (2018 Edition) Sections 39 and 26-1.02A, respectively; see:

http://www.dot.ca.gov/hq/esc/oe/construction_contract_standards/std_specs/2015_StdSpecs/2015_StdSpecs.pdf

Aggregate base should have an R-value of at least ($\geq$) 78. Minimum relative-compaction requirements for aggregate base should be 95 percent of the modified Proctor laboratory maximum density as determined by ASTM D 1557.

As an alternative, asphalt concrete can conform to Section 203-6 of the *Standard Specifications for Public Works Construction* (Green Book), 2018 Edition. Crushed aggregate base or crushed miscellaneous base can conform

to Sections 200-2.2 and 200-2.4 of the *Standard Specifications for Public Works Construction* (Green Book), 2018 Edition, respectively.

- 3.8.1. Portland Cement Concrete (PCC) Pavements:** For preliminary planning purposes, fire lanes may be constructed of Portland Cement Concrete (PCC) with a minimum thickness of 6½-inches assuming light axle loads and an average daily truck traffic (ADT) of less than 500. For medium/heavy axle loads and an ADT of 500 or more, a minimum PCC thickness of 8-inches should be used, such as for trash corrals and trash truck aprons, loading docks, etc.. All PCC pavements should have a minimum 28-day concrete compressive strength of 3,250 pounds-per-square-inch (psi), and have appropriate joints and saw cuts in accordance with either Portland Cement Association (PCA) or American Concrete Institute (ACI) guidelines. PCC subgrade should be compacted to 95 percent relative compaction in the upper 12-inches.

These PCC sections should be re-evaluated based on the precise grading plan, which indicates locations of concrete pavements. We recommend that the ADT be confirmed by the project civil designer or traffic consultant prior to completion of plans. For truck lanes and ramps, a 4-inch (minimum) layer of Class 2 aggregate base at 95 percent relative compaction should be considered beneath the PCC paving. This 4-inch layer of Class 2 aggregate may be used beneath other areas of PCC pavement to improve performance. Additional details should be added to the plans indicating pavement thickness transitions, pavement joint dowels, expansion joints and sawcut joints. Use of concrete cutoff or edge barriers should be considered at the perimeter of common parking or driveway areas when abutting either open (unfinished) or landscaped areas.

4 . 0 C O N S T R U C T I O N C O N S I D E R A T I O N S

4.1 **Additional Subsurface Exploration**

Additional subsurface exploration and geotechnical analyses at this site is recommended to further evaluate dumped fill soils within the borrow pit and to verify shallow soil conditions at the perimeter of the warehouse footprint. Several backhoe test pits are proposed to do this; at which time, this report can be updated based on refined Civil and Structural plans.

4.2 **Plans, Specifications, and Construction Review**

Because subsurface conditions may vary considerably from those predicted by subsurface exploration by others, and to check that our report recommendations have been properly implemented, we recommend that we be retained to 1) review final grading and foundation plans and specifications, and 2) observe and test all

earthwork. In addition, the assumed and/or actual geotechnical conditions can be greatly affected by the construction process and seasonal weather changes. For the above reasons, our geotechnical recommendations are contingent upon Leighton Consulting, Inc. providing geotechnical observation and testing services during construction.

4.3 Temporary Excavations

The contractor is responsible for all temporary slopes and trenches excavated at the site and the design of any required temporary shoring. Shoring, bracing and benching should be performed by the contractor in accordance with the 2018 Edition of the *California Construction Safety Orders*, Article 6, or more current. During construction, soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor shall be responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Soil types will vary, but **Type C** soils can be expected at this site. CAL/OSHA requires slopes in Type C soils to be cut back at gradients of 1½:1 (horizontal:vertical), if workers are to enter the excavation. Close coordination between the competent person and Leighton Consulting, Inc. should be maintained to facilitate construction while providing safe excavations.

5 . 0 L I M I T A T I O N S

This report was based on subsurface exploration by others at this time, in addition to information from prior geotechnical work by others. Such information is, by necessity, incomplete. The nature of many sites is such that differing soil or geologic conditions can be present within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, findings, conclusions and recommendations presented in this report are based on the assumption that Leighton Consulting, Inc. will provide geotechnical observation and testing during construction.

Until reviewed and accepted by the City of Torrance, this report may be subject to change. Changes may be required as part of the City 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 the City.

This report was prepared for Prologis, Inc. based on their needs, directions and requirements at the time of our exploration, in accordance with generally accepted

geotechnical engineering practices at this time in Los Angeles County. This report is not authorized for use by, and is not to be relied upon by, any party except Prologis, Inc., 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.

Environmental services were not included as part of this study. This report was prepared for the sole use of Prologis, Inc. and their design team for application to design of the proposed warehousing facility in accordance with generally accepted geotechnical engineering practices at this time in Los Angeles County.

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Project: 12452.002	Eng/Geol: TCB/JH
Scale: 1" = 500'	Date: September 2019
Base Map: ESRI ArcGIS Online 2019 Thematic Information: Leighton Author: Leighton Geomatics (btran)	

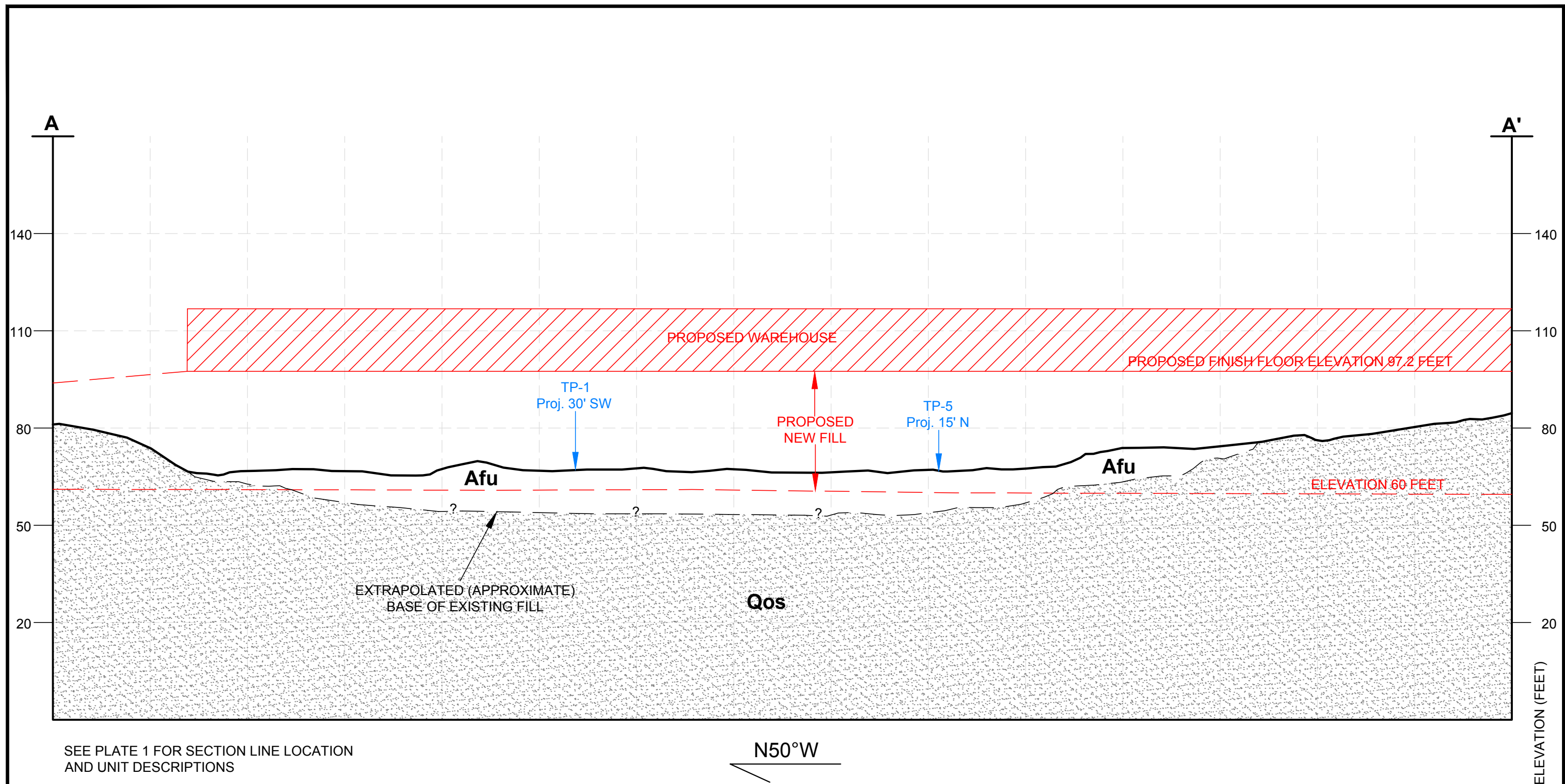
SITE LOCATION MAP

Proposed Warehousing Facility
3041± Del Amo Boulevard
Torrance, Los Angeles County, California
APN: 7352-015-004, 5 and 6

Figure 1



Leighton



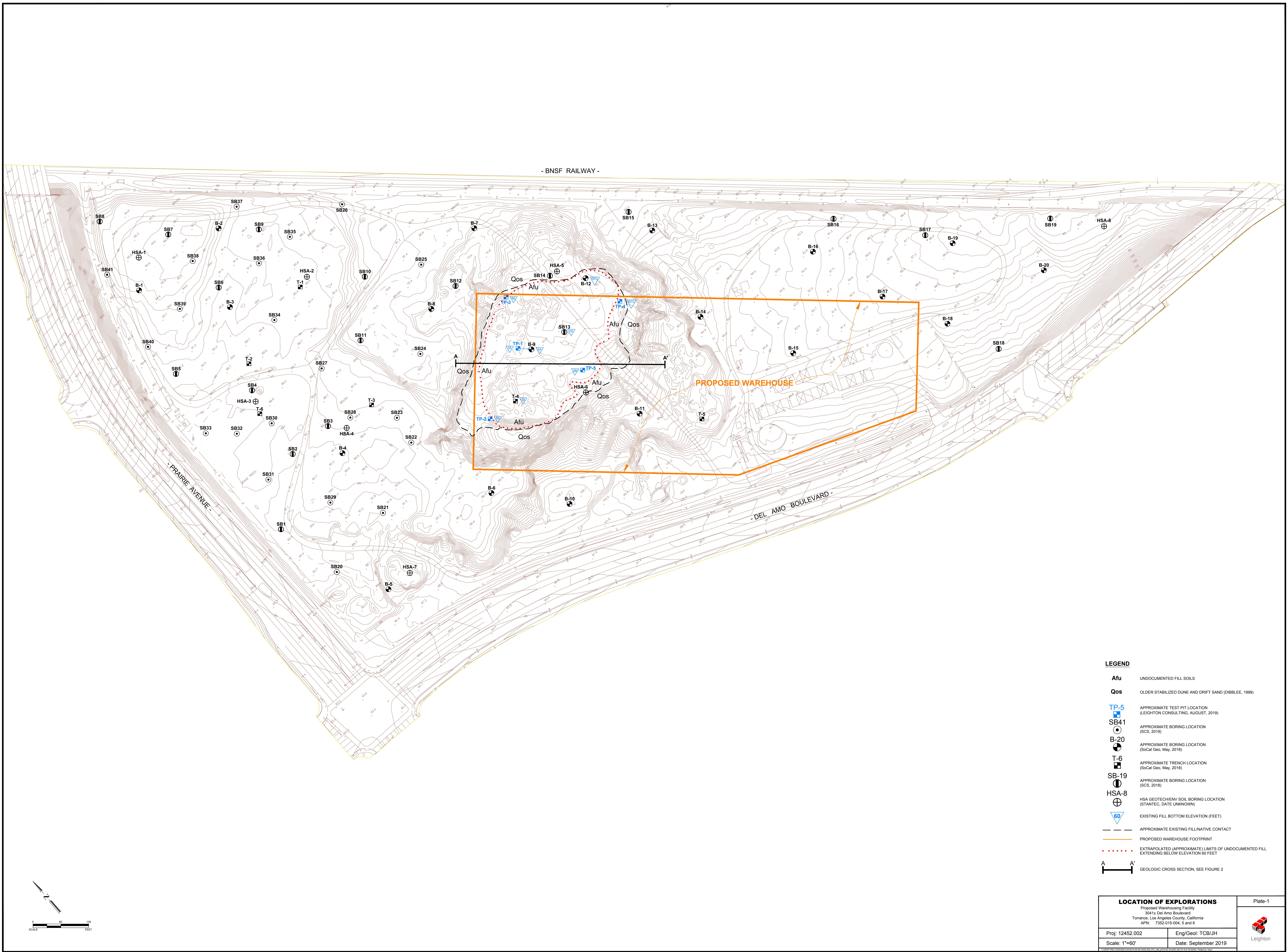
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Reference:	

GEOTECHNICAL CROSS SECTION A-A'

Proposed Warehousing Facility
 3041± Del Amo Boulevard
 Torrance, Los Angeles County, California
 APN: 7352-015-004, 5 and 6

Figure 2





LEGEND

Afu UNDOCUMENTED FILL SOILS

Qos OLDER STABILIZED DUNE AND DRIFT SAND (DIBBLEE, 1999)

TP-5 APPROXIMATE TEST PIT LOCATION (LEIGHTON CONSULTING, AUGUST, 2019)

SB41 APPROXIMATE BORING LOCATION (SCS, 2019)

B-20 APPROXIMATE BORING LOCATION (SoCal Geo, May, 2018)

T-6 APPROXIMATE TRENCH LOCATION (SoCal Geo, May, 2018)

SB-19 APPROXIMATE BORING LOCATION (SCS, 2019)

HSA-8 HSA GEOTECH/HEAVY SOIL BORING LOCATION (STANTEC, DATE UNKNOWN)

60 EXISTING FILL BOTTOM ELEVATION (FEET)

--- APPROXIMATE EXISTING FILL/NATIVE CONTACT

--- PROPOSED WAREHOUSE FOOTPRINT

..... EXTRAPOLATED (APPROXIMATE) LIMITS OF UNDOCUMENTED FILL EXTENDING BELOW ELEVATION 60 FEET

A A' GEOLOGIC CROSS SECTION, SEE FIGURE 2

LOCATION OF EXPLORATIONS		Plate-1
Proposed Warehousing Facility 3041s Del Amo Boulevard Torrance, Los Angeles County, California APN: 7352-015-004, 5 and 6		
Proj: 12452.002	Eng/Geol: TCB/JH	
Scale: 1"=60'	Date: September 2019	

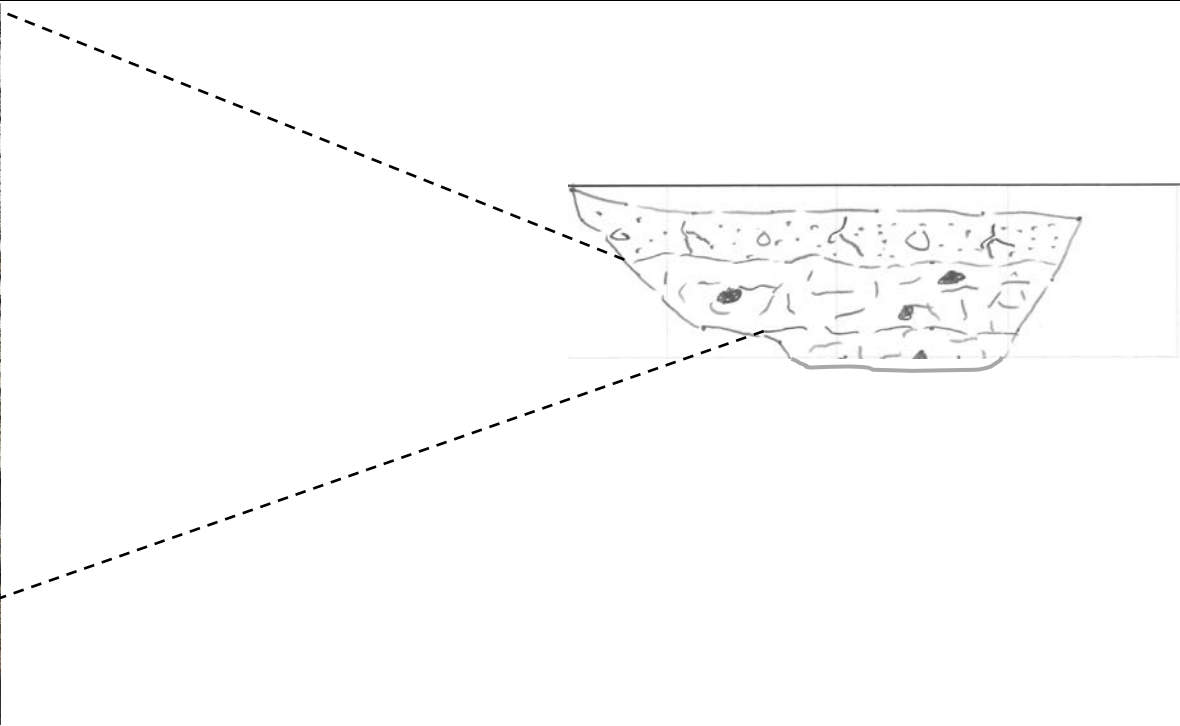
APPENDIX A

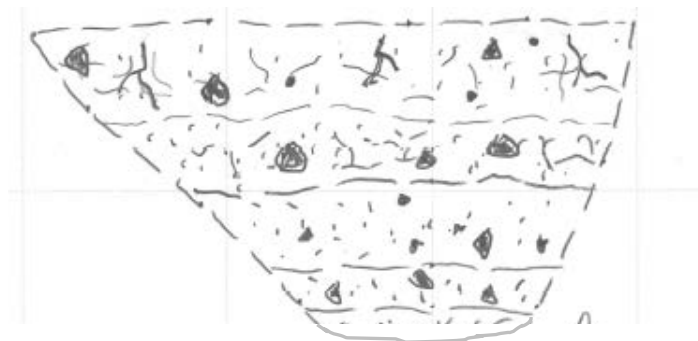
FIELD EXPLORATION

Our field exploration consisted of a surface reconnaissance, and subsurface exploration by excavating five test pits with a backhoe. Prior to beginning fieldwork, we marked proposed test pit 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, *Location of Explorations* (in pocket), and describe in more detail below:

Our Certified Engineering Geologist (CEG) visited this site to observe exposed geologic conditions. On August 22, 2019, five backhoe-excavated test pits were excavated, logged and sampled to depths ranging from 5- to 14-feet below existing grades within the borrow pit area. Encountered soils were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Bulk soil samples were also collected from these test pits. Soil samples were transported to our Irvine geotechnical laboratory (DSA LEA-63) for geotechnical testing. Test pit logs are included as part of this appendix. After sampling and logging, all five test pits were immediately backfilled with excavated soils.

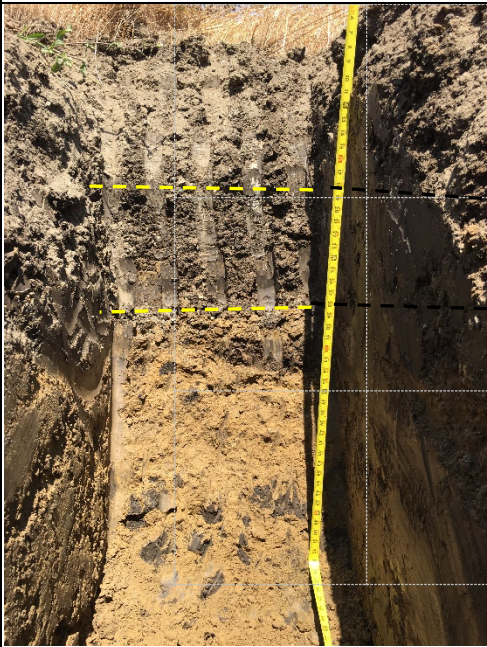
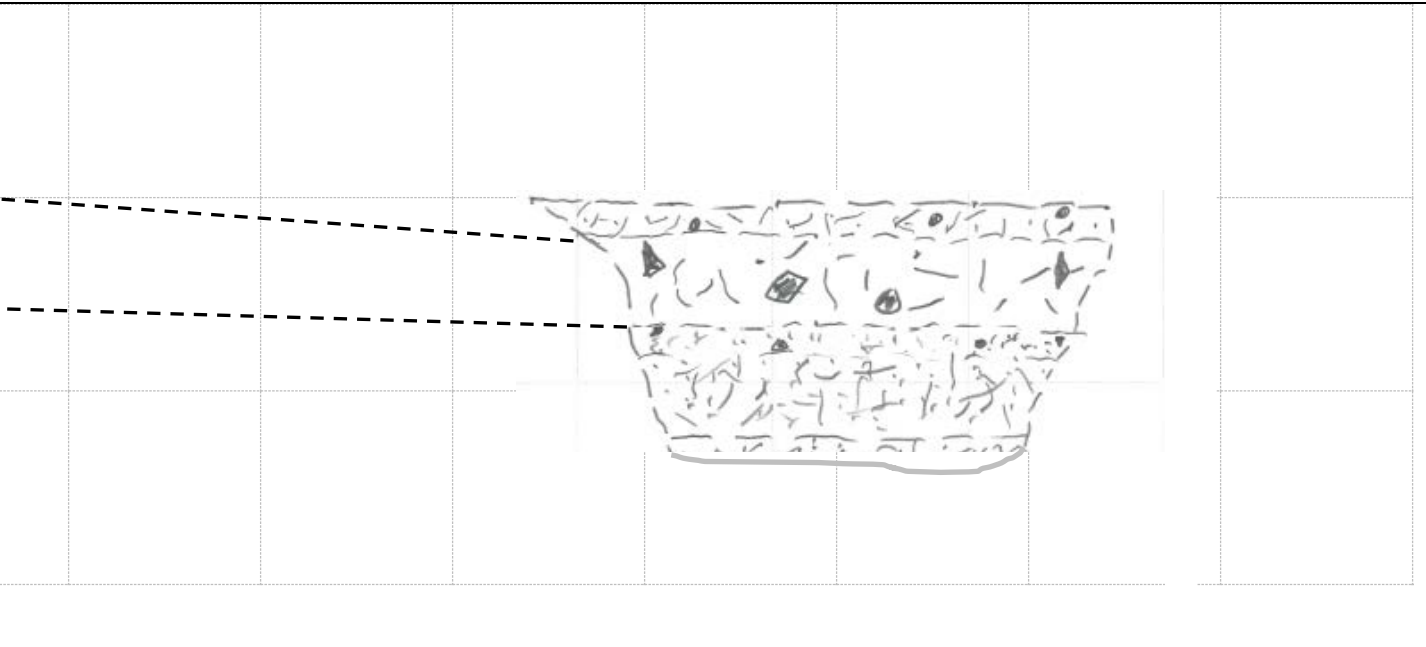
Attached test pit logs depict subsurface conditions at the specific approximate locations noted on our Plate 1, *Location of Explorations* (in pocket), during designated date(s) of exploration. Subsurface conditions at other site locations may differ from those encountered in our explorations. It is 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.

 Leighton Consulting, Inc. A LEIGHTON GROUP COMPANY	Project Name: Prologis Warehouse		Logged by: JLH		TEST PIT TP-1		
	Project Number: 12452.002		Elevation: 66 feet amsl				
	Equipment: Backhoe		Location/Grid: Adjacent prior boring B-9		Sample Results		
Earth Material Description: These earth material descriptions apply only to an exploration location 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.				Unified Soil Classification	Sample No.	Moisture (%)	Laboratory Tests
Earth Materials Exposed On: August 22, 2019							
IMPORTED ARTIFICIAL FILL – UNDOCUMENTED (0-13') @0'-18": SAND (SW), with gravel, light gray, dry, loose, scattered rootlets @18"-41": CLAY (CL), medium brown to black, moist, firm, local shale fragments @41"-72": CLAYEY SAND (SC), medium orange/brown, moist, med. dense, med-grained, cobble-size inclusions of black clay topsoil, 65% fines, LL=40, PI=27				CL-ML	GS-1 @6 feet	15	#200
Graphical Representation: Total Depth: 6 feet				Scale: 1-inch=5-feet	Level Surface	Trend: N20°W →	
 							

 Leighton Consulting, Inc. A LEIGHTON GROUP COMPANY	Project Name: Prologis Warehouse		Logged by: JLH		TEST PIT TP-2		
	Project Number: 12452.002		Elevation: 68 feet amsl				
	Equipment: Backhoe		Location/Grid: Western borrow pit		Sample Results		
Earth Material Description: These earth material descriptions apply only to an exploration location 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.				Unified Soil Classification	Sample No. Depth (feet)	Moisture (%)	Laboratory Tests
IMPORTED ARTIFICIAL FILL – UNDOCUMENTED (0-8') @0-2.4': CLAY (CL), with sand, medium gray and brown mottled, dry, soft, scattered rootlets, local pebbles, ¾-inch rock, with concrete and asphaltic rubble @2.4-4': CLAYEY SAND (SC), medium orange/brown, slightly moist, loose, fine-grained, scattered clay topsoil clasts @4': CLAYEY SAND (SC), light olive, moist, loose, scattered gravels and asphalt clasts, 3.1% organic content @6': asphalt fragment in same matrix as above @7': CLAY (CL), with sand, light orange-brown, moist, lift containing caliche @7-8': SAND (SW), dark brown, slightly moist, loose, asphalt fragment.				(CL)s (SM)g (CL)s	GS-1 @4 GS-2 @7-8 B-1 @4-6	16 8 5	ORG % #200
QUATERNARY TERRACE DEPOSITS (8') @8': SAND (SP), medium orange-brown, slightly moist, loose, friable, well sorted, fine to medium-grained, horizontal laminae, iron-stained, thinly laminated, non-cohesive, 1½% fines (passing #200 sieve)							
Graphical Representation: Total Depth: 8 feet 							
Scale: 1-inch=5-feet 							
Level Surface 							
Trend: S55°E 							

 Leighton Consulting, Inc. A LEIGHTON GROUP COMPANY	Project Name: Prologis Warehouse Logged by: J LH Project Number: 12452.002 Elevation: 62 feet amsl Equipment: Backhoe Location/Grid: Northwestern borrow pit		TEST PIT TP-3			
Earth Material Description: These earth material descriptions apply only to an exploration location 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 Results			
Earth Materials Exposed On: August 22, 2019		Soil Unit	Unified Soil Classification	Sample No. Depth (feet)	Moisture (%)	Laboratory Tests
<u>IMPORTED ARTIFICIAL FILL – UNDOCUMENTED (0-2')</u> @0 – 1.5': SAND (SW), with gravel, light orange/brown, dry, loose, scattered rootlets and metal fragments, local cobble/pebble clasts @1.5 - 2': CLAY (CL); with gravel, medium gray, brown and black (clay topsoil), dry, loose, scattered brick, asphalt and concrete (some boulder size) fragments			CL	B-1 @1.5-2	14	
Graphical Representation: Total Depth: 2 feet Scale: 1-inch=2-feet Surface Slope: 10° Trend: N35°E →						
 						

 Leighton Consulting, Inc. A LEIGHTON GROUP COMPANY	Project Name: Prologis Warehouse		Logged by: JLH		TEST PIT TP-4				
	Project Number: 12452.002		Elevation: 67 feet amsl						
	Equipment: Backhoe		Location/Grid: Northern borrow pit		Sample Results				
Earth Material Description: These earth material descriptions apply only to an exploration location 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.				Unified Soil Classification	Sample No. Depth (feet)	Moisture (%)	Laboratory Test		
Earth Materials Exposed On: August 22, 2019			Soil Unit						
<u>IMPORTED ARTIFICIAL FILL – UNDOCUMENTED (0-6.5')</u> @0'-2¼': CLAY (CL), with sand, medium orange-brown to medium gray-brown, dry, firm/soft, scattered pebbles, locally olive sand/gravel lenses @2¼-3¼': CLAY (CL), dark brown to black (clay topsoil), slightly moist, firm, local geo-fabric fragment, scattered cobble and brick fragments @3¼-4': SAND (SW), with gravel, light brown to tan, dry, loose, local concrete rubble (boulder and large cobble clasts), @4': CLAY (CL), dark brown to black (clay topsoil), moist, firm, local ¾-inch rock <u>QUATERNARY TERRACE DEPOSITS (6.5'-7')</u> @6.5-7': SAND (SP), medium orange-brown, slightly moist to moist, loose, massive friable, well sorted, fine to med-grained, indistinct thin horizontal laminae, iron-stained, non-cohesive, slightly micaceous				CL	GS-1 @2+-3+	16			
Graphical Representation: Total Depth: 7 feet				Scale: 1-inch=5-feet		Level Surface		Trend: N70°E →	
									

 Leighton Consulting, Inc. A LEIGHTON GROUP COMPANY	Project Name: Prologis Warehouse Logged by: JLH		TEST PIT TP-5			
	Project Number: 12452.002 Elevation: 67 feet amsl					
	Equipment: Backhoe Location/Grid: Southeastern borrow pit		Unified Soil Classification	Sample No. Depth (feet)	Moisture (%)	Laboratory Test
Earth Material Description: These earth material descriptions apply only to an exploration location 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.						
Earth Materials Exposed On: August 22, 2019		Soil Unit	CL-ML CL-ML	GS-1 @1-3 GS-2 @3-3+	15 18	
IMPORTED ARTIFICIAL FILL – UNDOCUMENTED (0-7') @0-0.8': CLAY (CL), sandy, medium gray, dry, soft/loose, scattered gravel @0.8': CLAY (CL), medium to dark gray and black, (clay topsoil), moist, firm/soft, local pebble-size brick and concrete fragments @3'-6': CLAYEY SAND (SC), medium orange-brown to black (clay topsoil), moist, loose/soft, local pebble-size angular fragments, @6': CLAY (CL), with sand, black, moist, soft, uniform @7': SAND (SW), with gravel, medium-brown, moist, loose						
Graphical Representation: Total Depth: 7 feet Scale: 1-inch=5-feet Level Surface Trend: N10°W →						
 						

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).

In-Situ Moisture and Density: As-sampled soil moisture content was measured (ASTM D2216) on selected samples recovered from our test pits. Results of these tests are shown on our logs at the appropriate sample depths in Appendix A.

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 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.

Atterberg Limits (ASTM D4318): Liquid Limit (LL), Plastic Limit (PL) and Plasticity Index (PI) were determined for selected soil samples in accordance with ASTM D 4318. These soil samples were air-dried and passed through a No. 40 sieve and moisturized. Liquid and plastic limit tests were performed on the soil fraction passing the No. 40 sieve. Results of these Atterberg limits tests are presented in this appendix on the “Atterberg Limits” sheets.

Expansion Index (EI, ASTM D4829): Expansion Index (EI) tests were performed in accordance with the ASTM D 4829 Standard Test Method, for shallow bulk soil samples from the site. EI results are included in this appendix on the “*Expansion Index of Soils*” sheets.

Organic Content (Los On Ignition): Organic content in soils was determined for a bulk soil sample by incineration. Results are included at the end of this appendix.

GRAVEL		SAND					FINES	
COARSE	FINE	COARSE	MEDIUM	FINE			SILT	CLAY

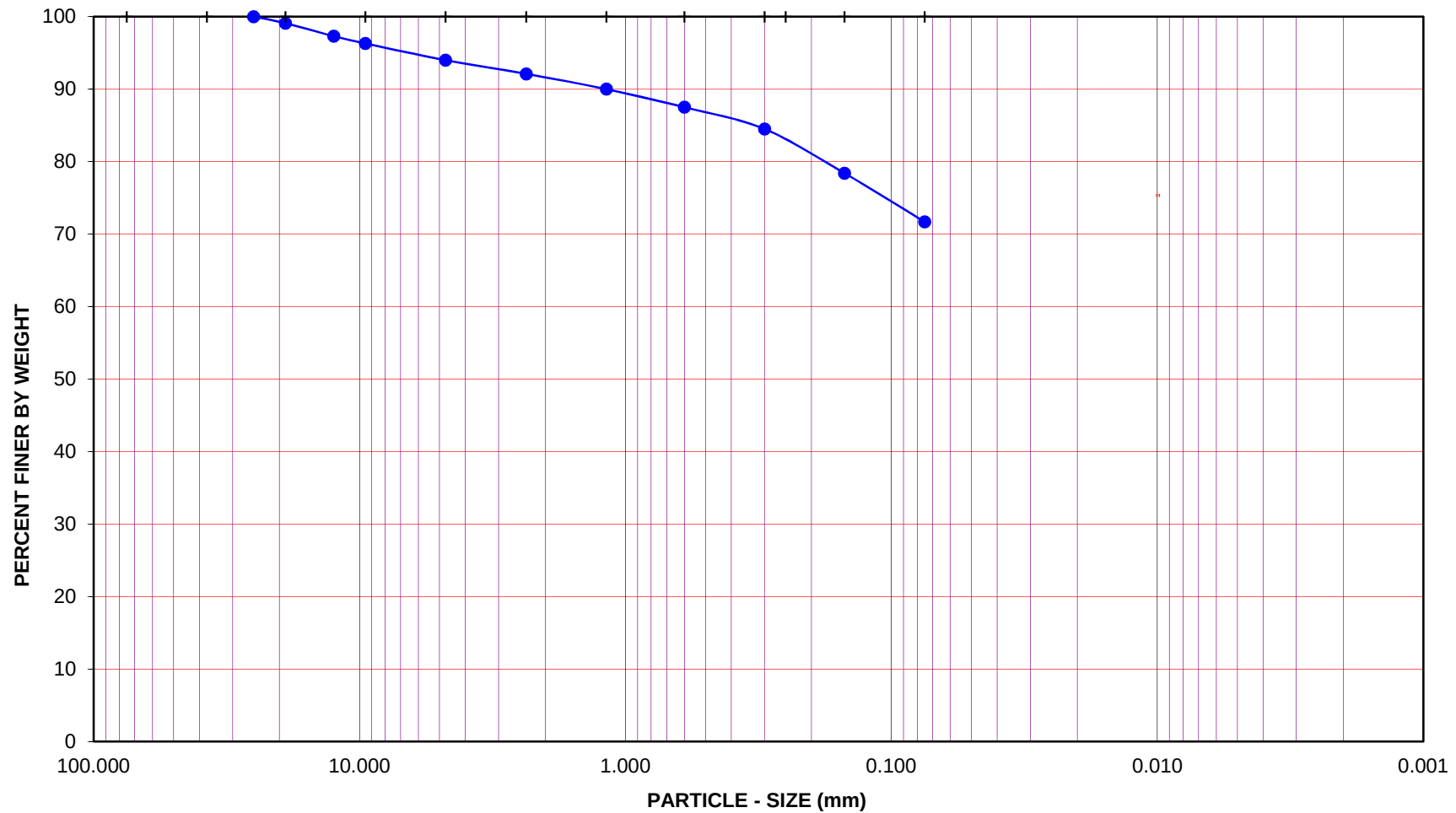
U.S. STANDARD SIEVE OPENING

3.0" 1 1/2" 3/4" 3/8"

U.S. STANDARD SIEVE NUMBER

#4 #8 #16 #30 #50 #100 #200

HYDROMETER



Project Name: Prologis Proposed Del Amo Warehousing Facility, Torrance

Project No.: 12452.002

Boring No.: TP-2

Sample No.: LB-1

Depth (feet): 4-6

Soil Type : (CL)s

Soil Identification: Olive brown lean clay with sand (CL)s

GR:SA:FI : (%) **6 : 22 : 72**



Leighton

**PARTICLE - SIZE
DISTRIBUTION
ASTM D 6913**

Sep-19

Boring No.	TP-1	TP-2						
Sample No.	LB-1	LB-2						
Depth (ft.)	5'-5' 5"	9-10						
Sample Type	Bulk	Bulk						
Soil Identification	Brown sandy lean clay s(CL)	Brown poorly-graded sand (SP)						
Moisture Correction								
Wet Weight of Soil + Container (g)	0.0	0.0						
Dry Weight of Soil + Container (g)	0.0	0.0						
Weight of Container (g)	1.0	1.0						
Moisture Content (%)	0.0	0.0						
Sample Dry Weight Determination								
Weight of Sample + Container (g)	749.4	626.4						
Weight of Container (g)	109.3	108.4						
Weight of Dry Sample (g)	640.1	518.0						
Container No.:								
After Wash								
Method (A or B)	A	A						
Dry Weight of Sample + Cont. (g)	331.7	618.7						
Weight of Container (g)	109.3	108.4						
Dry Weight of Sample (g)	222.4	510.3						
% Passing No. 200 Sieve	65.3	1.5						
% Retained No. 200 Sieve	34.7	98.5						
 Leighton PERCENT PASSING No. 200 SIEVE ASTM D 1140 Project Name: <u>Prologis Proposed Del Amo Warehousing Facility, Torrance</u> Project No.: <u>12452.002</u> Tested By: <u>S. Felter</u> Date: <u>08/27/19</u>								



Leighton

ATTERBERG LIMITS

ASTM D 4318

Project Name: Prologis Proposed Del Amo Warehousing Facility, Torrance Tested By: S. Felter Date: 09/05/19
 Project No. : 12452.002 Input By: J. Ward Date: 09/09/19
 Boring No.: TP-1 Checked By: J. Ward
 Sample No.: LB-1 Depth (ft.) 5'-5' 5"
 Soil Identification: Brown sandy lean clay s(CL)

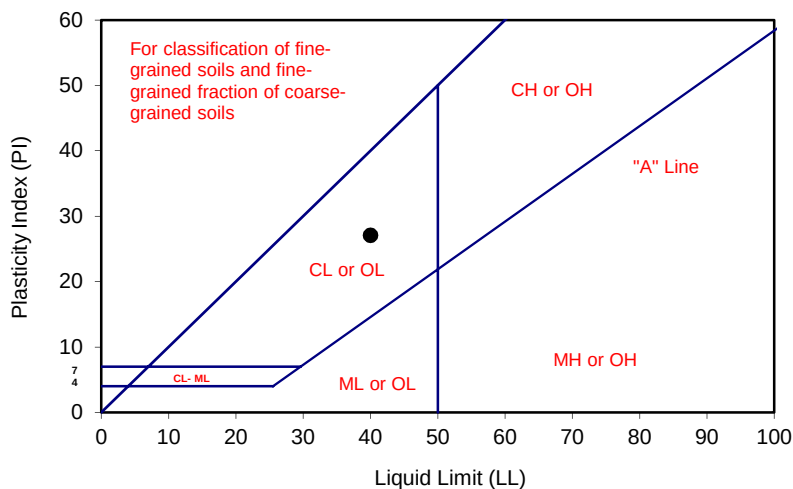
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			34	26	19	
Wet Wt. of Soil + Cont. (g)	10.25	10.32	20.92	20.76	22.37	
Dry Wt. of Soil + Cont. (g)	9.20	9.26	15.45	15.17	16.18	
Wt. of Container (g)	1.06	1.06	1.08	1.04	1.07	
Moisture Content (%) [Wn]	12.90	12.93	38.07	39.56	40.97	

Liquid Limit	40
Plastic Limit	13
Plasticity Index	27
Classification	CL

PI at "A" - Line = $0.73(LL-20)$ 14.6

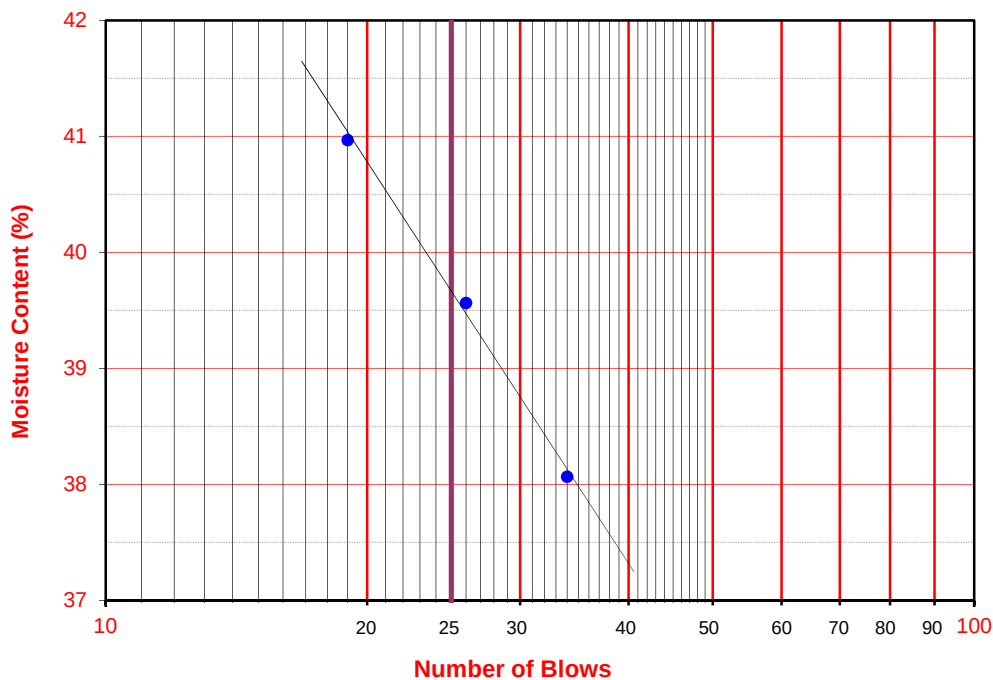
One - Point Liquid Limit Calculation

$$LL = Wn(N/25)^{0.121}$$



PROCEDURES USED

- ☐ Wet Preparation
Multipoint - Wet
- ☒ Dry Preparation
Multipoint - Dry
- ☒ Procedure A
Multipoint Test
- ☐ Procedure B
One-point Test





ATTERBERG LIMITS

ASTM D 4318

Project Name: Prologis Proposed Del Amo Warehousing Facility, Torrance Tested By: S. Felter Date: 09/04/19
 Project No. : 12452.002 Input By: J. Ward Date: 09/09/19
 Boring No.: TP-1 Checked By: J. Ward
 Sample No.: LB-2 Depth (ft.) 10-11
 Soil Identification: Dark brown fat clay (CH)

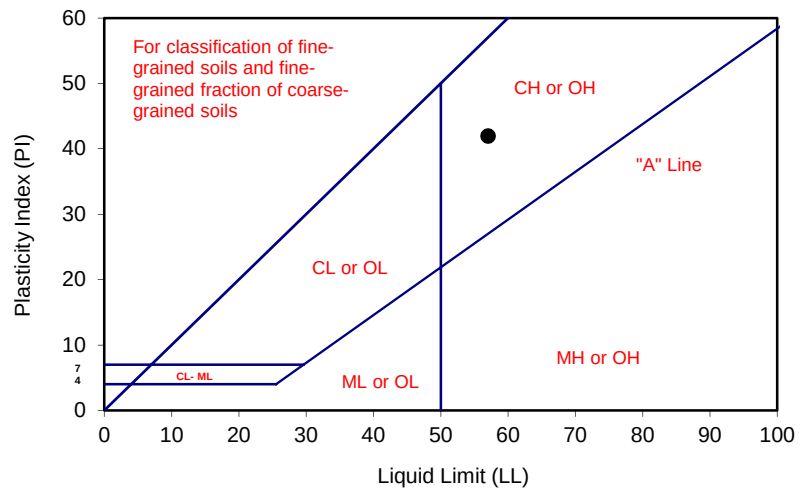
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			34	27	20	
Wet Wt. of Soil + Cont. (g)	10.26	10.17	20.22	20.38	20.21	
Dry Wt. of Soil + Cont. (g)	9.05	8.99	13.35	13.37	13.16	
Wt. of Container (g)	1.11	1.03	1.05	1.13	1.08	
Moisture Content (%) [Wn]	15.24	14.82	55.85	57.27	58.36	

Liquid Limit	57
Plastic Limit	15
Plasticity Index	42
Classification	CH

PI at "A" - Line = $0.73(LL-20)$ 27.01

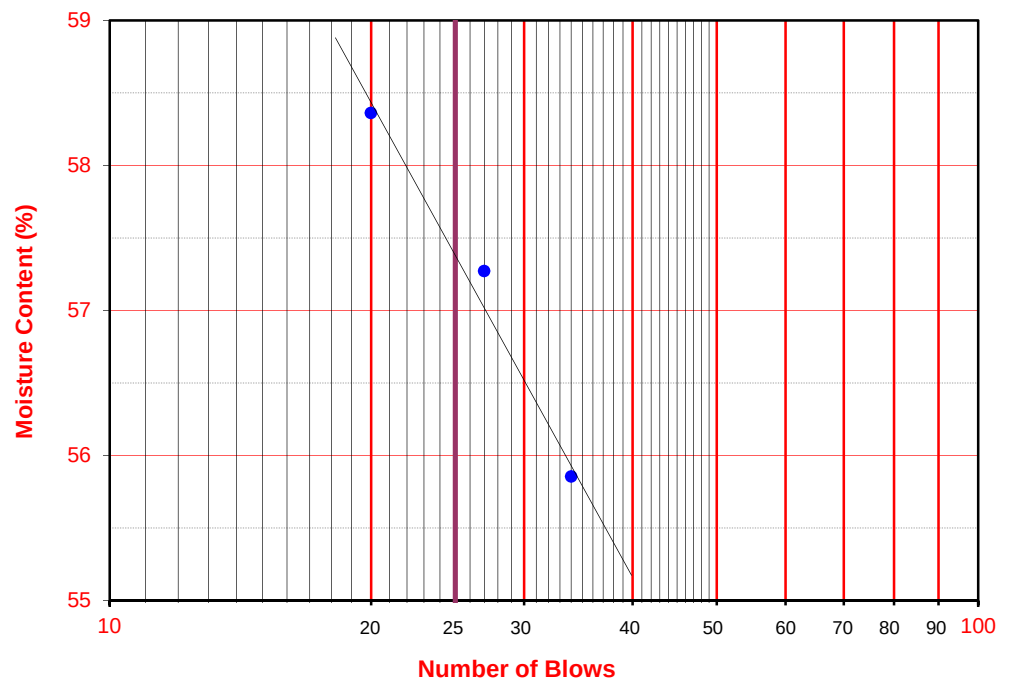
One - Point Liquid Limit Calculation

$$LL = W_n(N/25)^{0.121}$$



PROCEDURES USED

- ☐ Wet Preparation
Multipoint - Wet
- ☒ Dry Preparation
Multipoint - Dry
- ☒ Procedure A
Multipoint Test
- ☐ Procedure B
One-point Test





ATTERBERG LIMITS

ASTM D 4318

Project Name: Prologis Proposed Del Amo Warehousing Facility, Torrance Tested By: S. Felter Date: 09/05/19
 Project No. : 12452.002 Input By: J. Ward Date: 09/09/19
 Boring No.: TP-5 Checked By: J. Ward
 Sample No.: GS-3 Depth (ft.) 7.5
 Soil Identification: Dark gray fat clay (CH)

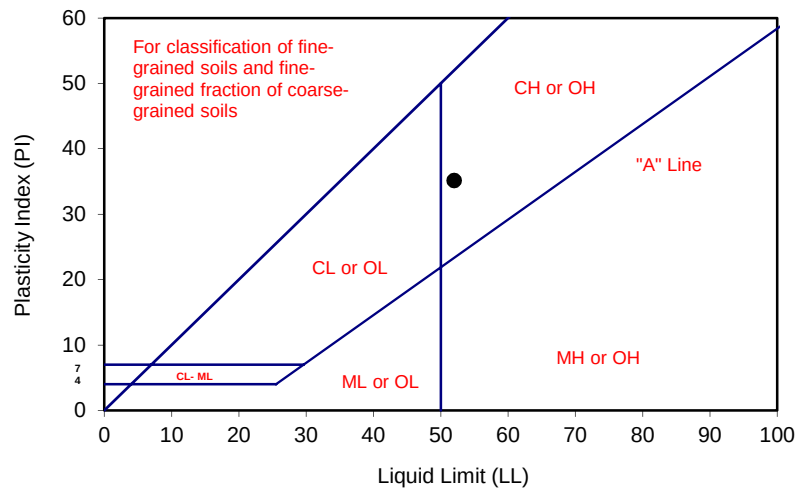
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			35	28	21	
Wet Wt. of Soil + Cont. (g)	10.06	10.10	22.65	21.12	21.64	
Dry Wt. of Soil + Cont. (g)	8.75	8.81	15.43	14.27	14.58	
Wt. of Container (g)	1.04	1.09	1.06	1.08	1.15	
Moisture Content (%) [Wn]	16.99	16.71	50.24	51.93	52.57	

Liquid Limit	52
Plastic Limit	17
Plasticity Index	35
Classification	CH

PI at "A" - Line = $0.73(LL-20)$ 23.36

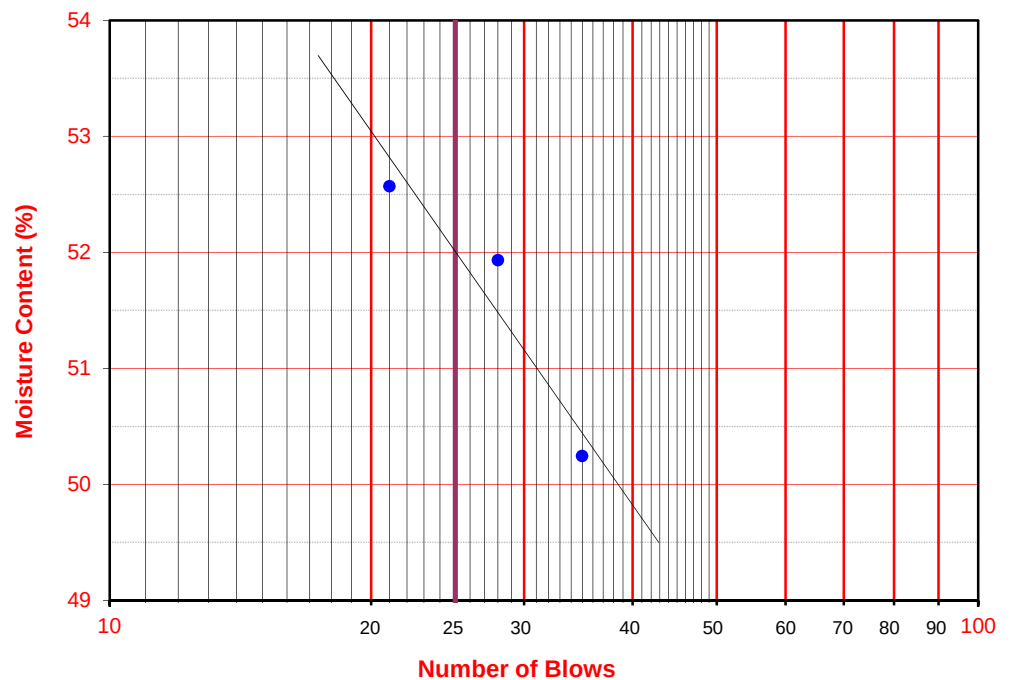
One - Point Liquid Limit Calculation

$$LL = W_n(N/25)^{0.121}$$



PROCEDURES USED

- ☐ Wet Preparation
Multipoint - Wet
- ☒ Dry Preparation
Multipoint - Dry
- ☒ Procedure A
Multipoint Test
- ☐ Procedure B
One-point Test





MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name: Prologis Proposed Del Amo Warehousing Facility, Torrance Tested By: G. Berdy Date: 09/05/19
 Project No.: 12452.002 Input By: J. Ward Date: 09/09/19
 Boring No.: TP-2 Depth (ft.): 4-6
 Sample No.: LB-1
 Soil Identification: Olive brown lean clay with sand (CL)s

Note: Corrected dry density calculation assumes specific gravity of 2.70 and moisture content of 1.0% for oversize particles

Preparation Method:	☒	Moist	Scalp Fraction (%)	Rammer Weight (lb.) =	10.0
		Dry	#3/4	Height of Drop (in.) =	18.0
Compaction Method:	☒	Mechanical Ram	#3/8		
		Manual Ram	#4	Mold Volume (ft ³)	0.03320

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3569	3648	3703	3712		
Weight of Mold (g)	1817	1817	1817	1817		
Net Weight of Soil (g)	1752	1831	1886	1895		
Wet Weight of Soil + Cont. (g)	430.7	446.8	412.7	460.5		
Dry Weight of Soil + Cont. (g)	404.4	411.3	371.7	406.6		
Weight of Container (g)	39.4	39.4	38.0	38.8		
Moisture Content (%)	7.21	9.55	12.29	14.65		
Wet Density (pcf)	116.3	121.6	125.2	125.8		
Dry Density (pcf)	108.5	111.0	111.5	109.8		

Maximum Dry Density (pcf) **111.7**

Optimum Moisture Content (%) **11.5**

Corrected Dry Density (pcf) **114.0**

Corrected Moisture Content (%) **10.9**

☒ **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 + 3/4 in. is <30%

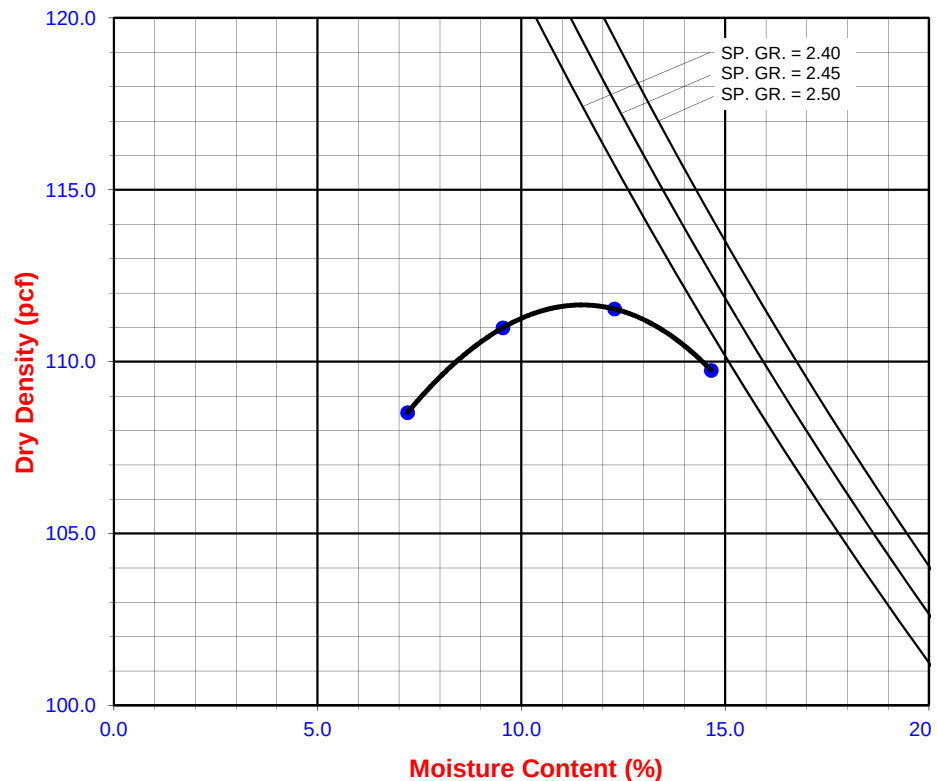
Particle-Size Distribution:

6:22:72

GR:SA:FI

Atterberg Limits:

LL,PL,PI





EXPANSION INDEX of SOILS

ASTM D 4829

Project Name: Prologis Proposed Del Amo Warehousing Facility, Torrance Tested By: SF/OHF Date: 08/27/19
 Project No.: 12452.002 Checked By: J. Ward Date: 09/09/19
 Boring No.: TP-1 Depth (ft.): 10-11
 Sample No.: LB-2
 Soil Identification: Dark brown fat clay (CH)

Dry Wt. of Soil + Cont.	(g)	1000.00
Wt. of Container No.	(g)	0.00
Dry Wt. of Soil	(g)	1000.00
Weight Soil Retained on #4 Sieve		0.00
Percent Passing # 4		100.00

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0745
Wt. Comp. Soil + Mold (g)	575.40	426.08
Wt. of Mold (g)	191.70	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	0	0
Wet Wt. of Soil + Cont. (g)	759.30	617.78
Dry Wt. of Soil + Cont. (g)	678.00	534.29
Wt. of Container (g)	0.00	191.70
Moisture Content (%)	11.99	24.37
Wet Density (pcf)	115.7	119.6
Dry Density (pcf)	103.3	96.2
Void Ratio	0.631	0.753
Total Porosity	0.387	0.430
Pore Volume (cc)	80.1	95.5
Degree of Saturation (%) [S _{meas}]	51.3	87.4

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
08/27/19	10:20	1.0	0	0.4925
08/27/19	10:30	1.0	10	0.4930
Add Distilled Water to the Specimen				
08/27/19	14:00	1.0	210	0.5530
08/28/19	6:13	1.0	1183	0.5670
08/28/19	7:30	1.0	1260	0.5670

Expansion Index (EI _{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	74
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**MOISTURE, ORGANIC MATTER and
ASH CONTENT of SOILS
ASTM D 2974 (Test Methods A & C)**

Project Name: Prologis Proposed Del Amo Warehousing Facility, Torrance Tested By: G. Bathala Date: 09/04/19
Project No. : 12452.002 Input By: J. Ward Date: 09/09/19

Boring No.	TP-2				
Sample No.	LB-1				
Depth (ft)	4-6				
Soil Description	Olive brown lean clay with sand (CL)s				
Wt. of Moist Soil + Container (gm)	231.20				
Wt. of Dried Soil + Container (gm)	221.55				
Wt. Container (gm)	39.50				
Crucible No.	12, 6				
Furnace Temperature (°C)	440				
Time In / Time Out	8:35/14:05				
Duration of Combustion (hr)	5:30				
Wt. of Dried Soil + Crucible (gm)	105.82				
Wt. of Ash + Crucible (gm)	103.81				
Wt. of Crucible (gm)	41.45				
Moisture Content @ 105 °C (%) "as received"	5.3				
Dry wt. of Soil (gm) (1)	64.37				
Wt. of Ash (gm) (2)	62.36				
Ash Content (%) = [(2) / (1)] x 100 (3)	96.9				
Organic Matter (%) = 100 - (3)	3.1				

Remarks: Moisture, ash & organic contents are calculated as percentages of oven-dried mass of test specimen.

APPENDIX C

EARTHWORK AND GRADING GUIDE SPECIFICATIONS

APPENDIX C

LEIGHTON CONSULTING, INC. EARTHWORK AND GRADING GUIDE SPECIFICATIONS

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C - 1 . 0 G E N E R A L

C-1.1 Intent

These Earthwork and Grading Guide Specifications are for grading and earthwork shown on the current, approved grading plan(s) and/or indicated in the Leighton Consulting, Inc. geotechnical report(s). These Guide Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the project-specific recommendations in the geotechnical report shall supersede these Guide Specifications. Leighton Consulting, Inc. shall provide geotechnical observation and testing during earthwork and grading. Based on these observations and tests, Leighton Consulting, Inc. may provide new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

C-1.2 Role of Leighton Consulting, Inc.

Prior to commencement of earthwork and grading, Leighton Consulting, Inc. shall meet with the earthwork contractor to review the earthwork contractor's work plan, to schedule sufficient personnel to perform the appropriate level of observation, mapping and compaction testing. During earthwork and grading, Leighton Consulting, Inc. shall observe, map, and document subsurface exposures to verify geotechnical design assumptions. If observed conditions are found to be significantly different than the interpreted assumptions during the design phase, Leighton Consulting, Inc. shall inform the owner, recommend appropriate changes in design to accommodate these observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include (1) natural ground after clearing to receiving fill but before fill is placed, (2) bottoms of all "remedial removal" areas, (3) all key bottoms, and (4) benches made on sloping ground to receive fill.

Leighton Consulting, Inc. shall observe moisture-conditioning and processing of the subgrade and fill materials, and perform relative compaction testing of fill to determine the attained relative compaction. Leighton Consulting, Inc. shall provide *Daily Field Reports* to the owner and the Contractor on a routine and frequent basis.

C-1.3 The Earthwork Contractor

The earthwork contractor (Contractor) shall be qualified, experienced and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Guide

Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing grading and backfilling in accordance with the current, approved plans and specifications.

The Contractor shall inform the owner and Leighton Consulting, Inc. of changes in work schedules at least one working day in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that Leighton Consulting, Inc. is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish earthwork and grading in accordance with the applicable grading codes and agency ordinances, these Guide Specifications, and recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of Leighton Consulting, Inc., unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, adverse weather, etc., are resulting in a quality of work less than required in these specifications, Leighton Consulting, Inc. shall reject the work and may recommend to the owner that earthwork and grading be stopped until unsatisfactory condition(s) are rectified.

C - 2 . 0 P R E P A R A T I O N O F A R E A S T O B E F I L L E D

C-2.1 Clearing and Grubbing

Vegetation, such as brush, grass, roots and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies and Leighton Consulting, Inc.. Care should be taken not to encroach upon or otherwise damage native and/or historic trees designated by the Owner or appropriate agencies to remain. Pavements, flatwork or other construction should not extend under the “drip line” of designated trees to remain.

Leighton Consulting, Inc. shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 3 percent of organic materials (by dry weight: ASTM D2974). Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area. As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that

are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

C-2.2 Processing

Existing ground that has been declared satisfactory for support of fill, by Leighton Consulting, Inc., shall be scarified to a minimum depth of 6 inches (15 cm). Existing ground that is not satisfactory shall be over-excavated as specified in the following Section C-2.3. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

C-2.3 Overexcavation

In addition to removals and over-excavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be over-excavated to competent ground as evaluated by Leighton Consulting, Inc. during grading. All undocumented fill soils under proposed structure footprints should be excavated

C-2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), (>20 percent grade) the ground shall be stepped or benched. The lowest bench or key shall be a minimum of 15 feet (4.5 m) wide and at least 2 feet (0.6 m) deep, into competent material as evaluated by Leighton Consulting, Inc.. Other benches shall be excavated a minimum height of 4 feet (1.2 m) into competent material or as otherwise recommended by Leighton Consulting, Inc.. Fill placed on ground sloping flatter than 5:1 (horizontal to vertical units), (<20 percent grade) shall also be benched or otherwise over-excavated to provide a flat subgrade for the fill.

C-2.5 Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by Leighton Consulting, Inc. as suitable to receive fill. The Contractor shall obtain a written acceptance (*Daily Field Report*) from Leighton Consulting, Inc. prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys and benches.

C - 3 . 0 F I L L M A T E R I A L

C-3.1 Fill Quality

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by Leighton Consulting, Inc. prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to Leighton Consulting, Inc. or mixed with other soils to achieve satisfactory fill material.

C-3.2 Oversize

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 6 inches (15 cm), shall not be buried or placed in fill unless location, materials and placement methods are specifically accepted by Leighton Consulting, Inc.. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 feet (3 m) measured vertically from finish grade, or within 2 feet (0.61 m) of future utilities or underground construction.

C-3.3 Import

If importing of fill material is required for grading, proposed import material shall meet the requirements of Section C-3.1, and be free of hazardous materials (“contaminants”) and rock larger than 3-inches (8 cm) in largest dimension. All import soils shall have an Expansion Index (EI) of 20 or less and a sulfate content no greater than ($\leq$) 500 parts-per-million (ppm). A representative sample of a potential import source shall be given to Leighton Consulting, Inc. at least four full working days before importing begins, so that suitability of this import material can be determined and appropriate tests performed.

C - 4 . 0 F I L L P L A C E M E N T A N D C O M P A C T I O N

C-4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill, as described in Section C-2.0, above, in near-horizontal layers not exceeding 8 inches (20 cm) in loose thickness. Leighton Consulting, Inc. may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers, and only if the building officials with the appropriate jurisdiction approve. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

C-4.2 Fill Moisture Conditioning

Fill soils shall be watered, dried back, blended and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM) Test Method D1557.

C-4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, each layer shall be uniformly compacted to not-less-than ($\geq$) 90 percent of the maximum dry density as determined by ASTM Test Method D1557. In some cases, structural fill may be specified (see project-specific geotechnical report) to be uniformly compacted to at least ($\geq$) 95 percent of the ASTM D1557 modified Proctor laboratory maximum dry density. For fills thicker than ($>$) 15 feet (4.5 m), the portion of fill deeper than 15 feet below proposed finish grade shall be compacted to 95 percent of the ASTM D1557 laboratory maximum density. Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

C-4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by back rolling of slopes with sheep's-foot rollers at increments of 3 to 4 feet (1 to 1.2 m) in fill elevation, or by other methods producing satisfactory results acceptable to Leighton Consulting, Inc.. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of the ASTM D1557 laboratory maximum density.

C-4.5 Compaction Testing

Field-tests for moisture content and relative compaction of the fill soils shall be performed by Leighton Consulting, Inc.. Location and frequency of tests shall be at our field representative(s) discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

C-4.6 Compaction Test Locations

Leighton Consulting, Inc. shall document the approximate elevation and horizontal coordinates of each density test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that Leighton

Consulting, Inc. can determine the test locations with sufficient accuracy. Adequate grade stakes shall be provided.

C - 5 . 0 E X C A V A T I O N

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by Leighton Consulting, Inc. during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by Leighton Consulting, Inc. based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, then observed and reviewed by Leighton Consulting, Inc. prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by Leighton Consulting, Inc..

C - 6 . 0 T R E N C H B A C K F I L L S

C-6.1 **Safety**

The Contractor shall follow all OSHA and Cal/OSHA requirements for excavation safety. Work should be performed in accordance with Article 6 of the *California Construction Safety Orders*, 2015 Edition or more current (see also: <http://www.dir.ca.gov/title8/sb4a6.html>).

C-6.2 **Bedding and Backfill**

All utility trench bedding and backfill shall be performed in accordance with applicable provisions of the 2018 Edition of the *Standard Specifications for Public Works Construction* (Green Book). Bedding material shall have a Sand Equivalent (SE) greater than 30 (SE>30). Bedding shall be placed to 1-foot (0.3 m) over the top of the conduit, and densified by jetting in areas of granular soils, if allowed by the permitting agency. Otherwise, the pipe-bedding zone shall 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 2018 Edition of the *Standard Specifications for Public Works Construction* (Green Book). Backfill over the bedding zone shall be placed and densified mechanically to a minimum of 90 percent of relative compaction (ASTM D1557) from 1 foot (0.3 m) above the top of the conduit to the surface. Backfill above the pipe zone shall **not** be jetted. Jetting of the bedding around the conduits shall be observed by Leighton Consulting, Inc. and backfill above the pipe zone (bedding) shall be observed and tested by Leighton Consulting, Inc..

C-6.3 Lift Thickness

Trench backfill, and engineered fill of any kind, lift thickness shall not exceed those allowed in the 2018 Edition of the *Standard Specifications for Public Works Construction* (Green Book); unless the Contractor can demonstrate to Leighton Consulting, Inc., the Owner **and** the appropriate-jurisdiction building official that fill lifts can be consistently compacted to specified relative compaction using alternative equipment and methods intended for production earthwork.

DEPARTMENT OF TRANSPORTATION
ENGINEERING SERVICE CENTER
Transportation Laboratory
P.O. Box 19128
Sacramento, California 95819



METHOD FOR INSTALLATION AND USE OF EMBANKMENT SETTLEMENT DEVICES

CAUTION: Prior to handling test materials, performing equipment setups, and/or conducting this method, testers are required to read **"SAFETY AND HEALTH"** in Part 4 of this method. It is the responsibility of the user of this method to consult and use departmental safety and health practices and determine the applicability of regulatory limitations before any testing is performed.

OVERVIEW

The installation, maintenance, and data collection procedures for the various embankment settlement devices used to monitor subsurface settlement are described in this method. Analysis of the settlement data is included as a separate part of this method.

Settlement devices are used to monitor the rate and magnitude of settlement occurring at a point within or beneath an embankment during and subsequent to construction. The data obtained from these devices are used to determine the allowable loading rate during embankment construction and the appropriate time for removal of surcharge and/or commencement of permanent structure construction.

This method is divided into the following parts:

1. Fluid Level Settlement Devices
2. Pipe Riser Settlement Device
3. Settlement Data Analysis
4. Safety and Health

The fluid level, vented standpipe unit may be used at most locations. A sealed standpipe unit must be installed at locations where

groundwater may interfere with the operation of the unit or where excess pore water is expected from the use of dredged material or wet soil in embankment construction. Where it is possible, the tube length between standpipe and indicator unit should generally be limited to a maximum of 100 m. Installations over longer distances can be made but are not advisable under normal circumstances, since it may result in inconsistent test data. Factors such as larger size tubes, change of platform location, or changes in elevation of the water line may have to be considered (see NOTE).

NOTE: There may be job conditions with respect to terrain, long tube length between standpipe and indicator unit, or anticipated large settlements that require special installations.

The pipe riser settlement device is used for monitoring fill settlement over soft foundation soils where the fluid level settlement devices are not feasible because of flat terrain, width of embankment construction, or other features which would make installation of a fluid level type of settlement platform undesirable. The pipe riser settlement device is a direct-reading unit which is exposed for the full duration of fill construction and surcharge removal. Because of the vulnerability of this unit to damage by the contractor's operations, the

pipe riser settlement device should be used only on those projects where the fluid level type of settlement device would be impractical.

PART 1. FLUID LEVEL SETTLEMENT DEVICES

SECTION 1 - VENTED STANDPIPE UNIT

A. APPARATUS

1. Vented standpipe unit (Figure 1)
2. Indicator unit (Figure 2)
3. Polyethylene tubing, 10-mm inside diameter
4. Hand tools - shovel, bar, posthold auger, hammer, adjustable wrenches, etc.
5. Water container (approximately 4-L capacity)

B. INSTALLATION

1. Select a location for the standpipe unit on the ground after approximately 0.3 m of fill has been placed above original ground and generally within the area where the maximum height of embankment will be placed. See Figure 4.
2. Select a point outside of the toe of the proposed embankment for the indicator unit. See Figure 4. Select this location so that sufficient vertical distance will be available for lowering the indicator unit as the standpipe unit settles. A hand level may be used to estimate the desired elevations for the indicator unit.
3. Because of terrain, excessive anticipated settlement, or other causes, it may be necessary to place the standpipe unit in the embankment at varying elevations above the original ground. In these cases, record the vertical distance between the base of the standpipe unit and original ground to allow proper consideration for embankment compression in the settlement analysis.

4. After embankment has been placed 1.0 to 1.5 m above the desired elevation for the standpipe unit, prepare a pit and trench in the embankment for the standpipe unit and tubing. See Figure 4. The bottom of the pit should normally be about 0.3 m above original ground. The trench should be cut to the same depth at the pit and should have a slight downward slope to the indicator unit location. Make sure that the trench is clear of any future construction, such as pile driving, ripping, ditching, etc.
5. Upon completion of the excavation, remove all rocks and large clods from the trench. Prepare a smooth, level area in the pit using fine embankment material.
6. Assemble the standpipe unit as shown in Figure 1. *Do not attach the pipe cap.* Firmly seat the standpipe unit on a prepared level area.
7. Install the indicator unit post at the previously selected point for the indicator unit. This post can be either a metal sign post or 4-by-4.
8. Using a hand level, attach the indicator unit to the post so that the 0.7-m graduation on the indicator unit scale is approximately level with the top of the spill tube on the standpipe unit.
9. Push the 10-mm water line through the metal tube conduit in the center of the vented standpipe unit until the end is approximately 5 mm above the top. Push the 10-mm air vent line through the other conduit until approximately 20 mm extends out the top. See Figure 1.
10. Unroll the water and air vent lines loosely in the trench from the standpipe to the indicator unit. It might be desirable to encase both lines in 19-mm flexible metal conduit for additional protection under rocky material.
11. Cut and attach the water and air lines to the indicator unit as shown in Figures 2 and 3. Then fill the system by pouring

water in the sight tube of the indicator unit (Figure 2) until water comes out of the top of the spill tube of the standpipe unit with no air bubbles showing in the line. Generally, 4 L of water is more than adequate for 100 m of tubing. When filling, attempt to keep the water level in the sight tube near the 0.7-m graduation. Do not allow the water level to drop below the bottom of the sight tube since this would allow air to enter the system.

NOTE: It is helpful if someone can watch the overflow at the standpipe unit while the system is being filled to look for evidence of entrapped air and to signal when the system is full.

If there is evidence of air bubbles entrapped in the water line, continue charging the system with water until the air is purged through the standpipe unit. After charging the system with water and purging the water line of all air, attach the indicator unit on the post to provide an initial reading of approximately 0.7 m.

Adjacent to the bottom of the indicator unit, place a reference nail in the post at the elevation of the 0.0-m graduation. This provides a reference point for surveys and relocation of the indicator unit. Complete the assembly of the standpipe unit by attaching the pipe cap as shown in Figure 1.

12. Cut the air line at approximately the 0.7-m graduation of the indicator unit. Then loop the air line inside of the indicator unit over the lock hasp. See Figure 2. The end of the air line should be pointing downward to prevent the entrance of water or debris. This air line must be free of water at all times since it serves to equalize atmospheric pressure at the standpipe unit and the indicator unit.

During cold weather when the air line is too stiff to be looped, cut the air line at the 0.3-m graduation mark. Then insert

the end of a 0.3-m length of 6-mm plastic tubing in the end of the air line and loop the smaller tubing inside the indicator unit.

13. Carefully backfill the trench and pit with material that is free from large rocks or sharp objects and compact by hand for a depth of at least 0.3 m. Special care must be taken around the base of the standpipe unit to prevent separating the base plate from the plywood platform and to prevent breaking or distorting the plastic tubing.
14. After hand backfilling and compacting for a depth of 0.3 m has been completed, mechanical methods may be used to finish the backfilling operation until the trench is level with the existing fill height. In those cases when the standpipe unit extends above the existing fill height, attach a marker post to the unit and mound fill material around it until it is completely covered. In no case should compaction equipment be allowed directly over an installation until a minimum of 0.3 m of compacted material has been placed over the standpipe unit.

SECTION 2 - SEALED STANDPIPE UNIT

A. APPARATUS

1. Sealed standpipe unit (Figure 5)
2. Plastic drain tubing, 13-mm inside diameter
3. Vented standpipe unit (as described in Part 1, Section 1-A)

B. INSTALLATION

Installation is similar to that for the vented standpipe unit with the following exceptions:

1. Install the device as shown in Figure 6.
2. Follow the procedure in Part 1, Sections B-1 through B-5.

3. Assemble the standpipe unit as shown in Figure 5. Do not attach the outer galvanized pipe. Firmly seat the standpipe unit on the prepared area in the pit.
4. Follow the procedure in Part 1, Sections B-7 and B-8.
5. Attach the 10-mm water tube to the baseplate as shown in Figure 5.
6. Unroll the water, air vent, and drain tubes loosely in the trench from the standpipe to the indicator unit.
7. Follow the procedure in Part 1, Sections B-10 and B-11.
8. Complete the assembly of the sealed standpipe unit by attaching the outer galvanized pipe, other fittings, air vent, and drain tubes.

NOTE: It may be desirable in some cases to fill around the sealed units with sand so that the tubes will be supported at their attachment points.

9. Cut the drain tube near the base of the indicator unit post. Position the drain tube so that water flows out freely and intrusion of soil or debris is prevented.
10. Follow the procedure in Part 1, Sections B-13 and B-14.

SECTION 3 - POST-INSTALLATION PROCEDURES

A. COLLECTION OF DATA

1. As soon as possible after the settlement device has been installed, determine the elevation of the reference to ± 2 mm by survey. This elevation should be checked periodically to correct settlement readings for settlement of indicator unit.
2. Settlement Readings
 - a. Note the height of water in the sight tube.

- b. Pour sufficient water in the sight tube to raise the water level approximately 50 mm.
 - c. Take a reading at 1 h. The water level after adding the water should drop to the first reading or slightly above it.
 - d. If little or no change is observed in the water level, or if the water level is below the 0.01-m graduation, refer to Part 2, Section 3.
3. Record the data as indicated in Figure 7. The form is normally used to record chronological data from a single settlement unit installed to monitor settlement. Instructions in filling out the form follow:

a. Settlement Data Report (Figure 7)

Column 1 - Enter the date of the reading.

Column 2 - Record the water level reading from the graduated scale on the indicator unit (after adding water, as indicated above).

Column 3 - Record the latest elevation of either the 0.0-m graduation on the indicator unit, or the nail reference as determined by the survey.

Columns 4 and 5 - These columns are not used.

Column 6 - Enter the changes in water level elevation and reference elevation since the last reading. This is the sum of the differences between the current and immediately previous entries in Columns 2 and 3.

Column 7 - Enter the total settlement as *minus* the elevation change since installation. This is obtained by changing the sign of the current data in Column 6 and adding this value to the previous entry in Column 7.

Column 8 - Enter the height of the fill at the surface as determined by survey (optional).

Column 9 - Enter the height of fill above original ground to the nearest 0.1 m.

Column 10 - Enter the number of calendar days elapsed since the settlement device was installed.

Column 11 - Enter any information that would be helpful in the analysis of data as shown. If it is necessary to lower the indicator unit on the post, enter the date and the vertical distance lowered; be sure to include the corrected values in Columns 2, 3, and 6.

B. MAINTENANCE

1. Most important to the continued functioning of fluid level settlement devices is the use of as little water as necessary when recharging the system before reading. For this reason, use only enough water to raise the level in the sight tube approximately 50 mm. Continuous additions of greater quantities of water will probably cause flooding of the standpipe unit.
2. If the water level in the sight tube does not drop after adding water, check the unit over a period of several days. Do not, however, add an excessive amount of water; just observe the system to see if the unit is slow to respond.
 - a. If the unit is not operating properly, remove the indicator box from the post and raise it up about 0.3 m. Disconnect the water line from the sight tube and attach the line upright on the post. Inspect the bottom of the sight tube and connector for debris. Remove any obstructions and reassemble the unit without losing water from the water line. After assembly, lower the indicator unit until water is observed in the sight tube, then recharge the system with clean water as necessary.
 - b. If the device is still not operating satisfactorily and the sealed standpipe unit is being used, plug the top of the sight tube and attempt to force compressed air through the air line and out the drain line. Do not use greater air pressure than necessary to obtain a small flow through the lines. Do not allow the water in the sight tube to overflow. Keep the top of the sight tube sealed during this operation.
 - c. If all other attempts to correct the malfunction fail, disconnect and drain the water line. Then apply compressed air at low pressure to the air line in an attempt to remove debris from the water line. If the sealed standpipe unit is used, plug the drain line during this operation. Occasionally, force air through the water line to clear the lines if no return is observed when pressurizing the air line. If successful in clearing obstructions from the water line in this manner, considerable care is required while recharging the system with water to not use too much water and to not introduce large voids in the system. For this reason, recharging the unit should be performed only by personnel experienced in this type of activity.
3. If the water level in the sight tube is below the 0.01-m graduation or if there is no water in the sight tube, look for leaks around the connection between the sight tube and the water line. If no leak is seen, measure the vertical difference between the 0.0-mm graduation on the indicator unit and the reference point. Remove the unit from the post and lower it approximately 0.5 m or until water is observed in the sight tube. If possible, and without adding water, adjust the height of the indicator unit on the post so that the water level in the sight tube is approximately at the 0.7-m graduation.

Add a small quantity of water and check the water level before attaching the indicator unit to the post. After adjusting the height of the indicator unit, again measure the vertical distance between the 0.0-m graduation on the indicator unit and the reference point, and record the correction on the settlement data form (Figure 7, Column 3).

4. Be sure to replace the cover on the indicator unit after each reading to prevent excessive loss by evaporation and contamination by debris.
5. Occasionally, it may be necessary to protect the air and water lines from rodents or pests. If such a problem exists, protect these lines in flexible conduit extending from the bottom of the indicator unit to below the ground surface. Although this should be done during installation, the conduit can be added later if extreme care is taken not to lose water continuity as described above.

PART 2. PIPE RISER SETTLEMENT DEVICE

A. APPARATUS

1. Pipe riser settlement device (Figure 8)
2. Hand tools - shovel, bar, hammer, pipe wrenches, etc.

B. INSTALLATION

1. It will usually be necessary to determine the location for installing the settlement device by survey. If settlement readings are to be continued after completion of the fill and removal of surcharge, it is imperative that the unit be located directly beneath the median of divided travel lanes or the shoulder of other roadways.
2. After approximately 1 m of embankment material has been placed, excavate a pit to a depth of approximately 0.5 m above original ground at the previously determined location for the settlement

device. Prepare a firm, level area free of large rocks or clods for the settlement device at the bottom of the pit.

3. Assemble the settlement device as shown in Figure 8. Attach a 19-mm pipe floor flange to the center of the wood platform with bolts or lag screws. Then screw a 1.8 m length of 19-mm pipe into the floor flange. Place a pipe coupling on the top of the 19-mm pipe and tighten all joints in the assembly using a pipe wrench.
4. Measure and record the distance from the top of the pipe coupling to the top of the wood platform. Then slip the 38 mm by 1.5-m protective sleeve, which may be either rigid polyvinyl chloride (PVC) or iron pipe, over the control pipe until it is about 0.5 m above the floor flange. Place a duct seal or other seal to hold the protective sleeve in place. See Figure 8. Do not attach the protective sleeve to the wood platform or the control pipe. This protective sleeve is used to absorb the friction between the fill material and the settlement unit and, therefore, must be free to move independently from the wood platform and control pipe.
5. Firmly seat the settlement device on the prepared area in the bottom of the pit. Then fill and compact by hand using fine embankment material free of large rocks and clods around the settlement device to a depth of 0.3 m.
6. Using a spirit level, check to make sure the control pipe is reasonably plumb, then carefully fill the pit with embankment material and compact in place.
7. Attach a post to the top of the protective sleeve to alert construction equipment operators of the obstruction.

NOTE: It has been found that a 1.8-m long 2-by-4 painted with alternate 0.3 m wide stripes of red and white is satisfactory for this use. It is recommended that flagging be attached to the top of this post. The post should be

attached so that it can be easily removed and reattached as additional pipe is added during embankment construction.

C. COLLECTION OF DATA

1. As soon as possible after the settlement device has been installed, determine the elevation of the top of the 19-mm pipe coupling attached to the control pipe. Normally, the elevations required will be obtained by a survey party.
2. During embankment construction, the elevation of the top of the control pipe should be determined by survey approximately twice weekly. After embankment construction is completed, the elevation should be determined frequently enough to indicate significant changes in the rate of settlement. Normally, the time between surveys will be weekly immediately subsequent to completion of the embankment, and the interval between surveys will increase with time.
3. During fill placement, it will be necessary to extend the lengths of the control pipe and protective sleeve. When extending the control pipe, use the following procedure:
 - a. Determine the elevation to the top of the existing control pipe coupling.
 - b. Remove the protective post, attach a coupling to the length of control pipe to be added, and tighten the joint with pipe wrenches.
 - c. Insert the added length in the coupling on top of the existing control pipe and tighten the joint by using one pipe wrench on the existing coupling and one pipe wrench on the added length of control pipe. While tightening the joint, do not allow the coupling between the control pipe and the added length to turn. Turn only the added length of control pipe.
 - d. Measure the added length of control pipe, including the coupling. If possible, check this distance by determining the elevation of the control pipe.
 - e. Record the length of additional control pipe added under Column 5 on the form shown in Figure 9. Be sure to add this length to the previous value shown in Column 4.
 - f. Add and secure a 1.5-m length of protective sleeve to the existing sleeve and secure to the top of the post.
4. Enter all data on the form shown in Figure 9 as follows:

Column 1 - Record the reading date.

Column 2 - This column is not used.

Column 3 - This column is not used.

Column 4 - Record the elevation to top of control pipe as determined by survey.

Column 5 - Record the length of the control pipe.

Column 6 - Record the height of the riser above the original ground (Column 4 minus Column 5).

Column 7 - Record the total settlement to the nearest 0.002 m. This figure is obtained by subtracting the figure in Column 6 for the day being read from the figure at the top of Column 6 (elevation at the time of installation).

Column 8 - Record the elevation of the surface of the fill as determined by survey (that is optional).

Column 9 - Record the height of the fill above original ground to the nearest 0.1 m.

Column 10 - Record the number of calendar days elapsed since the settlement device was installed.

Column 11 - Record any information that would be helpful in the analysis of data. Be sure to indicate in this column the date and length added to the control pipe.

PART 3. SETTLEMENT DATA ANALYSIS SCOPE

The procedure for plotting and analyzing settlement data obtained from all types of settlement devices is described in this method. Comprehensive settlement analyses are complex and require extensive knowledge of soil mechanics and soil structure of the area under study. Considerable information, however, can be obtained by the simplified method described in this part.

1. Plot the data on a semi-logarithmic chart as shown in Figure 10. Note that the scale for days is on the logarithmic abscissa of the chart and both settlement and fill height are scaled arithmetically on the ordinate.
2. Note that during construction, the rate of settlement increases in approximate proportion to the fill load applied. This is generally true in all cases where the rate of loading embankment is nearly constant. If embankment construction is suspended for an appreciable length of time, the negative slope indicating rate of settlement should become more positive or flatter until embankment construction resumes. In no case, however, should the rate of settlement curve assume a positive slope.
 - a. A sudden increase in the rate of settlement during construction is an indication of impending failure and would dictate that fill loading be stopped immediately.
 - b. If the rate of settlement remains excessive after suspending fill operations, additional corrective

measures must be taken to reduce the rate of settlement.

NOTE: This may include removing a portion of the embankment or constructing berms or struts. Such measures usually require a comprehensive analysis and, for that reason, the problem must be brought to the attention of the Project Engineer without delay.

3. After embankment construction has been completed, the rate of settlement will decrease with time, especially for soft foundation soils. However, a marked decrease in the rate of settlement may be noticed until an appreciable amount of time has elapsed since completion of the embankment.
 - a. Any significant increase in the rate of settlement after completion of the embankment is sufficient cause for immediate corrective action as described above.
 - b. When the plotted data indicate that the slope of the rate of settlement curve is essentially horizontal, the embankment surcharge may be removed and/or permanent structure construction may be started. For example, from data shown in Figure 10, a practical minimal rate of settlement was obtained at about 360 days; at this time the embankment surcharge was removed as shown.
4. Data should be collected throughout the life of the contract. Longer data-collection periods are necessary if significant rates of settlement are measured.
 - a. The time interval between readings may be increased as the indicated rate of movement decreases.
 - b. Collection of data may be required for several years on selected projects. Long-term settlement data are

frequently useful in the design of embankments where similar conditions are encountered.

A, Section 5.0, Part B, Sections 5.0, 6.0 and 10.0 and Part C, Section 1.0 of Caltrans Laboratory Safety Manual. Users of this method do so at their own risk.

PART 4. SAFETY AND HEALTH

Prior to handling, testing or disposing of any waste materials, testers are required to read: Part

REFERENCES:

None

End of Text (California Test 112 contains 19 pages)

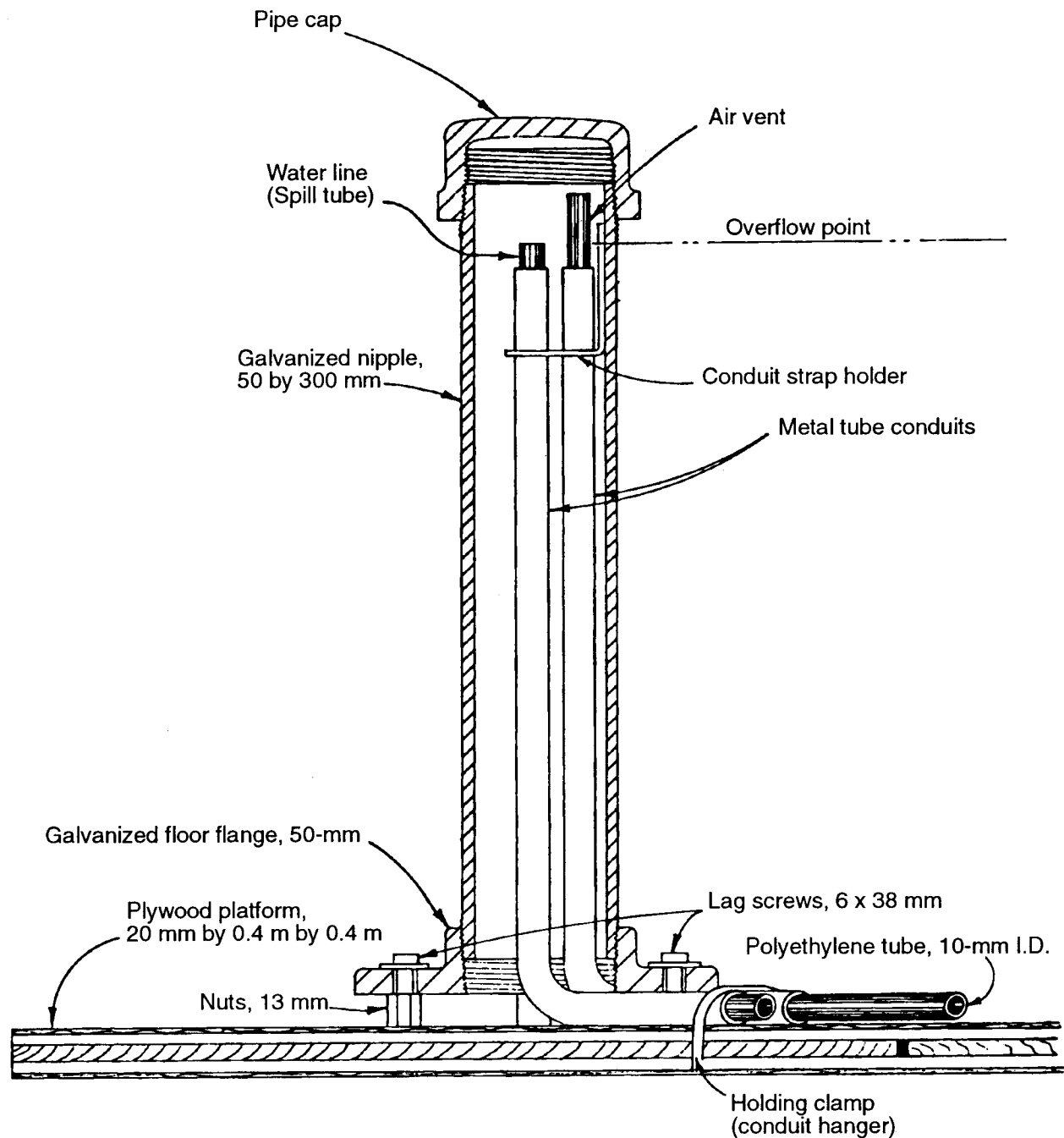


FIGURE 1 - VENTED STANDPIPE UNIT

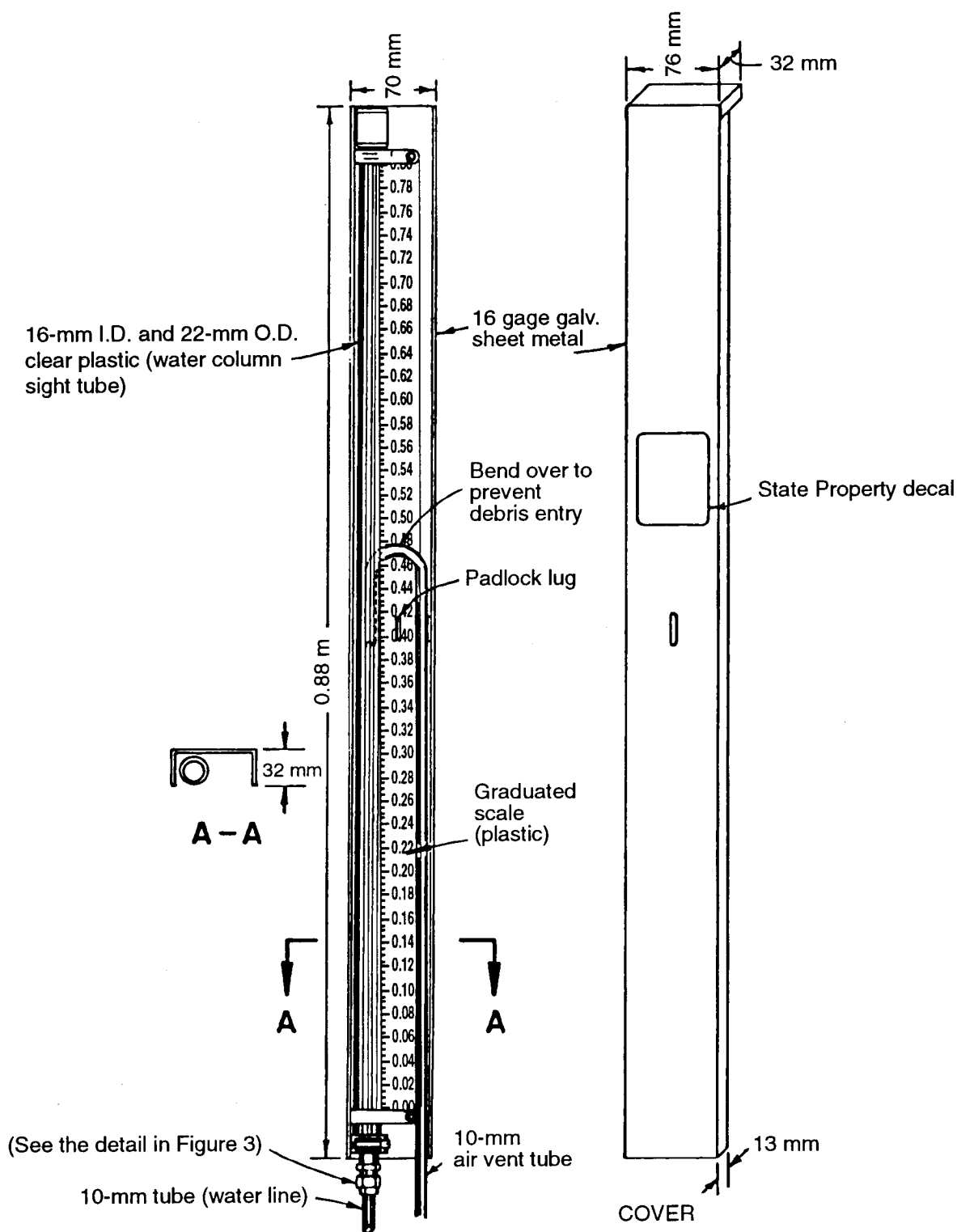


FIGURE 2 - INDICATOR UNIT

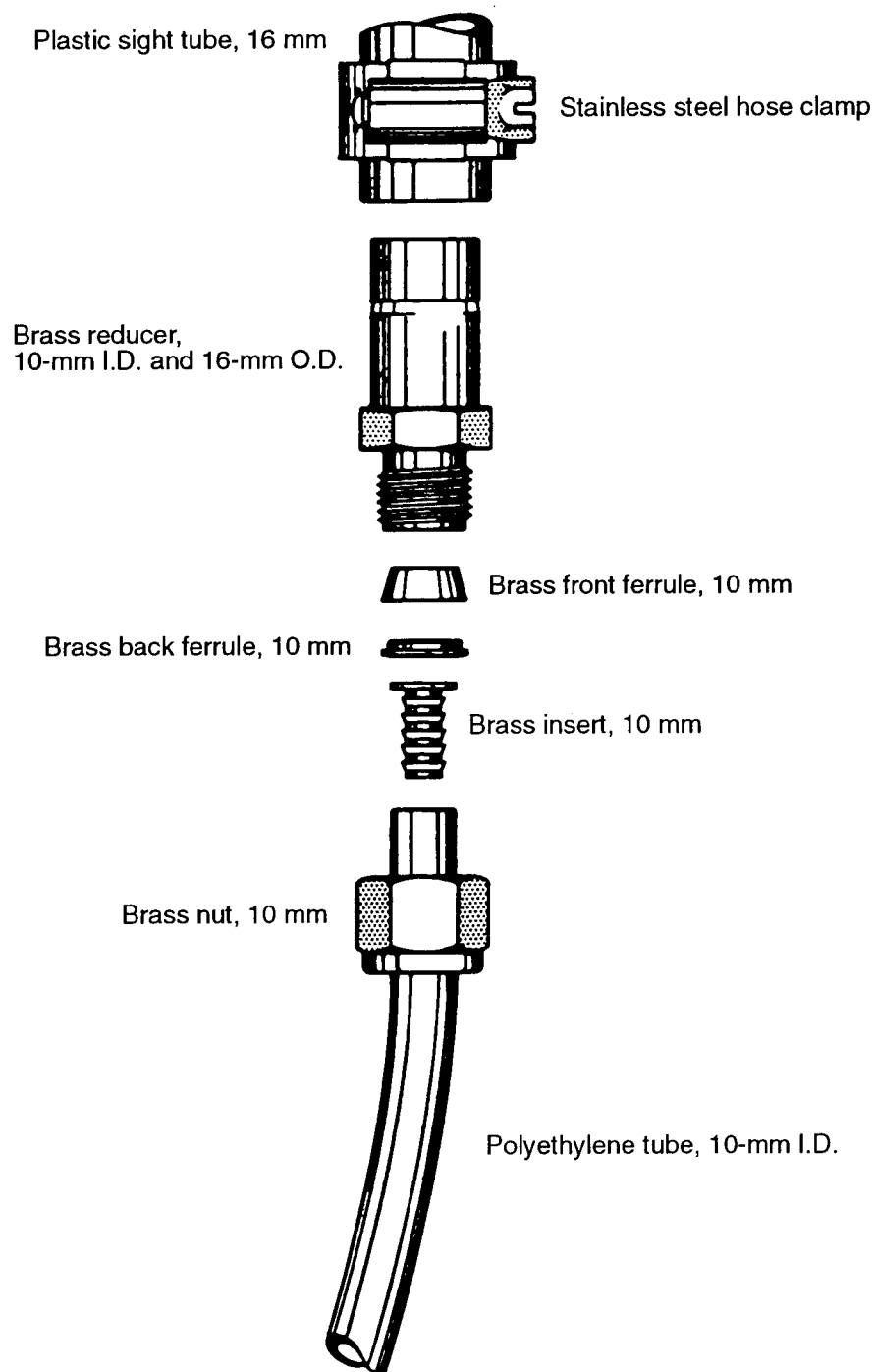


FIGURE 3 - DETAIL OF INDICATOR UNIT

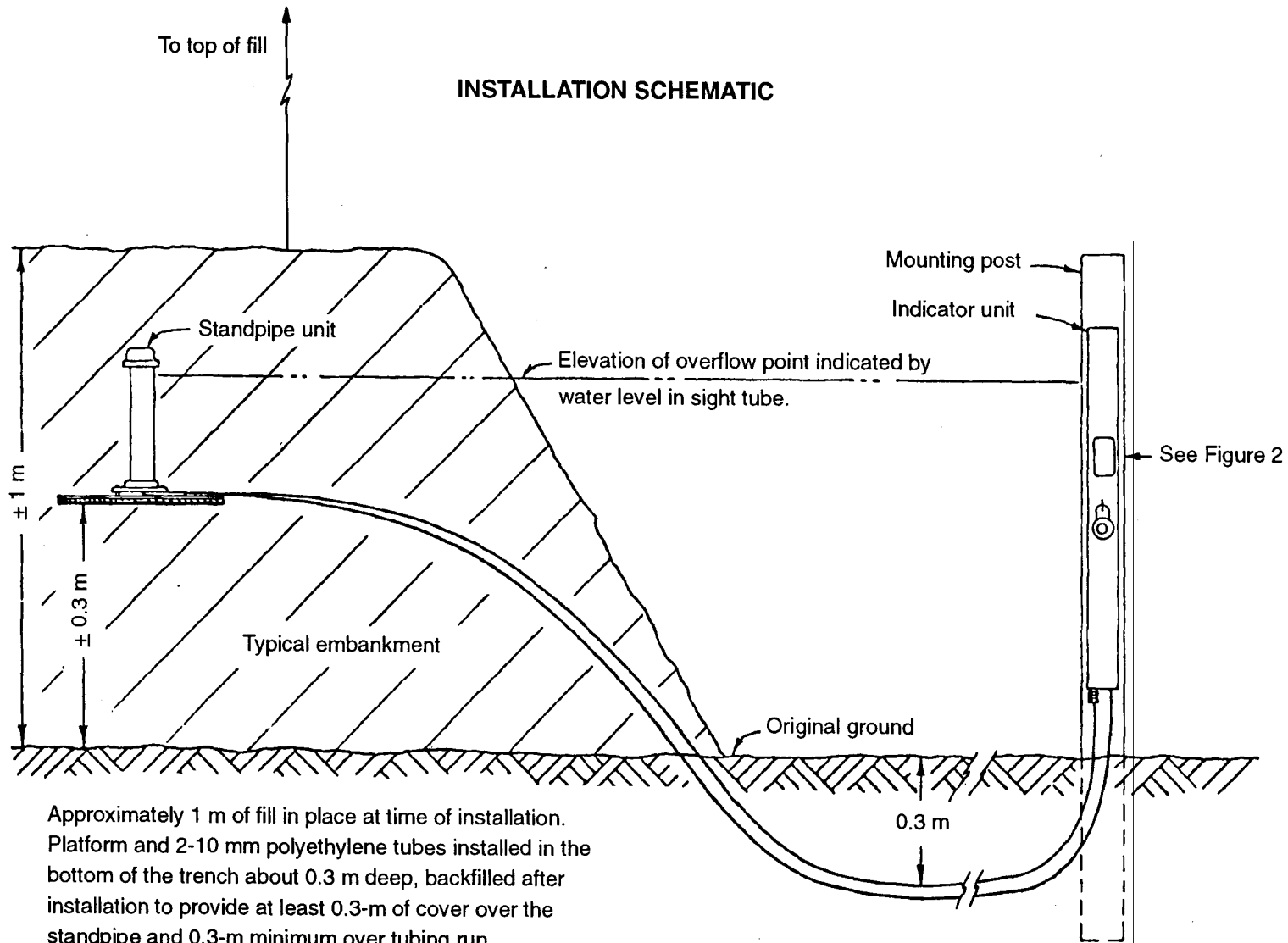


FIGURE 4 - SETTLEMENT INDICATOR DEVICE (VENTED TYPE)

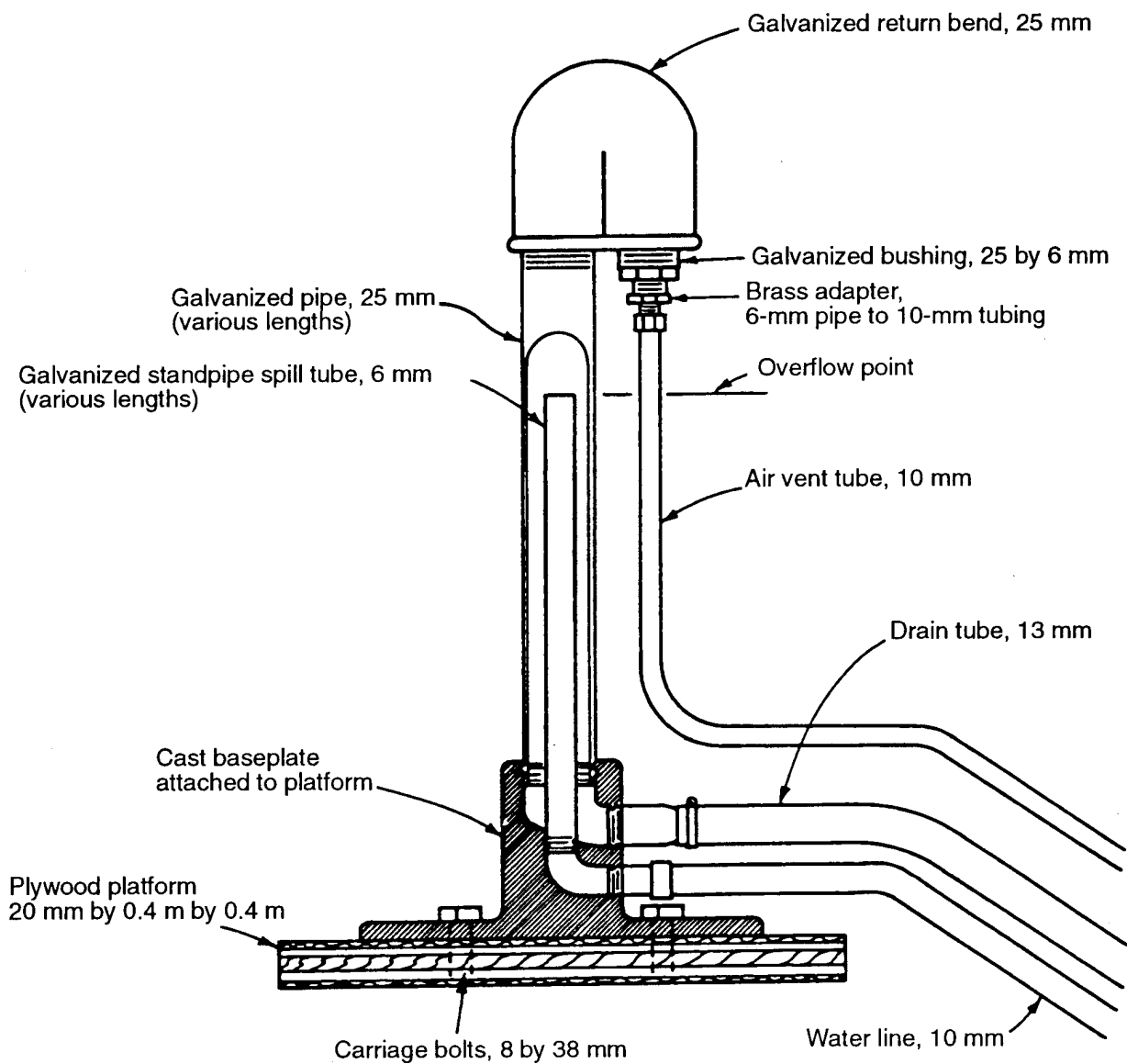


FIGURE 5 - SEALED STANDPIPE

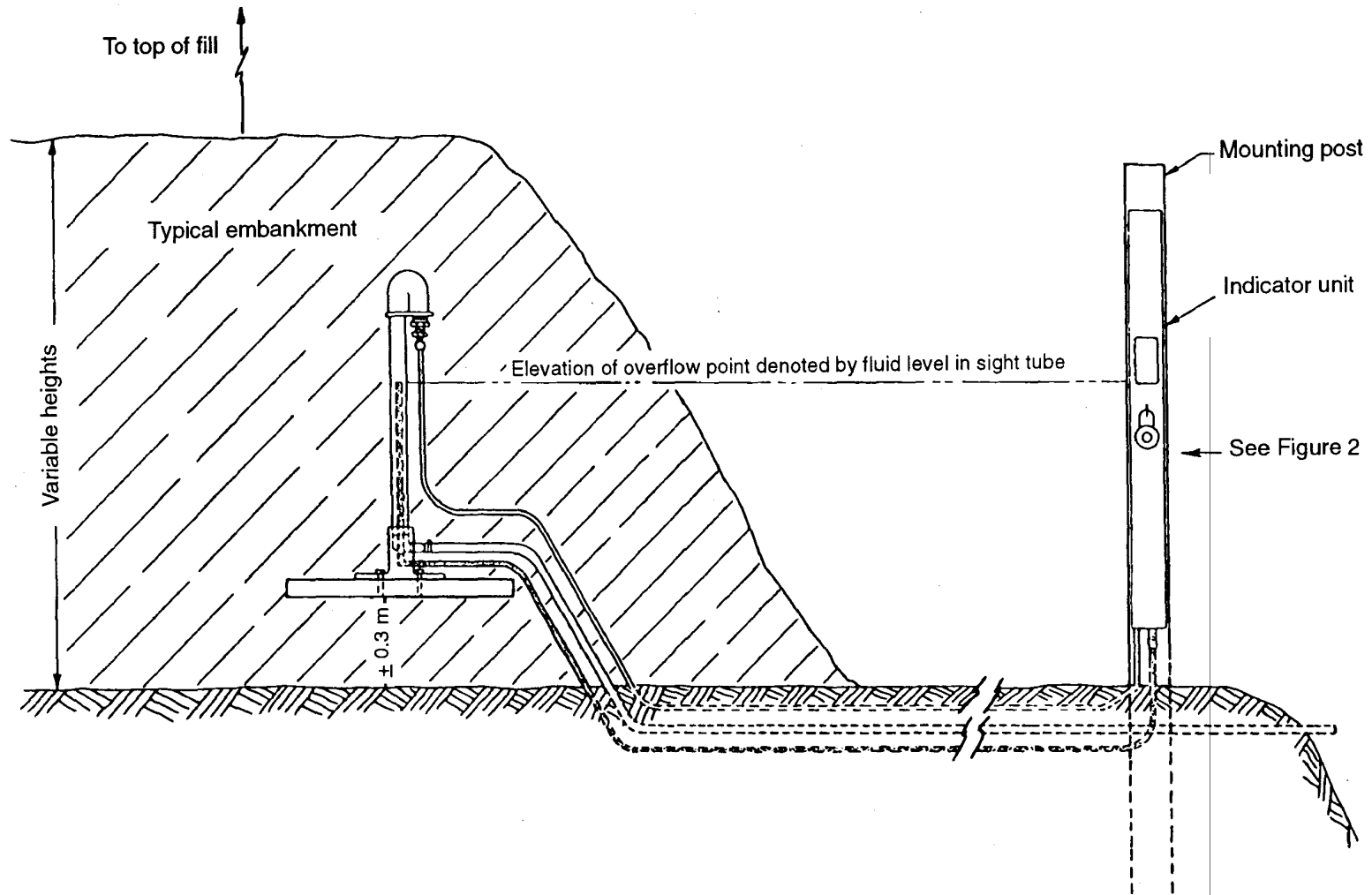


FIGURE 6 - SETTLEMENT INDICATOR DEVICE (SEALED FLUID LEVEL TYPE)

STATE OF CALIFORNIA		SETTLEMENT DATA		DEPARTMENT OF TRANSPORTATION	
Job Stamp	Install Loc.			Comments:	
	Br. No.		Unit No.		
	Sta. & Offset				
	Design Ht. (m) 6.4		Surcharge (m) 1.8		
	Waiting Period (days) 270				
	Controlled Loading 1 m per week				

Date (1)	Water Tube Reading (m) (2)	Reference Nail Elevation (m) (3)	RISER PIPE		Elevation Change (6)	Settlement (7)	FILL HEIGHT		Elapsed Time (days) (10)	Remarks (11)
			Top of Pipe Elevation (m) (4)	Length of Pipe (m) (5)			Surface Elevation (8)	Fill Above Original Ground (9)		
3/3/94	0.758	238.006			0.000	0.000	238.0	0.0	0	Platform set on O.G.
3/4/94	0.744				-0.014	0.014	238.9	0.9	1	
3/7/94	0.738				-0.006	0.020	239.8	1.8	4	
3/11/94	0.716				-0.022	0.042	240.7	2.7	8	
3/18/94	0.658				-0.058	0.100	241.7	3.7	15	
3/25/94	0.600				-0.058	0.158	242.6	4.6	22	
4/1/94	0.548	238.006			-0.052	0.210	243.5	5.5	29	
4/8/94	0.502				-0.046	0.256	244.4	6.4	36	
4/15/94	0.470				-0.032	0.288	245.3	7.3	43	
4/22/94	0.448				-0.022	0.310	246.2	8.2	50	Surcharge completed
4/26/94	0.430	238.000			-0.024	0.334	246.2	8.2	54	
5/12/94	0.408	238.000			-0.022	0.356	246.2	8.2	70	
5/25/94	0.396				-0.012	0.368	246.2	8.2	83	
6/8/94	0.384				-0.012	0.380	246.2	8.2	97	
7/6/94	0.388				0.004	0.376	246.2	8.2	125	
8/1/94	0.378	237.990			-0.020	0.396	246.2	8.2	151	
9/2/94	0.372				-0.006	0.402	246.2	8.2	183	
10/6/94	0.366	237.982			-0.014	0.416	246.2	8.2	217	
11/7/94	0.360				-0.006	0.422	246.2	8.2	249	
12/1/94	0.356	237.982			-0.004	0.426	246.2	8.2	273	Rain since last reading: 94mm
1/5/95	0.354	237.982			-0.002	0.428	246.2	8.2	308	Settlement terminated: 1/17/95
9/28/95	0.348	237.982			-0.006	0.434	244.4	6.4	574	Surcharge removed: 2/1/95

FIGURE 7 - SETTLEMENT DATA REPORT

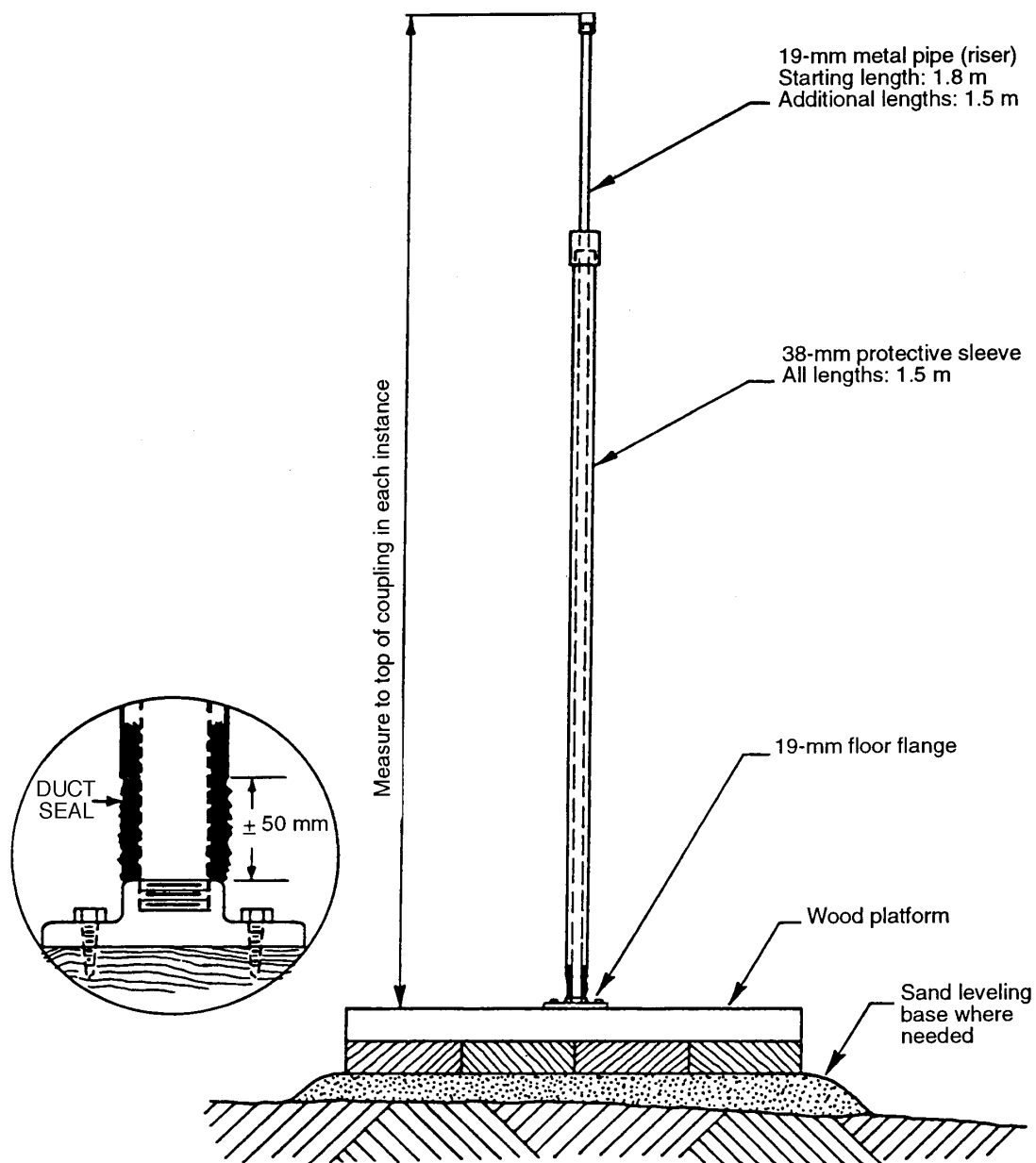
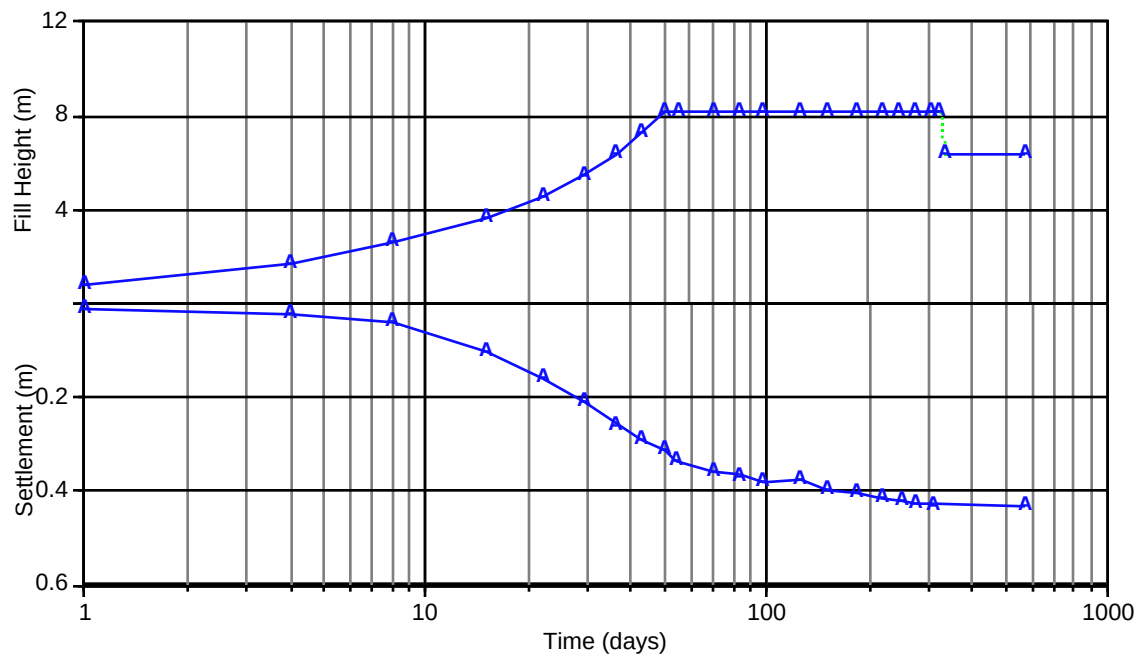


FIGURE 8 - PIPE RISER SETTLEMENT DEVICE

STATE OF CALIFORNIA		SETTLEMENT DATA		DEPARTMENT OF TRANSPORTATION	
Job Stamp		Install Loc.		Comments:	
		Br. No.	Unit No.		
		Sta. & Offset			
		Design Ht. (m) 15.1	Surcharge (m) 0		
		Waiting Period (days)			
		Controlled Loading			

Date (1)	Water Tube Reading (m) (2)	Reference Nail Elevation (m) (3)	RISER PIPE		Elevation Change (6)	Settlement (7)	FILL HEIGHT		Elapsed Time (days) (10)	Remarks (11)
			Top of Pipe Elevation (m) (4)	Length of Pipe (m) (5)			Surface Elevation (8)	Fill Above Original Ground (9)		
2/25/95			4.700	3.264	1.436	0.000		1.5	0	
2/27/95			6.304	4.880	1.424	0.012		2.4	2	
3/3/95			7.912	6.492	1.420	0.016		3.0	6	
3/6/95			7.916	6.492	1.424	0.012		4.6	9	
3/10/95			7.906	6.492	1.414	0.022		6.1	13	
3/20/95			9.498	8.102	1.396	0.040		7.8	23	
3/27/95			12.650	11.338	1.312	0.124		10.2	30	
4/6/95			17.300	16.176	1.124	0.312		14.9	40	
4/15/95			17.246	16.176	1.070	0.366		15.1	49	Fill completed
4/29/95			17.210	16.176	1.034	0.402		"	63	
5/7/95			17.184	16.176	1.008	0.428		"	71	
5/13/95			17.178	16.176	1.002	0.434		"	77	
5/23/95			17.160	16.176	0.984	0.452		"	87	
5/29/95			17.164	16.176	0.988	0.448		"	93	
6/6/95			17.152	16.176	0.976	0.460		"	101	
6/12/95			17.178	16.176	1.002	0.434		"	107	
6/20/95			17.138	16.176	0.962	0.474		"	115	
6/26/95			17.124	16.176	0.948	0.488		"	121	
7/17/95			17.124	16.176	0.948	0.488		"	142	
7/24/95			17.118	16.176	0.942	0.494		"	149	
8/4/95			17.108	16.176	0.932	0.504		"	160	

FIGURE 9 - SETTLEMENT DATA REPORT



Settlement Data

San Ramon Road OC
05-039733
Station: 2.3 m Rt. 8+50 F-30 Line
1 m Vertical Loading per Week

Abutment 1

Bridge No. 49-193
05-SLO-101-47.2/52.5
Installed: 3/3/94
Waiting Period Not to Exceed 270 days

FIGURE 10 - SETTLEMENT DATA ANALYSIS

APPENDIX D

GBA'S IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL-ENGINEERING REPORT

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*.



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