# APPENDIX E

*Geotechnical Report*

GEOTECHNICAL UPDATE REPORT
PROPOSED ALTA OCEANSIDE PROJECT
SOUTH OF COSTA PACIFIC WAY
WEST OF NORTH COAST HIGHWAY
OCEANSIDE, CALIFORNIA

Prepared for:

**WP WEST ACQUISITIONS, LLC**
**c/o WOOD PARTNERS**
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Project No. 12107.003

January 18, 2019

Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

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WP West Acquisitions, LLC
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7700 Irvine Center Drive, Suite 600
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Attention: Mr. Will Winkenhofer

Subject: Geotechnical Report, Proposed Alta Oceanside Project, South of Costa Pacific Way, West of North Coast Highway, Oceanside, California

In accordance with your request and authorization, we have prepared an updated geotechnical investigation report for the proposed Alta Oceanside project, located in Oceanside, California. Based on the results of our study, it is our professional opinion that the development of the site is geotechnically feasible provided the recommendations provided herein are incorporated into the design and construction of the proposed improvements. The accompanying report presents a summary of the existing conditions of the site, the results of our field investigation and laboratory testing, and provides geotechnical conclusions and recommendations relative to the proposed updated site development.

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If you have any questions regarding our report, please do not hesitate to contact this office. We appreciate this opportunity to be of service.

Respectfully submitted,

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# 1.0 INTRODUCTION

## 1.1 Purpose and Scope

This report presents the results of our preliminary geotechnical investigation for the proposed Alta Oceanside project is located in Oceanside, California (Figure 1). The purpose of our investigation was to evaluate the geotechnical conditions at the site and provide conclusions and recommendations relative to the proposed development. Our scope of services included the following:

- Review of published and unpublished geotechnical reports, maps and aerial photographs (Appendix A).
- Site reconnaissance.
- Excavation, logging and sampling of three exploratory borings and four field percolation tests. As background, four borings and two test pits had previously been performed on this site by Leighton (Leighton, 2001 and 2012). All boring and test pit logs are presented in Appendix B.
- Laboratory testing of representative soil samples obtained from the recent subsurface exploration. Results of these tests are presented in Appendix C.
- Development of seismic design parameters based on the 2016 California Building Code (CBC)
- Preparation of this report presenting our findings, conclusions, and geotechnical recommendations with respect to the proposed design, site grading and general construction considerations.

## 1.2 Site Location

The proposed site is located south of Costa Pacific Way and west of North Coast Highway in Oceanside, California. Currently, the western portion of the site is a vacant lot with sparse vegetation. The eastern portion of the site is occupied by commercial properties consisting of one-story buildings and paved parking lots. In general, the site is relatively flat; however, the northwestern corner of the site slopes down towards Costa Pacific Way. The existing surfaced elevations of the site are roughly 52 to 60 feet above mean sea level (msl). The site is bounded by Costa Pacific Way to the north, mobile homes to the west, North Coast Highway to the east, and mixture of mobile homes and hotel property to the south.

Site Latitude and Longitude
33.20317º N
-117.3851º W

## 1.3 Proposed Development

Leighton's understands that the project proposes the construction of a mixed-use residential and commercial development, with demolition of the existing commercial uses on the site. The residential component would include 309-units comprised of one, two, and three-bedroom residences. The commercial component would include 5,615 square feet of restaurant, retail and/or visitor uses on the ground floor along North Coast Highway. There are two buildings proposed, a five-story (65-foot tall) apartment/commercial building that would wrap around a six-level parking garage, and a second smaller three-story building located in the southwest corner of the site, where the 10 units include individual garages. Additionally, based on our review of the ALTA Oceanside Civil Base Map by Hunsaker & Associates, San Diego, Inc., (Hunsaker & Associates, 2019), typical cut and fill grading techniques would be required to bring the site to design grades. Based on our review, cuts and fills are currently proposed up to about 2 and 3 feet in thickness, respectively (excluding remedial grading). Import soils are anticipated, as the site does not appear to currently balance.

Preliminary grading and foundation plans or structural loads were not available prior to the preparation of this report. Currently, we are assuming that the parking structure will be constructed of reinforced concrete. For the apartment structures, we are assuming it will be constructed of wood frame construction. Associated improvements including underground utilities, hardscaping, landscaping and retaining walls are also anticipated.

## 1.4 Previous Geotechnical Studies

In 1991, Leighton performed a geotechnical investigation for the proposed Sea Walk Village, which was never built. That study included the excavation of one boring (B-4) and two test pits (T-9 and T-11) on the subject property. In 2001, Leighton performed an additional investigation for the initial phase of the Renaissance Terrace project located immediately west of the subject site, which included two test pit explorations (T-7 and T-8) beneath Costa Pacific Way. Subsequently, in 2012, a site-specific investigation was performed for a proposed development in the northwest portion of the project site that consisted of three borings, B-1, B-2 and B-3 (Leighton, 2012). Information from these previous reports was utilized as appropriate in this report.

## 2.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

Our current subsurface exploration consisted of the excavation of three (3) additional exploratory borings (HSA-1 through HSA-3) with a truck mounted hollow-stem auger drill rig. The purpose of these excavations was to evaluate the physical characteristics of the onsite soils pertinent to the proposed improvements. The borings allowed evaluation of the soils encountered within project site beneath the proposed structure(s), and to provide representative samples for laboratory testing. Additionally, four field percolation tests were performed at the site as part of the subsurface exploration. The field percolation test holes were also advanced using a heavy-duty truck-mounted hollow-stem auger drill rig to a depth of approximately 5 feet bgs. It should also be noted that no indications (odors, staining, etc.) of hydrocarbon impacted soils were observed during drilling. The approximate locations of the recent borings and previous explorations are shown on the Geotechnical Map, Figure 2.

The exploratory excavations were logged by a representative from our firm. Representative bulk and undisturbed samples were obtained at frequent intervals for laboratory testing, and logs of the borings are presented in Appendix B. Subsequent to logging and sampling, the borings were backfilled with bentonite.

In-situ field percolation testing was performed on August 14, 2018 in general accordance with Appendix C and D of the City of Oceanside BMP Design Manual, For Permeant Site Design, Strom Water Treatment and Hydromodification Management, dated February 2016. The level of introduced water in each field percolation test location was measured at 30-minute intervals using a water level sounder until readings where generally steady

Laboratory testing was performed on representative samples to evaluate the moisture, density, expansion potential of the soils to be encountered near foundation elevations, shear strength, and geo-chemical (corrosion) characteristics of the subsurface soils. A discussion of the laboratory tests performed, and a summary of the laboratory test results are presented in Appendix C. In-situ moisture and density test results are provided on the boring logs (Appendix B).

# 3.0 SUMMARY OF GEOTECHNICAL CONDITIONS

## 3.1 Regional Geology

The site is located within the coastal subprovince of the Peninsular Ranges Geomorphic Province, near the western edge of the southern California batholith. Throughout the last 54 million years, the area known as the "San Diego Embayment" has gone through several episodes of marine inundation and subsequent marine regression, resulting in the deposition of a thick sequence of marine and nonmarine sedimentary rocks on the basement rock of the Southern California batholithic.

Gradual emergence of the region from the sea occurred in Pleistocene time, and numerous wave-cut platforms, most of which were covered by relatively thin marine and nonmarine terrace deposits, formed as the sea receded from the land. Accelerated fluvial erosion during periods of heavy rainfall, coupled with the lowering of the base sea level during Quaternary times, resulted in the rolling hills, mesas, and deeply incised canyons which characterize the landforms we see in the general site area today.

## 3.2 Site Geology

Based on subsurface exploration, aerial photographic analysis, and review of pertinent geologic literature and maps, the geologic units underlying the site consists of relatively thin veneers of undocumented fill over Quaternary-aged Old Paralic Deposits, which overlies the Tertiary-aged San Mateo Formation. A brief description of the geologic units encountered on the site is presented below.

### 3.2.1 Undocumented Fill Soils (Afu)

The undocumented fill soils generally consist of loose to medium dense silty sands that are generally less than 1 to 2 feet in depth and our generally associated with previous development of the site. However, deeper areas of undocumented fills associated with previously site development (i.e., utility trench backfill and other excavations) should be anticipated across the site. The fill soils typically consisted of silty sand, are dry, loose, and may settle appreciably under additional fill or foundation and improvement loadings. Therefore, all undocumented fills (soils) should be removed and recompacted. These materials may be reused provided they are cleared of debris and/or oversized materials. All trash and debris should be removed, and properly disposed offsite, prior to fill placement and/or remedial grading.

### 3.2.2 Quaternary Old Paralic Deposits

Quaternary-aged Old Paralic (Terrace) Deposits were encountered at shallow depths during our investigation. As encountered, these soils were observed to generally consist of orange-brown to red brown, damp to moist, medium dense to very dense silty fine to medium grained sands with localized cobble lenses, and sandy silt. These units are massive and abundant iron oxide staining was visible throughout the exposures. The weathered near surface Old Paralic Deposits (upper 1 to 2 feet), where encountered, should be removed and recompacted, if not removed by planned excavation, should settlement sensitive improvements be proposed within its influence.

### 3.2.3 Tertiary-aged San Mateo Formation

The Tertiary-aged San Mateo Formation underlies the entire site at depth. As encountered in the recent boring and previous trench explorations to the west (Leighton, 2001), the San Mateo Formation generally consists of moderately well bedded to laminated, yellow-gray to orange-brown and light gray silty to very silty fine grained micaceous sandstone and massive, friable, gray silty fine- to medium-grained sandstone to sandy siltstone.

## 3.3 Geologic Structure

Based on the results of our current investigation, literature review, and our professional experience on nearby sites, the Old Paralic Deposits are generally massive with no well-defined bedding. The San Mateo Formation is massive to poorly-to moderately indurated sandstone with regional bedding of 5 degrees to the northwest.

## 3.4 Groundwater

Groundwater was not encountered within our recent explorations although localized perched water may seasonally be encountered along geologic contacts. In summary, groundwater is not expected to impact the proposed development considering the estimated depth of the proposed improvements. However, groundwater may be encountered during deep excavations, such as, piles for shoring, if a basement or deep excavation is proposed. In addition, seepage conditions may locally be encountered after periods of heavy rainfall or irrigation. These conditions can be treated on an individual basis during construction, if they occur.

## 3.5 Engineering Characteristics of Onsite Soils

Based on the results of our current geotechnical investigation, laboratory testing of representative onsite soils and our professional experience on adjacent sites with similar soils, the engineering characteristics of the onsite soils are discussed below.

### 3.5.1 Compressible Soils

The site is underlain by weathered Old Paralic Deposits (i.e., upper near surface material) and undocumented fill materials which are considered compressible in their current state. Recommendations for remedial grading of these soils are provided in the following sections of this report.

### 3.5.2 Expansion Potential

The onsite undocumented fill and Old Paralic Deposits are anticipated to be in the very low to low expansion range. Geotechnical observations and/or laboratory testing of the finish grade soils are recommended during construction to determine the actual expansion potential of soils at grade.

### 3.5.3 Soil Corrosivity

The National Association of Corrosion Engineers (NACE) defines corrosion as “a deterioration of a substance or its properties because of a reaction with its environment”. From a geotechnical viewpoint, the “environment” is the prevailing foundation soils and the “substances” are reinforced concrete foundations or various types of metallic buried elements such as piles, pipes, etc. that are in contact with or within close vicinity of the soil. In general soil environments that are detrimental to concrete have high concentrations of soluble sulfates and/or pH values of less than 5.5. The 2016 CBC and ACI 318-08 provides specific guidelines for the concrete mix-design when the soluble sulfate content of the soil exceeds 0.1 percent by weight or 1,000 parts per million (ppm).

The results of our laboratory tests on representative formational soils from the site indicated a soluble sulfate content of less than 0.015 percent and a pH of 6.83, and indicate chloride content of approximately 200 ppm. A minimum resistivity value of 3,075 ohm-cm indicating a moderate corrosion potential. The test results indicate that the sulfate contents of the subsurface soils are categorized as having a sulfate exposure class “S0” (formerly identified as negligible) ACI 318R-14 Table 19.3.2.1. Utilizing Caltrans criteria, the site is not considered to be corrosive to concrete. However, is considered corrosive to uncoated buried metal conduits.

#### 3.5.4 Excavation Characteristics

It is anticipated the onsite surficial fill soils and Paralic (Terrace) Deposits can be excavated with conventional heavy-duty construction equipment. Excavations within the San Mateo Formation may encounter moderately cemented material that can locally be difficult to excavate. Heavy ripping or breaking may be needed during the excavation of moderately to well cemented material, if encountered. Oversize material (larger than 8 inches in maximum dimensions), if encountered, should be placed in non-structural areas or hauled off site.

### 3.6 Infiltration

In accordance with City of Oceanside BMP Design Manual, Section D.3.3.2, the Borehole Percolation Test Method was selected and performed between August 14, 2018. Specifically, four 8-inch diameter borings, PT-1 through PT-4, were advanced to a depth of approximately 5 feet below the existing ground surface (bgs). Approximate locations of the percolation test borings are shown on the attached Geotechnical Map (Figure 2). Given the test holes consisted of silty sand soils, the boreholes were filled with water and allowed to pre-soak for at least 15 hours prior to performing the in-situ percolation testing.

In summary, the in-situ percolation testing was performed in accordance with Oceanside BMP Design Manual, Section D.3.3.2. The water level in each test hole was measured at approximately 30-minute intervals using a water level sounder, which is accurate to 0.01 feet. Refilling of the boreholes was done as needed. The total testing period lasted approximately 6 hours. Field testing data is presented in Appendix D. After the conclusion of in-situ percolation testing, the boreholes were backfilled.

### 3.6.1 Percolation and Infiltration Rates

Based on our field percolation testing, the in-situ percolation rates and calculated infiltration rates at tested locations and depths are summarized in Table 1 below. It is important to note that percolation rates are not equal to infiltration rates. As a result, we have made a distinction between percolation rates where water movement is considered laterally and vertically versus infiltration rates where only the vertical direction is considered. We have used the Porchet Method to convert measured percolation rates to calculated infiltration rates in accordance with County of San Diego Standards (2016). In addition, we have included a recommended infiltration rate with a minimum factor of safety of 2 for the preliminary design of potential infiltration systems.

Table 1
Percolation and Infiltration Rates

| Perc. Test No. | Boring Depth (ft) | Soil Type | Measured Percolation Rate (mins/in) | Calculated Infiltration Rate (inches/hr) | Recommended Infiltration Rate w/ FS of 2 (inches/hr) |
|---|---|---|---|---|---|
| PT-1 | 5.0 | Silty Sand | 250 | 0.012 | 0.006 |
| PT-2 | 4.89 | Silty Sand | 250 | 0.012 | 0.006 |
| PT-3 | 4.59 | Silty Sand | 475 | 0.015 | 0.008 |
| PT-4 | 4.78 | Silty Sand | 125 | 0.028 | 0.014 |

It should be emphasized that the percolation test results are only representative of the tested location and depth where they are performed.

Varying subsurface conditions may exist outside of the test locations, which could alter the calculated infiltration rates indicated above.

In addition, it is possible that long term infiltration rates within measured soil strata may be much lower than the values obtained by our current testing. Long term infiltration can be influenced by: variable vertical character and limited lateral extent of more permeable soil strata, reduction of permeability rates over time due to silting of the soil pore spaces, and other factors not discussed here. Accordingly, the possibility of future surface ponding of water as well as shallow groundwater impacts on subterranean structures such as basements, underground utilities, etc. should be anticipated as possible future conditions in all design aspects of the site.

#### 3.6.2 Geotechnical Feasibility of Infiltration

From a geotechnical perspective, the following factors should also be considered for the feasibility of infiltration:

- Soil and Geologic Conditions

  Based on our review and our current investigation, the geologic units underlying the site consist of undocumented fill soils, Paralic Deposits and the San Mateo Formation at depth.

  Old Paralic Deposits were encountered within the percolation test locations and is anticipated to vary in depth. These soils generally consisted of medium to very dense silty sands, which have very low infiltration rates. The San Mateo Formation at depth was not encountered within percolation test boreholes.

- Settlement and Volume Change

  Based on our evaluation, the on-site soils remaining following construction are not considered susceptible to hydro-collapse or consolidation. Considering planned grading and foundation design measures, settlement potential is considered negligible due to infiltration. Our testing also did not indicate the presence of highly expansive soils. In addition, it is our professional opinion that site soils are not liquefiable due to their dense condition and absence of a shallow ground water.

- Slope Stability

  The topography of the site is relatively flat with exception of the 2 to 1 cut slope at the northwest corner of the site. Infiltration near slopes should be mitigated during BMP design.

- Utility Considerations

  Currently, the locations of existing and proposed underground utilities are unknown relative to the tested sites.

- Groundwater Mounding

  Groundwater was not encountered during our site investigation. Based on the anticipated depth of groundwater across the site and the generally very low infiltration rates measured across the project site, groundwater mounding resulting from possible infiltration of storm water is not considered significant.

- Retaining Wall and Foundations

  There are currently existing retaining walls and subsurface basements located to the west down gradient of the sites which can be affected. However, mitigation can include subsurface vertical barriers and subdrains to limit subsurface water migration and preached groundwater conditions.

- Findings

  The measured percolation and calculated infiltration rates presented above may be used for the planning level screening phase of design. Once the locations of proposed infiltration facilities/systems are known, additional percolation testing may be needed to verify values provided in this letter for use in the design phase. During the design phase, it should be noted that an elevated factor or safety may also be used by designers in lieu of additional field testing.

Based on the results of our current infiltration study, the site may be considered a “Non-Infiltration Site” based on City of Oceanside BMP Design Manual, For Permeant Site Design, Strom Water Treatment and Hydromodification Management (2016). Specifically, the recommended infiltration rate is between 0.006 and 0.014 inches per hour. Note that PT-4 infiltration test data indicated a partially infiltration rate, but this data is considered non-characteristic of soil conditions at the site. Therefore, this result was disregarded during feasibility elevation of the site for classification of site infiltration. We attribute this very low infiltration rate due to the dense nature of the underlying soils. The City of Oceanside Worksheet I-8 is presented in Appendix D.

Based on our professional experience, sites having such low infiltration rates are best suited for Low Impact Development (LID) BMPs that contain and filter surface waters by the use of flow-through planters and bioretention areas which are partially or fully lined with an impermeable liner and have subdrain systems that ties into an approved existing or proposed storm drain system. It should be noted that shallow bioswales, infiltration basins, and other unlined onsite detention and retention systems utilized in areas having less than 0.01 inches per hour infiltration rates can potentially create perched ground water conditions and surface seepage conditions off-site, if not mitigated by BMP design.

# 4.0 FAULTING AND SEISMICITY

## 4.1 Faulting

Our discussion of faults on the site is prefaced with a discussion of California legislation and state policies concerning the classification and land-use criteria associated with faults. By definition of the California Mining and Geology Board, an active fault is a fault which has had surface displacement within Holocene time (about the last 11,000 years). The state geologist has defined a pre-Holocene fault as any fault considered to have been active during Quaternary time (last 1,600,000 years). This definition is used in delineating Earthquake Fault Zones as mandated by the Alquist-Priolo Earthquake Faulting Zones Act of 1972 and as most recently revised in 2007 (Bryant and Hart, 2007). The intent of this act is to assure that unwise urban development and certain habitable structures do not occur across the traces of active faults. Based on our review, the site is not located within any Earthquake Fault Zone (EFZ) as created by the Alquist-Priolo Act.

Our review of available geologic literature (Appendix A) indicates that there are no known major or active faults on or in the immediate vicinity of the site. The nearest active regional fault is the Newport-Inglewood Fault Zone located offshore approximately 3.7 miles west of the site.

## 4.2 Seismicity

The site can be considered to lie within a seismically active region, as can all of Southern California. The effect of seismic shaking may be mitigated by adhering to California Building Code (CBC, 2016) for proposed subject lot. The following seismic design parameters have been determined in accordance with the 2016 CBC and the USGS Seismic Design Values tool (Version 5.1.0).

| Table 2<br>2016 CBC Mapped Spectral Acceleration Parameters | | | |
|---|---|---|---|
| Site Class | | | D |
| Site Coefficients | $F_a$<br>$F_v$ | =<br>= | 1.029<br>1.547 |
| Mapped $MCE_R$ Spectral Accelerations | $S_S$<br>$S_1$ | =<br>= | 1.178<br>0.453 |
| Site Modified $MCE_R$ Spectral Accelerations | $S_{MS}$<br>$S_{M1}$ | =<br>= | 1.212<br>0.701 |
| Design Spectral Accelerations | $S_{DS}$<br>$S_{D1}$ | =<br>= | 0.808<br>0.467 |

Utilizing ASCE Standard 7-10, in accordance with Section 11.4.1, the following additional parameters for the peak horizontal ground acceleration are associated with the Geometric Mean Maximum Considered Earthquake (MCEG). The mapped MCEG peak ground acceleration (PGA) is 0.468g for the site. For a Site Class D, the $F_{PGA}$ is 1.032 and the mapped peak ground acceleration adjusted for Site Class effects ($PGA_M$) is 0.483g for the site.

### 4.2.1 Shallow Ground Rupture

Ground rupture because of active faulting is not likely to occur on site due to the absence of known active faults. Cracking due to shaking from distant seismic events is not considered a significant hazard, although it is a possibility at any site in Southern California.

### 4.2.2 Liquefaction and Dynamic Settlement

Liquefaction and dynamic settlement of soils can be caused by strong vibratory motion due to earthquakes. Both research and historical data indicate that loose, saturated, granular soils are susceptible to liquefaction and dynamic settlement. Liquefaction is typified by a loss of shear strength in the affected soil layer, thereby causing the soil to behave as a viscous liquid. This effect may be manifested by excessive settlements and sand boils at the ground surface.

Based on our evaluation, the on-site soils are not considered liquefiable due to their relatively dense condition. Considering planned excavation and foundation design measures, dynamic settlement potential is also considered negligible.

4.2.3 Tsunamis and Seiches

A tsunami is a sea wave generated by submarine earthquakes, landslides or volcanic activity that displaces a relatively large volume of water in a very short period. Several factors at the originating point such as; earthquake magnitude, type of fault, depth of earthquake, focus, water depth, and the ocean bottom profile all contribute to the size and momentum of a tsunami (Iida, 1969). Factors such as the distance away from the originating point, coastline profile (including width of the continental shelf), and angle at which the tsunami approaches also affect the size and severity of a tsunami.

The Southern California coastline is not only favorably oriented (i.e. not directly in line with any of the major originating tsunami zones), it has a relatively wide (about 140 miles) and rugged continental shelf or borderland, which acts as a diffuser and reflector of remotely, generated tsunami wave energy (Joy, 1968). In addition, the existing geologic and seismic conditions (such as the abundance of strike-slip faults, and the scarcity of large submarine earthquakes) along the coastline also tend to minimize the likelihood of a localized tsunami.

Based on our review of the San Diego County Tsunami Inundation Map for Emergency Planning, Oceanside/San Luis Rey Quadrangle, (CalEMA, 2009), the favorable geologic and seismic conditions along the coastline, and site elevation, there is little potential for catastrophic damage due to tsunamis.

## 5.0 CONCLUSIONS

Based on the results of our geotechnical investigation of the site, it is our professional opinion that the proposed development is feasible from a geotechnical standpoint, provided the following conclusions and recommendations are implemented during the design and construction of the project. The following is a summary of the significant geotechnical factors that may affect development of the site.

- Based on our subsurface exploration, the undocumented artificial fill and upper 1 to 2 feet of the weathered Old Paralic Deposits are considered unsuitable for supporting additional fill or structural loads and should be removed and recompacted, if not removed by planned excavation, should settlement sensitive improvements be proposed within its influence.

- Results of our exploration indicate that dense Old Paralic Deposits can be excavated with conventional heavy-duty earthwork equipment. However, excavation within the San Mateo Formation may require heavy ripping.

- All undocumented fill soil and weathered Old Paralic Deposits (upper 1 to 2 feet) should be removed to competent Old Paralic Deposits, if not removed by proposed excavation, within areas proposed for settlement-sensitive improvements. On a preliminary basis, remedial grading is estimated to consists of the removal of the upper 1 to 2 feet, or greater, of undocumented fill soils and/or weathered Old Paralic Deposits below exiting grades. Actual depths of removals will be evaluated in the field during grading by the geotechnical engineer.

- In order to reduce the potential for differential settlements between cut and fill materials, the entire cut portion of cut/fill transitions beneath the proposed building and/or structures should be overexcavated to a minimum depth of 3 feet below finish grade, or to a maximum ratio of fill thickness of 3:1 (maximum to minimum), and replaced with compacted fill.

- Ground water was not encountered within the recent borings, and ground water is not expected to impact the proposed development considering the estimated excavation depths being less than 10 feet bgs. However, seepage conditions may locally be encountered after periods of heavy rainfall or irrigation.

- The onsite undocumented fill and Old Paralic Deposits appear to be suitable material for reuse as compacted fill provided they are relatively free of organic material, debris, and rock fragments larger than 6 inches in maximum dimension.

- Based on laboratory testing, the onsite soils are expected to have a negligible potential for sulfate attack on concrete. These soils are also considered to have a potential for corrosion to buried uncoated metal conduits. Laboratory testing should be performed on the finish grade soils to verify the corrosivity characteristics.

- Active or potentially active faults are not known to exist on or in the immediate vicinity of the site. In addition, the on-site soils are not considered liquefiable due to their relatively dense condition.

# 6.0 RECOMMENDATIONS

## 6.1 Earthwork

We anticipate that earthwork at the site will consist of site preparation, excavation, and backfill. We recommend that earthwork on the site be performed in accordance with the current City of Oceanside grading ordinances and the following recommendations and the General Earthwork and Grading Specifications for Rough Grading included in Appendix D. In case of conflict, the following recommendations shall supersede those in Appendix D.

### 6.1.1 Site Preparation and Remedial Grading

Prior to grading, all areas to receive structural fill or engineered structures should be cleared of surface and subsurface obstructions, including any existing debris, old improvements, foundations, and undocumented or loose fill soils, and stripped of vegetation. Removed vegetation and debris should be properly disposed off site. All areas to receive fill and/or other surface improvements should be scarified to a minimum depth of 6 inches, brought to optimum moisture conditions, and recompacted to at least 90 percent relative compaction (based on American Standard of Testing and Materials [ASTM] Test Method D1557).

The undocumented artificial fill and upper ±1 to ±2 feet of the weathered Old Paralic Deposits are considered unsuitable for supporting additional fill or structural loads and should be removed and reprocessed within the proposed improvement areas. However, locally deeper removals cannot be precluded and should be anticipated. The actual depth and extent of the required removals should be evaluated during grading operations by the geotechnical consultant.

### 6.1.2 Transition and Overexcavation Areas

In order to reduce the potential for differential settlements between cut and fill materials, the entire cut portion of cut/fill transitions beneath the proposed building and/or structures should be overexcavated to a minimum depth of 3 feet below finish grade, or to a maximum ratio of fill thickness of 3:1 (maximum to minimum), and replaced with compacted fill. The recommended overexcavation should extend at least 5 feet beyond the proposed footprint of the buildings and/or structures. The preliminary cut/fill daylight line is shown on the Geotechnical Map (Figure 2) and the recommended overexcavation depth is shown on Geologic Cross Section A-A' (Figure 3).

#### 6.1.3 Excavations and Oversize Material

Excavations of the onsite materials may generally be accomplished with conventional heavy-duty earthwork equipment. Temporary shallow excavations less than 5 feet in depth with vertical sides should remain stable for the period required to construct the utility, provided the trenches are free of adverse geologic conditions (i.e., friable sands) and are not surcharged by static building loads or traffic loads. It should be noted that artificial fill soils, if encountered, are typically less dense than the Paralic Deposits and may cave during excavation. Friable layers within Paralic deposits may slough or cave during shoring construction especially if excavations are below the water table. In accordance with OSHA requirements, excavations deeper than 5 feet should be laid back or shored in accordance with Section 6.2, if workers are to enter such excavations.

#### 6.1.4 Fill Placement and Compaction

The onsite soils are generally suitable for use as compacted fill provided they are free of organic material, debris, and rock fragments larger than 6 inches in maximum dimension. The onsite soils typically possesses a moisture content below optimum and may require moisture conditioning prior to reuse as compacted fill. All fill soils should be brought to 2 percent above-optimum moisture conditions and compacted in uniform lifts to at least 90 percent relative compaction based on laboratory standard ASTM Test Method D1557. Note that for retaining wall (if any) supporting settlement sensitive improvements, such as patio pavements, sidewalks and/or foundations, we recommend backfill soils be compacted to 95 percent relative compaction based on ASTM Test Method D1557. The optimum lift thickness required to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in lifts not exceeding 8 inches in thickness.

Placement and compaction of fill should be performed in general accordance with the current City of Oceanside grading ordinances, sound construction practice, and the General Earthwork and Grading Specifications for Rough Grading presented in Appendix D.

## 6.2 Shoring of Excavations

If deep excavations are needed, we recommend that excavations be retained either by a cantilever or braced shoring system with cast-in-place soldier piles and sheeting or lagging (i.e. shotcrete and/or wood), as needed. It should be noted that a tie-back restrained pile system may encounter a caving condition. Based on our experience with similar projects, if lateral movement of the shoring system on the order of 1 inch cannot be tolerated, we recommend the utilization of a braced pile system.

Shoring of excavations is typically performed by specialty contractors with knowledge of the San Diego County area soil conditions. Lateral earth pressures for design of shoring are presented below:

Cantilever Shoring System

Active pressure = 35H(psf), triangular distribution
Passive Pressure = 350h (psf)
H = wall height (active case) or h = embedment (passive case)

Multi-Braced Shoring System

Active Pressure = 29H (psf), rectangular distribution
Passive Pressure = 350h (psf)
H = wall height (active case) or h = embedment (passive case)

General

All pressures are based on dewatered conditions, with the water table at least 4 feet below the base of the excavation. All shoring systems should consider additional loading of adjacent surcharging loads.

If portions of the planned excavations are proposed with sloped temporary excavations, we recommend a maximum slope of 1 to 1 (horizontal to vertical).

Sloped excavations should be observed by the geotechnical consultant during excavation.

Settlement monitoring of adjacent building, sidewalks and adjacent settlement sensitive structures should be considered to evaluate the performance of the shoring. Shoring of the excavation is the responsibility of the contractor. Extreme caution should be used to minimize damage to existing pavement, utilities, and/or structures caused by settlement or reduction of lateral support.

## 6.3 Surface Drainage and Erosion

Surface drainage should be controlled at all times. The proposed structure should have an appropriate drainage system to collect roof runoff. Positive surface drainage should be provided to direct surface water away from the structure toward the street or suitable drainage facilities. Planters should be designed with provisions for drainage to the storm drain system. Ponding of water should be avoided adjacent to the structure.

Regarding Low Impact Development (LID) measures, we are of the opinion that infiltration basins, and other onsite storm water retention and infiltration systems can potentially create adverse perched ground water conditions. In addition, the existing onsite soils are anticipated to provide relatively low or minimal infiltration rates for the surface water. Therefore, given the site geologic conditions, relatively very low infiltration rate, and protect type, infiltration type LID measures are not considered to be appropriate for this site and project.

## 6.4 Foundation and Slab Considerations

Foundations and slabs should be designed in accordance with structural considerations and the following recommendations. These recommendations assume that the soils encountered within 5 feet of pad grade have a very low to medium potential (i.e. expansion index <70) for expansion. If highly expansive soils are encountered, additional foundation design may be necessary.

### 6.4.1 Foundations

Based on the current site conditions, we recommend the proposed buildings and/or structures be supported by conventional footings on compacted fill soils. For conventional continuous or isolated spread footings, a maximum allowable bearing pressure of 3,000 pounds per square foot (psf) is recommended. Footings should extend a minimum of 24 inches beneath the lowest adjacent soil grade. The allowable pressure may be increased by one-third when considering loads of short duration such as wind or seismic forces. The minimum recommended width of footings is 18 inches for continuous footings and 24 inches for square or round footings. Continuous footings should be designed in accordance with the structural engineer requirements and have a minimum reinforcement of four No. 5 reinforcing bars (two top and two bottom). Reinforcement of isolated footings should be per the structural engineer's design.

The recommended allowable bearing capacities are based on a maximum total and differential settlement of 1 inch and 3/4 of an inch, respectively, with all footings founded in competent artificial fill material. Since settlements are a function of footing size and contact bearing pressures, some differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists.

#### 6.4.2 Foundation Setback

We recommend a minimum horizontal setback distance from the face of slopes for all structural foundations, footings, and other settlement-sensitive structures as indicated on the table below. This distance is measured from the outside bottom edge of the footing, horizontally to the slope face, and is based on the slope height. However, the foundation setback distance may be revised by the geotechnical consultant on a case-by-case basis if the geotechnical conditions are different than anticipated.

| Table 3<br>Minimum Foundation Setback from Slope Faces | |
|---|---|
| Slope Height | Setback |
| less than 5 feet | 5 feet |
| 5 to 15 feet | 7 feet |
| 15 to 30 feet | 10 feet |

Please note that the soils within the structural setback area possess poor lateral stability, and improvements (such as retaining walls, sidewalks, fences, pavements, etc.) constructed within this setback area may be subject to lateral movement and/or differential settlement. Potential distress to such improvements may be mitigated by providing a deepened footing or a grade beam foundation system to support the improvement. Depending on their proximity to the top of slopes, these buildings may require retaining walls and/or deepened foundations.

In addition, open or backfilled utility trenches that parallel or nearly parallel structure footings should not encroach within an imaginary 1:1 (horizontal to vertical) downward sloping line starting 9 inches above the bottom edge of the footing and should also not be located closer than 18 inches from the face of the footing. Deepened footings should meet the setbacks as described above. Also, over-excavation should be accomplished such that deepening of footings to accomplish the setback will not introduce a cut/fill transition bearing condition.

Where pipes may cross under footings, the footings should be specially designed. Pipe sleeves should be provided where pipes cross through footings or footing walls and sleeve clearances should provide for possible footing settlement, but not less than 1 inch around the pipe.

#### 6.4.3 Slab Design

The building slab-on-grade floors should be at least 5 inches thick and be reinforced with No. 3 rebars 18 inches on center, each way. All reinforcing should be placed at mid-height in the slab. In addition, the parking structure slab should also be designed for the anticipated traffic loading using a modulus of subgrade reaction of 250 pci. It is anticipated that actual slab thickness will be increased to accommodate design pressure. Slabs should have crack joints at spacings designed by the structural engineer. Columns should be structurally isolated from slabs.

Floor slabs with moisture-sensitive flooring should be underlain by 2 inches of sand, a 10-mil plastic sheeting moisture barrier and an addition 2 inches of sand (i.e., a total sand thickness of 4 inches). We recommend control joints be provided across the slab at appropriate intervals as designed by the project architect. In addition, we recommend using reinforcement and/or dowels between footings and floor slabs and sidewalks at doorways and openings to mitigate potential differential movements.

#### 6.4.4 Moisture Conditioning

The slab subgrade soils underlying the foundation systems should be presoaked in accordance with the recommendations presented in Table 4 prior to placement of the moisture barrier and slab concrete. The subgrade soil moisture content should be checked by a representative of Leighton prior to slab construction.

Presoaking or moisture conditioning may be achieved in a number of ways. But based on our professional experience, we have found that minimizing the moisture loss on pads that have been completed (by periodic wetting to keep the upper portion of the pad from drying out) and/or berming the lot and flooding for a short period of time (days to a few weeks) are some of the more efficient ways to meet the presoaking recommendations. If flooding is performed, a couple of days to let the upper portion of the pad dry out and form a crust so equipment can be utilized should be anticipated.

| Table 4<br>Presoaking Recommendations Based on Finish Grade Soil Expansion Potential | |
|---|---|
| Expansion Potential | Presoaking Recommendations |
| Very Low | Near-optimum moisture content to a minimum depth of 6 inches |
| Low | 120 percent of the optimum moisture content to a minimum depth of 12 inches below slab subgrade |
| Medium | 130 percent of the optimum moisture content to a minimum depth of 18 inches below slab subgrade |

6.4.5 Lateral Earth Pressures and Retaining Wall Design

Should retaining walls be added to the project, Table 5 presents the lateral earth pressure values for level or sloping backfill for walls backfilled with and bearing against fully drained soils of very low to low expansion potential (less than 50 per ASTM D4829).

| Table 5<br>Static Equivalent Fluid Weight (pcf) | | |
|---|---|---|
| Conditions | Level | 2:1 Slope |
| Active | 35 | 55 |
| At-Rest | 55 | 65 |
| Passive | 350<br>(Maximum of 3 ksf) | 150<br>(Sloping Down) |

Walls up to 10 feet in height should be designed for the applicable equivalent fluid unit weight values provided above. If conditions other than those covered herein are anticipated, the equivalent fluid unit weight values should be provided on an individual case-by-case basis by the geotechnical engineer. A surcharge load for a restrained or unrestrained wall resulting from automobile traffic may be assumed to be equivalent to a uniform lateral pressure of 75 psf which is in addition to the equivalent fluid pressure given above. For other uniform surcharge loads, a uniform pressure equal to 0.35q should be applied to the wall. The wall pressures assume walls are backfilled with free draining materials and water is not allowed to accumulate behind walls. A typical drainage design is contained in Appendix E. Wall backfill should be compacted by mechanical methods to at least 90 percent relative compaction (based on ASTM D1557). If foundations are planned over the backfill, the backfill should be compacted to 95 percent. Wall footings should be designed in accordance with the foundation design recommendations and reinforced in accordance with structural considerations. For all retaining walls, we recommend a minimum horizontal distance from the outside base of the footing to daylight as outlined in Section 6.4.2.

Lateral soil resistance developed against lateral structural movement can be obtained from the passive pressure value provided above. Further, for sliding resistance, the friction coefficient of 0.35 may be used at the concrete and soil interface. These values may be increased by one-third when considering loads of short duration including wind or seismic loads. The total resistance may be taken as the sum of the frictional and passive resistance provided that the passive portion does not exceed two-thirds of the total resistance.

To account for potential redistribution of forces during a seismic event, retaining walls providing lateral support where exterior grades on opposites sides differ by more than 6 feet fall under the requirements of 2016 CBC Section 1803.5.12 and/or ASCE 7-10 Section 15.6.1 and should also be analyzed for seismic loading. For that analysis, an additional uniform lateral seismic force of 9H should be considered for the design of the retaining walls with level backfill, where H is the height of the wall. This value should be increased by 150% for restrained walls.

## 6.5 Control of Surface Waters

Surface water should be transported off the site in approved drainage devices or unobstructed swales. We recommend a minimum flow gradient for unpaved drainage within 5 feet of structures of 2 percent sloping away. All area drain inlets should be maintained and kept clear of debris in order to function properly. In addition, landscaping should not cause any obstruction to site drainage. Rerouting of drainage patterns and/or installation of area drains should be performed, if necessary, by a qualified civil engineer or a landscape architect.

## 6.6 Concrete Flatwork

Concrete sidewalks and other flatwork (including construction joints) should be designed by the project civil engineer and should have a minimum thickness of 4 inches with No. 4 bars at 24 inches on center or No. 3 bars at 18 inches on center. For all concrete flatwork, the upper 12 inches of subgrade soils should be moisture conditioned to at least 2 percent above optimum moisture content depending on the soil type and compacted to at least 90 percent relative compaction based on ASTM Test Method D1557 prior to the concrete placement. Moisture testing should be confirmed 24 hours prior to concrete placement.

## 6.7 Preliminary Pavement Design

Flexible pavements for the project are not currently anticipated. However, should flexible pavements be constructed, they should be constructed in accordance with current Caltrans and the requirements outlined in the City of Oceanside Engineers Design and Processing Manual (2004). Based on a review of previous geotechnical report and this current study, for planning purposed flexible pavement with an assumed R-Value of 20 and a Traffic Index of 5 should be for 4 inches of Asphalt Concrete (AC) over 6 inches of Aggregate Base (AB).

For areas subject to regular truck loading (i.e., trash truck apron and Fire Lane), we recommend a full depth of Portland Cement Concrete (PCC) section of 7 inches and should be reinforced with No. 3 reinforcement bars spaced 24 inches on center each way, to reduce the potential for shrinkage cracking. Control joints should be spaced every 10 feet. Actual steel reinforcement and crack-control joints as designed by the project structural engineer. We recommend that sections be as nearly square as possible. A 3,500-psi mix that produces a 550-psi modulus of rupture should be utilized.

All pavement section materials should conform to and be placed in accordance with the latest revision of the California Department of Transportation Standard Specifications (Caltrans) and American Concrete Institute (ACI) codes. The upper 8 inches of subgrade soil and all aggregate base (if utilized) should be compacted to a relative compaction of at least 95 percent (based on ASTM Test Method D1557) and to a moisture content above optimum content.

If pavement areas are adjacent to heavily watered landscape areas, we recommend some measure of moisture control be taken to prevent the subgrade soils from becoming saturated. It is recommended that the concrete curbing separating the landscaping area from the pavement extend below the aggregate base to help seal the ends of the sections where heavy landscape watering may have access to the aggregate base. Concrete swales should be designed in roadway or parking areas subject to concentrated surface runoff.

### 6.8 Construction Observation and Plan Reviews

The recommendations provided in this report are based on preliminary design information and subsurface conditions disclosed by a series of widely spaced borings. The interpolated subsurface conditions should be checked in the field during construction. Construction observation of all onsite excavations and field density testing of all compacted fill should be performed by a representative of this office so that construction is in accordance with the recommendations of this report. We recommend that where possible excavation exposures be geologically mapped by the geotechnical consultant during grading for the presence of potentially adverse geologic conditions.

Final project civil, foundation, and shoring, drawings should be reviewed by Leighton before excavation to see that the recommendations provided in this report are incorporated in the project plans.

## 7.0 LIMITATIONS

The conclusions and recommendations in this report are based in part upon data that were obtained from a limited number of observations, site visits, excavations, samples, and tests. Such information is by necessity incomplete. The nature of many sites is such that differing geotechnical or geological conditions can occur within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report can be relied upon only if Leighton has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site.

FIGURES


| Project: 12107.003 | Eng/Geol: BEV |
|---|---|
| Scale: 1 " = 2,000 ' | Date: January 2019 |

Base Map: Bing Maps 2019
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

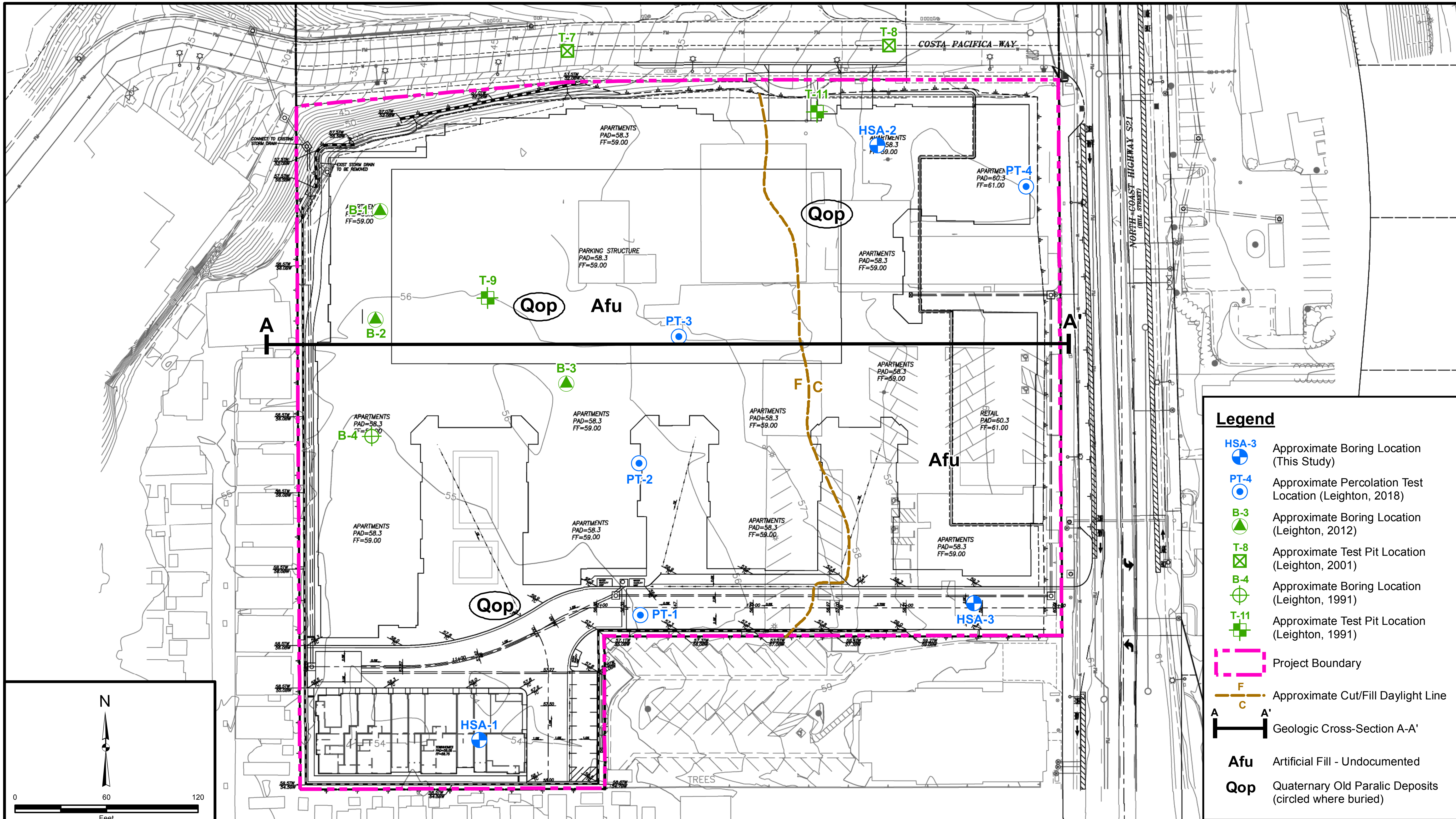
# SITE LOCATION MAP

Alta Oceanside Project
Oceanside, California

Figure 1

Leighton

Map Saved as V:\Drafting\12107\003\Maps\12107-003_F01_SLM_2019-01-17.mxd on 1/17/2019 1:04:07 PM




| | |
|---|---|
| Project: 12107.003 | Eng/Geol: BEV |
| Scale: 1 " = 60 ' | Date: January 2019 |

Base Map: Civil Base by Hunsaker & Associates, 1/3/2019.

Author: Leighton Geomatics (mmurphy)

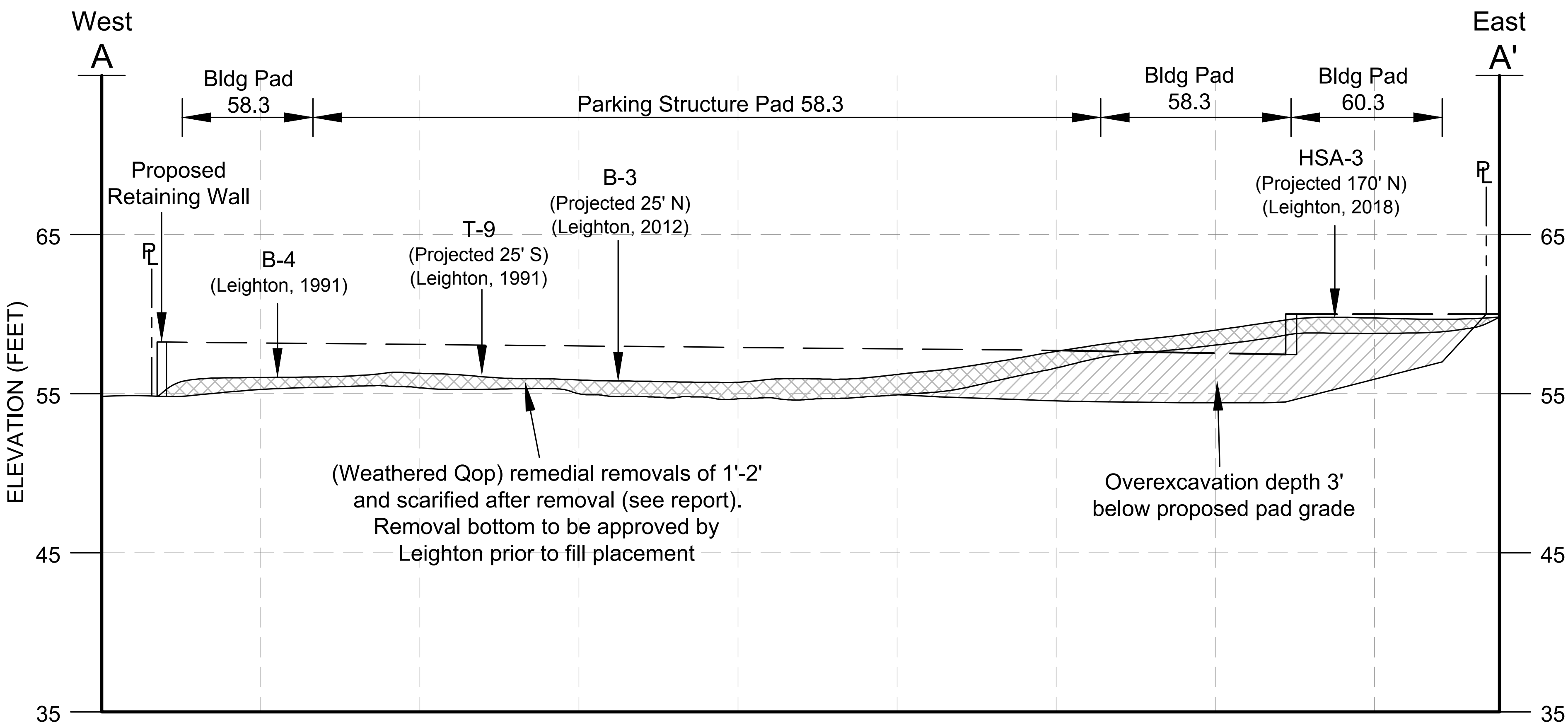
# GEOTECHNICAL MAP

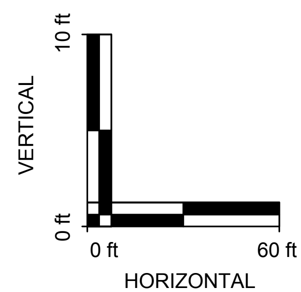
Alta Oceanside Project
Oceanside, California

Figure 2

Leighton

Map Saved as V:\Drafting\12107\003\Maps\12107-003_F02_ELM_2019-01-17.mxd on 1/17/2019 11:16:05 AM




See Figure 2 for Legend

| | | | |
|---|---|---|---|
| Project: 12107.003 | Eng/Geol: BEV | **GEOLOGIC CROSS-SECTION A-A'** | Figure 3 |
| Scale: 1"=60'H 1"=10'V | Date: January 2019 | Alta Oceanside Project | Leighton |
| Reference: | | Oceanside, California | |
| Author: MAM | | | |

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# APPENDIX A

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# APPENDIX A

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# APPENDIX B

# BORING, TEST PIT LOGS

# AND FIELD PERCOLATION TEST RESULTS

# GEOTECHNICAL BORING LOG KEY

Date ______ Sheet 1 of 1

Project KEY TO BORING LOG GRAPHICS Project No. ______

Drilling Co. ______ Type of Rig ______

Hole Diameter ______ Drive Weight ______ Drop "

Elevation Top of Elevation ' Location ______

| Elevation Feet | Depth Feet | Graphic Log (N S) | Attitudes | Sample No. | Blows Per Foot | Dry Density pcf | Moisture Content, % | Soil Class. (U.S.C.S.) | DESCRIPTION<br>Logged By ______<br>Sampled By ______ | Type of Tests |
|---|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | | | | | Asphaltic concrete. | |
| | | | | | | | | | Portland cement concrete. | |
| | | | | | | | | CL | Inorganic clay of low to medium plasticity; gravelly clay; sandy clay; silty clay; lean clay. | |
| | | | | | | | | CH | Inorganic clay; high plasticity, fat clays. | |
| | | | | | | | | OL | Organic clay; medium to plasticity, organic silts. | |
| | 5 | | | | | | | ML | Inorganic silt; clayey silt with low plasticity. | |
| | | | | | | | | MH | Inorganic silt; diatomaceous fine sandy or silty soils; elastic silt. | |
| | | | | | | | | ML-CL | Clayey silt to silty clay. | |
| | | | | | | | | GW | Well-graded gravel; gravel-sand mixture, little or no fines. | |
| | | | | | | | | GP | Poorly graded gravel; gravel-sand mixture, little or no fines. | |
| | 10 | | | | | | | GM | Silty gravel; gravel-sand-silt mixtures. | |
| | | | | | | | | GC | Clayey gravel; gravel-sand-clay mixtures. | |
| | | | | | | | | SW | Well-graded sand; gravelly sand, little or no fines. | |
| | | | | | | | | SP | Poorly graded sand; gravelly sand, little or no fines. | |
| | | | | | | | | SM | Silty sand; poorly graded sand-silt mixtures. | |
| | 15 | | | | | | | SC | Clayey sand; sand-clay mixtures. | |
| | | | | | | | | | Bedrock. | |
| | | | | | | | | | Ground water encountered at time of drilling. | |
| | | | | B-1 | | | | | Bulk Sample 1. | |
| | 20 | | | B-1 | | | | | Bulk Sample 2. | |
| | | | | C-1 | | | | | Core Sample. | |
| | | | | G-1 | | | | | Grab Sample. | |
| | | | | R-1 | | | | | Modified California Sampler (3" O.D., 2.5 I.D.). | |
| | | | | SH-1 | | | | | Shelby Tube Sampler (3" O.D.). | |
| | 25 | | | S-1 | | | | | Standard Penetration Test SPT (Sampler (2" O.D., 1.4" I.D.). | |
| | | | | PUSH | | | | | Sampler Penetrates without Hammer Blow. | |
| | | | | | | | | | Bulk Sample 2. | |
| | 30 | | | | | | | | | |

**SAMPLE TYPES:**

S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
SH SHELBY TUBE

**TYPE OF TESTS:**

DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION
SA SIEVE ANALYSIS
AT ATTERBURG LIMITS
EI EXPANSION INDEX
RV R-VALUE

LEIGHTON

# GEOTECHNICAL BORING LOG HSA-1

| | | | |
|---|---|---|---|
| **Project No.** | 12107.003 | **Date Drilled** | 8-9-18 |
| **Project** | Wood Partners/North Coast Highway | **Logged By** | ERB |
| **Drilling Co.** | Baja Exploration | **Hole Diameter** | 8" |
| **Drilling Method** | Hollow Stem Auger - 140lb - Autohammer - 30" Drop | **Ground Elevation** | 52' |
| **Location** | See Map | **Sampled By** | ERB |

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

| Elevation Feet | Depth Feet | Attitudes | Sample No. | Blows Per 6 Inches | Dry Density pcf | Moisture Content, % | Soil Class. (U.S.C.S.) | SOIL DESCRIPTION | Type of Tests |
|---|---|---|---|---|---|---|---|---|---|
| | 0 | | B-1 0-5' | | | | | ARTIFICIAL FILL (Afu)<br>@ 0-1': Silty SAND, light reddish-brown, dry, loose | EI, CR |
| 50 | | | | | | | SM | QUATERNARY OLD PARALIC DEPOSITS (Qop)<br>@ 1': Silty SAND, light reddish-brown, dry, very dense, fine sand, partially indurated, trace manganese nodules, micaceous | |
| | | | R-1 | 40<br>50/2" | 123.6 | 2.4 | | | DS |
| | 5 | | S-1 | 8<br>8<br>10 | | | | @ 5': Silty SAND, olive-gray to brown, dry, medium dense, fine sand, friable | |
| 45 | | | | | | | | @ 7': Minor gravel layer | |
| | 10 | | R-2 | 26<br>40<br>50/4" | 120.0 | 7.0 | SP | @ 10': Poorly-graded SAND, medium reddish-brown, dry, very dense, fine to coarse sand, partially indurated, micaceous | |
| 40 | | | | | | | GM | @ 12.5'-15': Dense gravel layer encountered | |
| | 15 | | S-2 | 23<br>30<br>38 | 112.0 | 6.0 | SM | @ 15': Silty SAND, medium brown, dry, very dense, fine to medium sand, friable, micaceous | |
| 35 | | | | | | | | | |
| | | | R-3 | 23<br>33<br>50/4" | | | SP | @ 18': Poorly-graded SAND, dark reddish-brown, damp, very dense, medium to coarse sand, partially indurated, horizontal oxidation lenses observed, micaceous | |
| | 20 | | | | | | | **Total Depth = 19.5 Feet (bgs)**<br>**No groundwater encountered at time of drilling**<br>**Backfilled with bentonite on 8/9/18** | |
| 30 | | | | | | | | | |
| | 25 | | | | | | | | |
| 25 | | | | | | | | | |
| | 30 | | | | | | | | |

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

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

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

# GEOTECHNICAL BORING LOG HSA-2

| | | | |
|---|---|---|---|
| **Project No.** | 12107.003 | **Date Drilled** | 8-9-18 |
| **Project** | Wood Partners/North Coast Highway | **Logged By** | ERB |
| **Drilling Co.** | Baja Exploration | **Hole Diameter** | 8" |
| **Drilling Method** | Hollow Stem Auger - 140lb - Autohammer - 30" Drop | **Ground Elevation** | 60' |
| **Location** | See Map | **Sampled By** | ERB |

| Elevation Feet | Depth Feet | Graphic Log (N S) | Attitudes | Sample No. | Blows Per 6 Inches | Dry Density pcf | Moisture Content, % | Soil Class. (U.S.C.S.) | SOIL DESCRIPTION | Type of Tests |
|---|---|---|---|---|---|---|---|---|---|---|
| 60 | 0 | | | B-1 0-5' | | | | SM | ARTIFICIAL FILL (Af)<br>@ 0-2': Silty SAND, light reddish-brown, dry, loose, trace asphalt debris | |
| | | | | S-1 | 17 50/2" | | | SM | QUATERNARY OLD PARALIC DEPOSITS (Qop)<br>@ 2': Silty SAND, light grayish-brown, dry, very dense, fine sand<br><br>@4': Color changed to olive-gray | |
| 55 | 5 | | | R-1 | 40 50/4" | 124.0 | 11.0 | SP | @ 5': Poorly-graded SAND, medium olive-gray, damp, very dense, medium to coarse sand, manganese nodules throughout, indurated, horizontal oxidation, micaceous | |
| 50 | 10 | | | S-2 | 17 23 35 | | | ML | @ 10': Sandy SILT, dark gray to dark brown, moist, very stiff, fine SAND, oxidation throughout, mottling throughout, infilled fracture surfaces observed throughout, micaceous | |
| 45 | 15 | | | R-2 | 25 26 36 | 105.0 | 15.0 | | @ 15': Continues at depth | |
| | | | | S-3 | 12 17 19 | | | SM | @ 18': SAND content increases, becomes silty sand | |
| 40 | 20 | | | | | | | | **Total Depth = 19.5 Feet (bgs)**<br>**No groundwater encountered at time of drilling**<br>**Backfilled with bentonite on 8/9/18** | |
| 35 | 25 | | | | | | | | | |
| 30 | 30 | | | | | | | | | |

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

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

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

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

# GEOTECHNICAL BORING LOG HSA-3

| | | | |
|---|---|---|---|
| **Project No.** | 12107.003 | **Date Drilled** | 8-9-18 |
| **Project** | Wood Partners/North Coast Highway | **Logged By** | ERB |
| **Drilling Co.** | Baja Exploration | **Hole Diameter** | 8" |
| **Drilling Method** | Hollow Stem Auger - 140lb - Autohammer - 30" Drop | **Ground Elevation** | 57' |
| **Location** | See Map | **Sampled By** | ERB |

## SOIL DESCRIPTION

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

| Elevation Feet | Depth Feet | Graphic Log (N S) | Attitudes | Sample No. | Blows Per 6 Inches | Dry Density pcf | Moisture Content, % | Soil Class. (U.S.C.S.) | SOIL DESCRIPTION | Type of Tests |
|---|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | B-1 0-5' | | | | SM | 3" ASPHALT CONCRETE over 9" AGGREGATE BASE | |
| 55 | | | | | | | | | ARTIFICIAL FILL (Afu)<br>@ 1': Silty SAND with GRAVEL, light yellowish-brown, damp, medium dense, fine to medium sand, subrounded to rounded gravel (1"-3") throughout, debris throughout<br>@ 3': Dark brown silty SAND observed in cuttings | |
| | 5 | | | R-1 B-2 5'-10' | 40 50/3: | 114.0 | 13.0 | SM | QUATERNARY OLD PARALIC DEPOSITS (Qop)<br>@ 5': Silty SAND, olive-brown, damp, very dense, fine sand, indurated, oxidation throughout, trace micas | |
| 50 | | | | | | | | | @ 8': Silty SAND, olive-brown observed in cuttings, noted increase in density | |
| | 10 | | | R-2 | 30 45 50/5" | 134.0 | 11.0 | SP | SAN MATEO FORMATION (Tsm)<br>@ 10': Poorly-graded SANDSTONE, light gray, damp, very dense, fine to coarse sand, partial induration, micaceous<br>@ 12': plasticity observed to increase at depth | |
| 45 | | | | | | | | | | |
| | 15 | | | R-3 | 22 40 50/2" | 126.0 | 9.0 | | @ 15': Poorly-graded SANDSTONE continues at depth, medium orangish-brown, damp, very dense, fine to carse sand, partial induration, manganese nodules observed throughout, micaceous | |
| 40 | | | | | | | | | **Total Depth = 16.5 Feet (bgs)**<br>**No groundwater encountered at time of drilling**<br>**Backfilled with bentonite on 8/9/18** | |
| | 20 | | | | | | | | | |
| 35 | | | | | | | | | | |
| | 25 | | | | | | | | | |
| 30 | | | | | | | | | | |
| | 30 | | | | | | | | | |

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

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

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# GEOTECHNICALBORING LOG B-1

| | | | |
|---|---|---|---|
| **Project No.** | 042593-001 | **Date Drilled** | 5-1-12 |
| **Project** | CH Oceanside | **Logged By** | BCP |
| **Drilling Co.** | Baja Exploration | **Hole Diameter** | 8" |
| **Drilling Method** | Hollow Stem Auger - 140lb - Autohammer - 30" Drop | **Ground Elevation** | 55' |
| **Location** | See Boring Location Map | **Sampled By** | BCP |

**SOIL DESCRIPTION**

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

| Elevation Feet | Depth Feet | Graphic Log (N S) | Attitudes | Sample No. | Blows Per 6 Inches | Dry Density pcf | Moisture Content, % | Soil Class. (U.S.C.S.) | SOIL DESCRIPTION | Type of Tests |
|---|---|---|---|---|---|---|---|---|---|---|
| 55 | 0 | | | | | | | | OLD PARALIC (TERACE) DEPOSITS<br>@Surface: Poorly graded SAND, reddish brown, minor silt | |
| 50 | 5 | | | | | | | | @ 4': Poorly graded, fine to medium SAND, light brown, some gravel, trace silt | |
| 45 | 10 | | | R-1 | 8<br>17<br>24 | 104 | 1 | SW | @ 10': well graded, fine-grained SAND, light brown, dry, hard, very dense | |
| | | | | | | | | | **Refusal @ 12 Feet**<br>**Total Depth = 12 Feet**<br>**Groundwater was not encountered during drilling**<br>**Backfilled with bentonite on 5/1/12** | |
| 40 | 15 | | | | | | | | | |
| 35 | 20 | | | | | | | | | |
| 30 | 25 | | | | | | | | | |
| 25 | 30 | | | | | | | | | |

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

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

# GEOTECHNICALBORING LOG B-2

| | | | |
|---|---|---|---|
| **Project No.** | 042593-001 | **Date Drilled** | 5-1-12 |
| **Project** | CH Oceanside | **Logged By** | BCP |
| **Drilling Co.** | Baja Exploration | **Hole Diameter** | 8" |
| **Drilling Method** | Hollow Stem Auger - 140lb - Autohammer - 30" Drop | **Ground Elevation** | 55' |
| **Location** | See Boring Location Map | **Sampled By** | BCP |

| Elevation Feet | Depth Feet | Graphic Log (N S) | Attitudes | Sample No. | Blows Per 6 Inches | Dry Density pcf | Moisture Content, % | Soil Class. (U.S.C.S.) | SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.* | Type of Tests |
|---|---|---|---|---|---|---|---|---|---|---|
| 55 | 0 | | | | | | | | OLD PARALIC (TERRACE) DEPOSITS<br>SAND with silt (SP), Surface: Poorly graded fine-grained silty SAND, reddish brown | |
| 50 | 5 | | | | | | | | | |
| 45 | 10 | | | | | | | | @10': well graded, fine-grained SAND, light brown | |
| | | | | | | | | | @ 12': Becomes poorly graded Gravelly SAND, fine to medium grained sand, well rounded gravels, trace silt. | |
| | | | | | | | | | **Refusal @ 13.5 Feet**<br>**Total Depth = 13.5 Feet**<br>**Groundwater was not encountered during drilling**<br>**Backfilled with bentonite on 5/1/12** | |
| 40 | 15 | | | | | | | | | |
| 35 | 20 | | | | | | | | | |
| 30 | 25 | | | | | | | | | |
| 25 | 30 | | | | | | | | | |

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

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

# GEOTECHNICALBORING LOG B-3

| | | | |
|---|---|---|---|
| **Project No.** | 042593-001 | **Date Drilled** | 5-1-12 |
| **Project** | CH Oceanside | **Logged By** | BCP |
| **Drilling Co.** | Baja Exploration | **Hole Diameter** | 8" |
| **Drilling Method** | Hollow Stem Auger - 140lb - Autohammer - 30" Drop | **Ground Elevation** | 55' |
| **Location** | See Boring Location Map | **Sampled By** | BCP |

| Elevation Feet | Depth Feet | Graphic Log (N S) | Attitudes | Sample No. | Blows Per 6 Inches | Dry Density pcf | Moisture Content, % | Soil Class. (U.S.C.S.) | **SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.* | Type of Tests |
|---|---|---|---|---|---|---|---|---|---|---|
| 55 | 0 | | | | | | | | OLD PARALIC (TERRACE) DEPOSITS<br>@ Surface: poorly graded, fine-grained silty SAND, reddish brown | |
| 50 | 5 | | | | | | | | | |
| 45 | 10 | | | R-1 | 22<br>34<br>50+ | 112 | 5 | SW | @ 10': well graded, fine-grained SAND, light brown to orange, dry, hard, very dense, trace silt | |
| 40 | 15 | | | S-1 | 18<br>50+ | | | GP | @15': well graded, fine-grained SAND, light brown to orange, dry, hard, very dense, trace silt<br>SAN ONOFRE BRECCIA(?)<br>@ 15.5': Becomes poorly graded Gravelly SAND, light brown, fine to medium sand, angular gravels, dry, hard, very dense | |
| | | | | B-1 | | | | SW | | |
| 35 | 20 | | | R-2 | 18<br>37<br>44 | 101 | 3 | SW | @ 20': well graded, fine to very fine SAND, light brown to tan, dry, hard, very dense | |
| 30 | 25 | | | S-2 | 14<br>18<br>24 | | | SW | @ 25': well graded, very fine SAND, light brown to white, dry to moist, hard, very dense | |
| 25 | 30 | | | | | | | | | |

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

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

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# GEOTECHNICALBORING LOG B-3

| | | | |
|---|---|---|---|
| **Project No.** | 042593-001 | **Date Drilled** | 5-1-12 |
| **Project** | CH Oceanside | **Logged By** | BCP |
| **Drilling Co.** | Baja Exploration | **Hole Diameter** | 8" |
| **Drilling Method** | Hollow Stem Auger - 140lb - Autohammer - 30" Drop | **Ground Elevation** | 55' |
| **Location** | See Boring Location Map | **Sampled By** | BCP |

| Elevation Feet | Depth Feet | Graphic Log (N S) | Attitudes | Sample No. | Blows Per 6 Inches | Dry Density pcf | Moisture Content, % | Soil Class. (U.S.C.S.) | SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.* | Type of Tests |
|---|---|---|---|---|---|---|---|---|---|---|
| 25 | 30 | | | R-3 | 15<br>36<br>50+ | 107 | 2 | SW | @ 30': well graded, fine-grained SAND, light brown to orange, dry, hard, dense | |
| | | | | | | | | | **Total Depth = 30 Feet**<br>**Groundwater was not encountered during drilling**<br>**Backfilled with bentonite on 5/1/12** | |
| 20 | 35 | | | | | | | | | |
| 15 | 40 | | | | | | | | | |
| 10 | 45 | | | | | | | | | |
| 5 | 50 | | | | | | | | | |
| 0 | 55 | | | | | | | | | |
| -5 | 60 | | | | | | | | | |

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

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

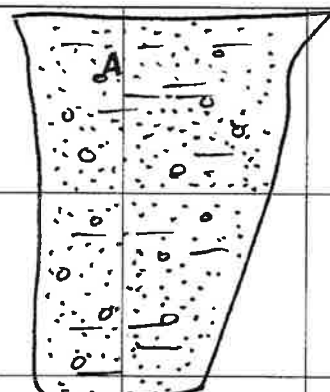
LOG OF TRENCH: T-7

Project Name: Concordia/Sea Walk
Logged by: MDJ
Project Number: 040259-001
Elevation: 55'
Equipment: 310 Backhoe
Location/Grid: See Geotechnical Map

| GEOLOGIC ATTITUDES | DATE: 12/15/00 DESCRIPTION: | GEOLOGIC UNIT | USCS | Sample No. | Moisture (%) | Density (pcf) |
|---|---|---|---|---|---|---|
| | QUATERNARY TERRACE DEPOSITS | Qt | | | | |
| | A @ 0'-10': Silty medium SAND: red-brown, damp, dense, becomes very dense at 9'; gravel rare | | SM | 1 @7'-8' | | |

ENGINEERING PROPERTIES

GRAPHICAL REPRESENTATION: SCALE: 1"=5' SURFACE SLOPE: 0° TREND: N25W




Total Depth = 10 Feet
No Ground Water Encountered
Backfilled: 12/15/2000

LOG OF TRENCH: T-8

Project Name: Concordia/Sea Walk | Logged by: MDJ

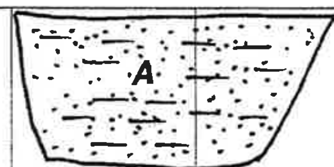
Project Number: 040259-001 | Elevation: 57'

Equipment: 310 Backhoe | Location/Grid: See Geotechnical Map

| GEOLOGIC ATTITUDES | DATE: 12/15/00 DESCRIPTION: | GEOLOGIC UNIT | USCS | Sample No. | Moisture (%) | Density (pcf) |
|---|---|---|---|---|---|---|
| | QUATERNARY TERRACE DEPOSITS | Qt | | | | |
| | A @ 0'-4': Silty medium SAND: brown, damp, dense to very dense; massive, slightly porous 0'-1'; gravel rare | | SM | | | |
| | Practical refusal at 4 feet on slightly cemented sand | | | | | |

ENGINEERING PROPERTIES

GRAPHICAL REPRESENTATION: SCALE: 1"=5' SURFACE SLOPE: 0° TREND: N-S




Total Depth = 4 Feet
No Ground Water Encountered
Backfilled: 12/15/2000

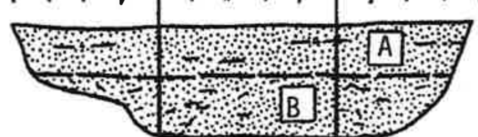
LOG OF TRENCH NO: T-11

Project Name: Seawalk Village
Project Number: 4910693-01
Equipment: Case 580 Backhoe
Logged By: JB
Elevation: 55'±
Location: See Geotechnical Map
TRENCH NO. T-11

| GEOLOGIC ATTITUDES | DATE: 9/18/91 DESCRIPTION: | GEOLOGIC UNIT | U.S.C.S. | Sample No. | Moisture (%) | Density (% Compaction) |
|---|---|---|---|---|---|---|
| | PLEISTOCENE TERRACE DEPOSITS | | | | | |
| | A Light brown to red-brown, damp to moist, dense, silty fine- to medium-grained sand; slightly porous in upper .5 foot | Qt | SM | | | |
| | B Olive-brown to brown, moist, dense, slightly clayey fine- to medium-grained; abundant calcium carbonate flecks present | | SC | | | |

ENGINEERING PROPERTIES: U.S.C.S., Sample No., Moisture (%), Density (% Compaction)

GRAPHIC REPRESENTATION NE Wall SCALE: 1" = 5' SURFACE SLOPE: Horiz TREND: N70W




Total depth = 3 feet
No ground water encountered
Backfilled: 9/18/91

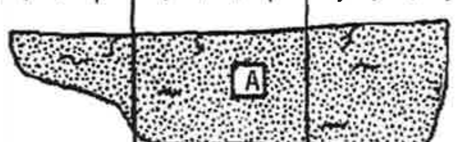
LOG OF TRENCH NO: T-9

Project Name: Seawalk Village
Project Number: 4910693-01
Equipment: Case 580 Backhoe
Logged By: JB
Elevation: 53'±
Location: See Geotechnical Map
TRENCH NO. T-9

| GEOLOGIC ATTITUDES | DATE: 9/18/91 DESCRIPTION: | GEOLOGIC UNIT | U.S.C.S. | Sample No. | Moisture (%) | Density (% Compaction) |
|---|---|---|---|---|---|---|
| | TERRACE DEPOSITS<br>A Light brown to dark orange-brown, damp to moist, dense, slightly clayey, fine- to medium-grained sand; upper .5 foot slightly dessicated and porous; some scattered root hairs; massive | Qt | SC | 1 @0-3' | | |

ENGINEERING PROPERTIES

GRAPHIC REPRESENTATION NW Wall SCALE: 1" = 5' SURFACE SLOPE: Horiz. TREND: N80E




Total depth = 3 feet
No ground water encountered
Backfilled: 9/18/91

# GEOTECHNICAL BORING LOG

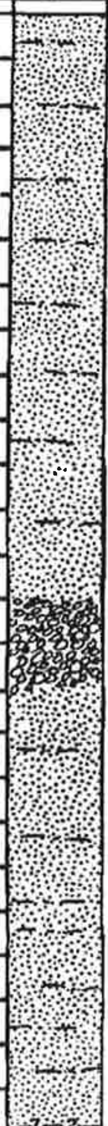
Date 9/24/91 Drill Hole No. B-4 Sheet 1 of 1

Project P-H/Sea Walk Village Job No. 4910693-01

Drilling Co. A&W Drill Rentals Type of Rig Rotary Wash

Hole Diameter 5" Drive Weight 340 lb. Drop 18 in.

Elevation Top of Hole ±53' Ref. or Datum mean sea level



| Depth Feet | Attitudes | Tube Sample No. | Blows Per Foot | Dry Density pcf | Moisture Content, % | Soil Class. (U.S.C.S.) | GEOTECHNICAL DESCRIPTION Logged by JB Sampled by JB |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | | PLEISTOCENE TERRACE DEPOSITS<br>Red-brown to brown, damp to moist, dense, silty, fine- to medium-grained sand |
| 5 | | | | | | | |
| 10 | | | | | | | |
| | | | | | | | @13': Very gravelly, slow drilling |
| 15 | | | | | | | @15.5' to 17': Hole caving |
| 20 | | No recovery | 54(?) | | | SM | @20': No recovery of sample; based on cuttings appears to be yellow-brown to brown, generally silty, fine to medium sand |
| | | | | | | GC | SAN ONOFRE BRECCIA (?)<br>@25': Light brown-orange, moist to damp, dense, slightly clayey, sandy gravel; gravel is fine- to coarse-grained, subrounded to angular granitic and metamorphic rock clasts |
| 25 | | 1 | 60+ | 106.5 | 9.9 | | |
| | | | | | | | Total depth = 25 feet<br>No apparent ground water<br>Refusal at 25 feet<br>Backfilled: 9/24/91 |
| 30 | | | | | | | |

# Boring Log

| | | | |
|---|---|---|---|
| Project | Lot C, Oceanside | Boring Number | 3 |
| Number | | Diameter | 8" |
| Date | 2/19/90 | Drive Weight | 140 lbs/ 30" drop |
| Geologist | SF | Surface Elevation | |

Location of Boring 225' west of curb, northeasterly portion of lot

| Depth (Feet) | Sample Number | Blows 6 inches | Graphic Log | Description |
|---|---|---|---|---|
| 0 | | | | TERRACE DEPOSITS |
| | | 22 | | 0' reddish-brown, silty, very fine- to fine-grained sand; loose to medium dense, slightly moist. |
| | | 30 | | 2' light brown, very silty, very fine- to fine-grained sand; medium dense to dense, slightly moist. |
| | | 38 | | 5' very dense, partially cemented. |
| 10 | | 12 | | 7.5' abundant subrounded to subangular gravels, scattered coarse-grained sand lenses. |
| | | 15 | | 14' light grayish-brown, poorly sorted sandy matrix containing abundant gravels. |
| | | 14 | | 15' cobbly. |
| | | 71 | | |

TOTAL DEPTH = 15.5' (refusal)
NO GROUNDWATER

Leighton

# FIELD PERCOLATION TEST DATA SHEET

| | | | |
|---|---|---|---|
| Project Name: | WP West Acq./ North Coast HWY Update Report | Project No.: | 12107.003 |
| Proj. Address: | 939 & 1009 N. Coast Highway, Oceanside California | | |

## SOIL TYPE / TEST LOCATION / BOREHOLE

Soil Type: Brown Silty Sand — Hole #: P-1

Location: P-1

Hole Dia: 8"

Depth 5'

Tested by: Pre-Saturation Date: 8.13.18 — Test Date: 8.14.18

Notes: Measurements in 1/100ths of feet (ft)

| Time of Day | Interval / Notes | Initial Depth to Water (ft) | Final Depth of Water (ft) | Δ in Water Level (ft) | Percolation Rate (min/inch) |
|---|---|---|---|---|---|
| 9:09 AM | Start | 1.74 | - | - | - |
| 9:37 AM | 28 | 1.74 | 1.76 | 0.02 | 116.67 |
| 10:05 AM | 28 | 1.76 | 1.78 | 0.02 | 116.67 |
| 10:35 AM | 30 | 1.78 | 1.80 | 0.02 | 125.00 |
| 11:05 AM | 30 | 1.80 | 1.83 | 0.03 | 83.33 |
| 11:35 AM | 30 | 1.83 | 1.84 | 0.01 | 250.00 |
| 12:08 PM | 33 | 1.84 | 1.85 | 0.01 | 275.00 |
| 12:35 PM | 27 | 1.85 | 1.87 | 0.02 | 112.50 |
| 1:03 PM | 28 | 1.87 | 1.89 | 0.02 | 116.67 |
| 1:41 PM | 38 | 1.89 | 1.90 | 0.01 | 316.67 |
| 2:20 PM | 39 | 1.90 | 1.91 | 0.01 | 325.00 |
| 2:50 PM | 30 | 1.91 | 1.92 | 0.01 | 250.00 |
| 3:20 PM | 30 | 1.92 | 1.93 | 0.01 | 250.00 |
| | | | | | |
| | | | | | |
| | | | | | |

Notes: Last reading used to determine percolation rate

Final Field Percolation Rate: 250 min/inch

Leighton

# FIELD PERCOLATION TEST DATA SHEET

| | | | |
|---|---|---|---|
| Project Name: | WP West Acq./ North Coast HWY Update Report | Project No.: | 12107.003 |
| Proj. Address: | 939 & 1009 N. Coast Highway, Oceanside California | | |

## SOIL TYPE / TEST LOCATION / BOREHOLE

Soil Type: Brown Silty Sand — Hole #: P-2

Location: P-2

Hole Dia: 8"

Depth 4.89

Tested by: Pre-Saturation Date: 8.13.18 — Test Date: 8.14.18

Notes: Measurements in 1/100ths of feet (ft)

| Time of Day | Interval / Notes | Initial Depth to Water (ft) | Final Depth of Water (ft) | Δ in Water Level (ft) | Percolation Rate (min/inch) |
|---|---|---|---|---|---|
| 9:25 AM | Start | 1.48 | - | - | - |
| 9:46 AM | 21 | 1.48 | 1.50 | 0.02 | 87.50 |
| 10:08 AM | 22 | 1.50 | 1.53 | 0.03 | 61.11 |
| 10:38 AM | 30 | 1.53 | 1.56 | 0.03 | 83.33 |
| 11:08 AM | 30 | 1.56 | 1.59 | 0.03 | 83.33 |
| 11:39 AM | 31 | 1.59 | 1.61 | 0.02 | 129.17 |
| 12:10 PM | 31 | 1.61 | 1.62 | 0.01 | 258.33 |
| 12:39 PM | 29 | 1.62 | 1.63 | 0.01 | 241.67 |
| 1:08 PM | 29 | 1.63 | 1.65 | 0.02 | 120.83 |
| 1:43 AM | 35 | 1.65 | 1.66 | 0.01 | 291.67 |
| 2:18 PM | 35 | 1.66 | 1.67 | 0.01 | 291.67 |
| 2:50 AM | 32 | 1.67 | 1.68 | 0.01 | 266.67 |
| 3:20 PM | 30 | 1.68 | 1.69 | 0.01 | 250.00 |
| | | | | | |
| | | | | | |
| | | | | | |

Notes: Final reading used to determine percolation rate

Final Field Percolation Rate: 250 min/inch

Leighton

# FIELD PERCOLATION TEST DATA SHEET

| | | | |
|---|---|---|---|
| Project Name: | WP West Acq./ North Coast HWY Update Report | Project No.: | 12107.003 |
| Proj. Address: | 939 & 1009 N. Coast Highway, Oceanside California | | |

## SOIL TYPE / TEST LOCATION / BOREHOLE

Soil Type: Brown Silty Sand Hole #: P-3

Location: P-3

Hole Dia: 8"

Depth 4.54

Tested by: Pre-Saturation Date: 8.13.18 Test Date: 8.14.18

Notes: Measurements in 1/100ths of feet (ft)

| Time of Day | Interval / Notes | Initial Depth to Water (ft) | Final Depth of Water (ft) | Δ in Water Level (ft) | Percolation Rate (min/inch) |
|---|---|---|---|---|---|
| 9:16 AM | Start | 2.09 | - | - | - |
| 9:41 AM | 25 | 2.09 | 2.1 | 0.02 | 104.17 |
| 10:05 AM | 24 | 2.1 | 2.11 | 0.01 | 200.00 |
| 10:35 AM | 30 | 2.11 | 2.11 | 0 | NP |
| 11:06 AM | 31 | 2.11 | 2.12 | 0.01 | 258.33 |
| 11:37 AM | 31 | 2.12 | 2.12 | 0 | NP |
| 12:08 PM | 31 | 2.12 | 2.12 | 0 | NP |
| 12:37 PM | 29 | 2.12 | 2.12 | 0 | NP |
| 1:05 PM | 28 | 2.12 | 2.13 | 0.01 | 233.33 |
| 1:43 PM | 38 | 2.13 | 2.13 | 0 | NP |
| 2:18 PM | 35 | 2.13 | 2.14 | 0.01 | 291.67 |
| 2:44 PM | 26 | 2.14 | 2.14 | 0 | NP |
| 3:15 PM | 31 | 2.14 | 2.15 | 0.01 | 258.33 |
| | | | | | |
| | | | | | |
| | | | | | |

Notes: Final reading used to determind percolation rate

Final Field Percolation Rate: 258 min/inch

NP = No Percolation Rate

Leighton

# FIELD PERCOLATION TEST DATA SHEET

| | | | |
|---|---|---|---|
| Project Name: | WP West Acq./ North Coast HWY Update Report | Project No.: | 12107.003 |
| Proj. Address: | 939 & 1009 N. Coast Highway, Oceanside California | | |

## SOIL TYPE / TEST LOCATION / BOREHOLE

Soil Type: Brown Silty Sand    Hole #: P-4

Location: P-4

Hole Dia: 8"

Depth 4.78

Tested by:    Pre-Saturation Date: 8.13.18    Test Date: 8.14.18

Notes: Measurements in 1/100ths of feet (ft)

| Time of Day | Interval / Notes | Initial Depth to Water (ft) | Final Depth of Water (ft) | Δ in Water Level (ft) | Percolation Rate (min/inch) |
|---|---|---|---|---|---|
| 8:56 AM | Start | 1.81 | - | - | - |
| 9:28 AM | 32 | 1.81 | 1.84 | 0.03 | 88.89 |
| 10:00 AM | 32 | 1.84 | 1.87 | 0.03 | 88.89 |
| 10:30 AM | 30 | 1.87 | 1.90 | 0.03 | 83.33 |
| 11:00 AM | 30 | 1.90 | 1.94 | 0.04 | 62.50 |
| 11:32 AM | 32 | 1.94 | 1.97 | 0.03 | 88.89 |
| 12:05 PM | 33 | 1.97 | 2.00 | 0.03 | 91.67 |
| 12:27 PM | 22 | 2.00 | 2.02 | 0.02 | 91.67 |
| 1:00 PM | 33 | 2.02 | 2.05 | 0.03 | 91.67 |
| 1:37 PM | 37 | 2.05 | 2.07 | 0.02 | 154.17 |
| 2:14 PM | 37 | 2.07 | 2.10 | 0.03 | 102.78 |
| 2:44 PM | 30 | 2.10 | 2.12 | 0.02 | 125.00 |
| 3:15 PM | 31 | 2.12 | 2.14 | 0.02 | 129.17 |
| | | | | | |
| | | | | | |
| | | | | | |

Notes: Final reading used to determine percolation rate

Final Field Percolation Rate: 129 min/inch

# APPENDIX C

# SUMMARY OF LABORATORY TESTING

# APPENDIX C

## Laboratory Testing Procedures and Test Results

Moisture and Density Determination Tests: Moisture content and dry density determinations were performed on relatively undisturbed samples obtained from the test borings. The results of these tests are presented in the boring logs. Where applicable, only moisture content was determined from "undisturbed" or disturbed samples.

Direct Shear Test: One direct shear test was performed on selected relatively undisturbed sample which was soaked for a minimum of 24 hours under a surcharge equal to the applied normal force during testing. After transfer of the sample to the shear box and reloading of the sample, the pore pressures set up in the sample (due to the transfer) were allowed to dissipate for a period of approximately 1 hour prior to application of shearing force. The samples were tested under various normal loads utilizing a motor-driven, strain-controlled, direct-shear testing apparatus at a strain rate of 0.05 inches per minute. The test results are presented on the attached figure.

Expansion Index Test: The expansion potential of selected material was evaluated by the Expansion Index Text, ASTM Test Method 4829. The specimen was molded under a given compactive energy to approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimen was loaded to an equivalent 144 psf surcharge and was inundated with water until volumetric equilibrium was reached. The result of this test is presented in the table below:

| Sample Location | Sample Description | Expansion Index | Expansion Potential |
|---|---|---|---|
| HSA-1 @ 0 to 5 feet | Silty SAND (SM) | <1 | Very Low |

APPENDIX C (Continued)

Minimum Resistivity and pH Tests: Minimum resistivity and pH tests were performed in general accordance with Caltrans Test Method CT643 for Steel or CT532 for concrete and standard geochemical methods. The results are presented in the table below:

| Sample Location | Sample Description | pH | Minimum Resistivity (ohms-cm) |
|---|---|---|---|
| HSA-1 @ 0-5’ | Silty SAND (SM) | 6.83 | 3,075 |

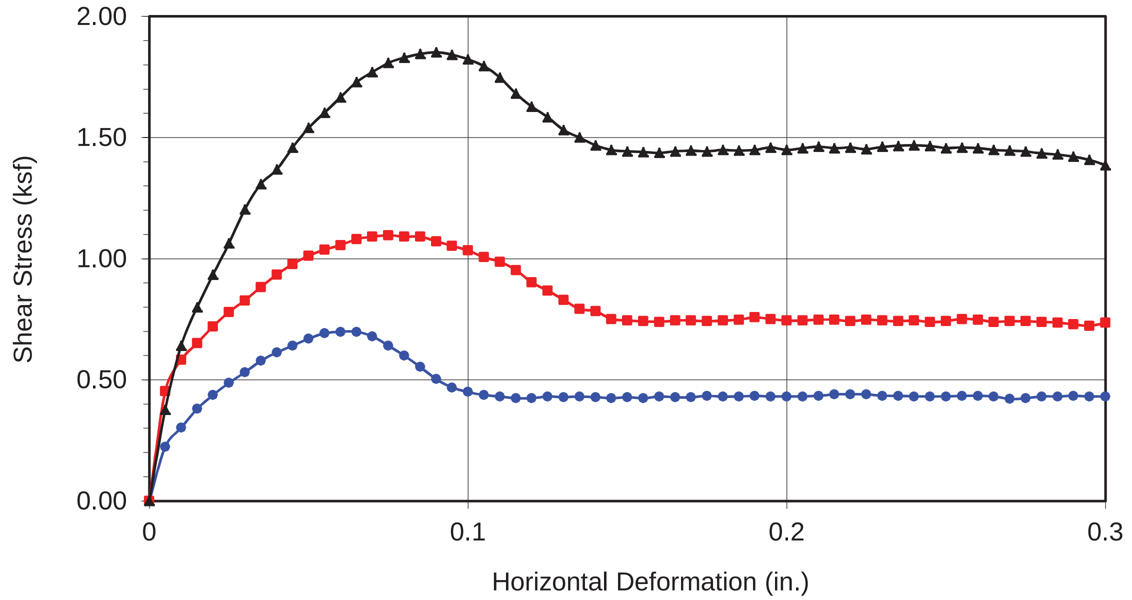
Chloride Content: Chloride content was tested in accordance with Caltrans Test Method CT422. The results are presented below:

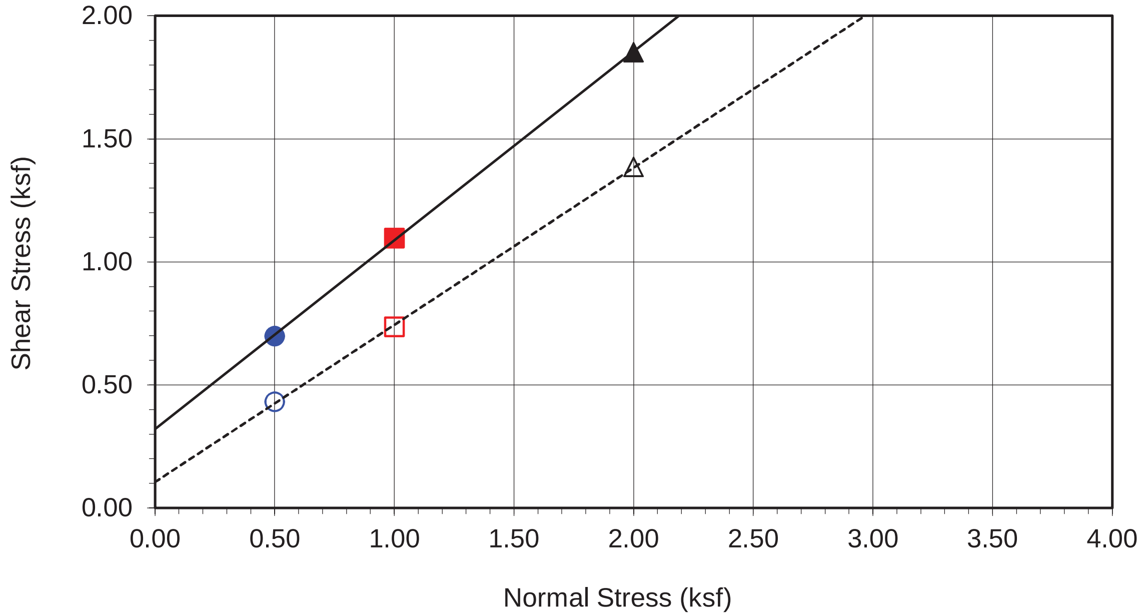
| Sample Location | Sample Description | Chloride Content, ppm |
|---|---|---|
| HSA-1 @ 0-5’ | Silty SAND (SM) | 200 |

Soluble Sulfates: The soluble sulfate contents of selected samples were determined by standard geochemical methods (Caltrans Test Method CT417). The test results are presented in the table below:

| Sample Location | Sample Description | Sulfate Content (%) | Potential Degree of Sulfate Attack* |
|---|---|---|---|
| HSA-1 @ 0-5’ | Silty SAND (SM) | <0.015 | Negligible |

* Based on the 2011 edition of American Concrete Institute (ACI) Committee 318R, Table No. 4.2.1.




| | |
|---|---|
| **Boring No.** | **HA-1** |
| **Sample No.** | **R-1** |
| **Depth (ft)** | **3-4** |
| Sample Type: | Ring |

Soil Identification:
Yellowish brown silty sand (SM)

**Strength Parameters**

| | C (psf) | ϕ (°) |
|---|---|---|
| Peak | 321 | 37 |
| Ultimate | 106 | 33 |

| | | | |
|---|---|---|---|
| Normal Stress (kip/ft²) | 0.500 | 1.000 | 2.000 |
| Peak Shear Stress (kip/ft²) | ● 0.698 | ■ 1.097 | ▲ 1.852 |
| Shear Stress @ End of Test (ksf) | ○ 0.431 | □ 0.736 | △ 1.386 |
| Deformation Rate (in./min.) | 0.0500 | 0.0500 | 0.0500 |
| Initial Sample Height (in.) | 1.000 | 1.000 | 1.000 |
| Diameter (in.) | 2.415 | 2.415 | 2.415 |
| Initial Moisture Content (%) | 2.37 | 2.37 | 2.37 |
| Dry Density (pcf) | 122.9 | 123.6 | 124.5 |
| Saturation (%) | 17.2 | 17.6 | 18.1 |
| Soil Height Before Shearing (in.) | 0.9809 | 0.9901 | 0.9578 |
| Final Moisture Content (%) | 12.0 | 11.4 | 10.9 |

Leighton

**DIRECT SHEAR TEST RESULTS**
**Consolidated Undrained**

Project No.: 12107.003

939 & 1009 WP NC Hwy

08-18

*DS HA-1, R-1 @ 3-4*

# APPENDIX D

# CITY OF OCEANSIDE

# WORKSHEET I-8

| Categorization of Infiltration Feasibility Condition | | | Form I-8 |
|---|---|---|---|
| **Part 1 - Full Infiltration Feasibility Screening Criteria**<br>**Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?** | | | |
| Criteria | Screening Question | Yes | No |
| 1 | **Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D. | | X |
| Provide basis:<br><br>Based on our field percolation testing, the infiltration rates of the soils at the subject site are less than 0.5 inches per hour. The calculated infiltration rates via the Porchet Method with an applied safety factor of 2 range from 0.006 inches per hour to 0.008 inches per hour throughout the site.<br><br>Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability. | | | |
| 2 | **Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2. | X | |
| Provide basis:<br><br>If the infiltration rates were greater than 0.5 inches per hour, it may be possible that the risk of geotechnical hazards would not be increased provided mitigation is performed for any underground utilities/structures, slopes (i.e., setbacks) within the vicinity of the proposed infiltration site.<br><br>Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability. | | | |

<table>
<tr><th colspan="4">Form I-8 Page 2 of 4</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>3</td><td>Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:<br>If the infiltration rates were greater than 0.5 inches per hour, it may be possible that the risk of groundwater contamination would not be increased provided there are no contaminated soil or groundwater sites within 250 feet of the proposed infiltration site. In addition, the previous explorations indicate the groundwater is greater than 50 feet bgs.<br><br>Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>4</td><td>Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:<br>If the infiltration rates were greater than 0.5 inches per hour, it may be possible that potential water balance issues would not be affected provided there are no unlined site drainages/creeks/streams within 250 feet of the proposed infiltration site.<br><br>Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>Part 1 Result *</td><td colspan="2">If all answers to rows 1 - 4 are “Yes” a full infiltration design is potentially feasible. The feasibility screening category is Full Infiltration<br><br>If any answer from row 1-4 is “No”, infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a “full infiltration” design. Proceed to Part 2</td><td>Go to Part 2</td></tr>
</table>

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by the City to substantiate findings

## Form I-8 Page 3 of 4

**Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria**

**Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?**

| Criteria | Screening Question | Yes | No |
|---|---|---|---|
| 5 | **Do soil and geologic conditions allow for infiltration in any appreciable rate or volume?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D. | | X |

Provide basis:

Based on our field percolation testing, the calculated infiltration rates of the soils at the subject site are between 0.006 to 0.008 inches per hour, which is currently assumed to be a "Non infiltration" condition. Note that the calculated infiltration rate via the Porchet Method has an applied safety factor of 2.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

| Criteria | Screening Question | Yes | No |
|---|---|---|---|
| 6 | **Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level**? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2. | X | |

Provide basis:

For a "Non infiltration" condition (between 0.006 to 0.008 inches per hour), the risk of geotechnical hazards will not be increased by non infiltration provided mitigation is performed for any underground utilities/structures, slopes (i.e., setbacks) within the vicinity of the proposed infiltration site. Mitigation can include subsurface vertical barriers and subdrains to limit groundwater migration and perched groundwater conditions.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

<table>
<tr><th colspan="4">Form I-8 Page 4 of 4</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>7</td><td>Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:<br>For a "Non infiltration" condition (between 0.006 to 0.008 inches per hour), the risk of groundwater contamination will not be increased by partial infiltration provided there are no contaminated soil or groundwater sites within 250 feet of the proposed infiltration site. In addition, the previous explorations indicate the groundwater is greater than 50 feet below ground surface.<br><br>Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</td></tr>
<tr><td>8</td><td>Can infiltration be allowed without violating downstream water rights? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:<br>For a "Non infiltration" condition (between 0.006 to 0.008 inches per hour), violation of downstream water rights is not anticipated based on the site location and that there are no unlined site drainages/creeks/streams within 250 feet of the proposed infiltration site.<br><br>Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</td></tr>
<tr><td>Part 2 Result*</td><td colspan="2">If all answers from row 5-8 are yes then partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration.<br><br>If any answer from row 5-8 is no, then infiltration of any volume is considered to be infeasible within the drainage area. The feasibility screening category is No Infiltration.</td><td>Non Infiltration</td></tr>
</table>

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by the City to substantiate findings

# APPENDIX E

# GENERAL EARTHWORK AND GRADING SPECIFICATIONS

# FOR ROUGH GRADING

## 1.0 General

### 1.1 Intent

These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

### 1.2 The Geotechnical Consultant of Record

Prior to commencement of work, the owner shall employ the Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultants shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

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

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

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

## 2.0 Preparation of Areas to be Filled

### 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 the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. 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.

2.2 Processing

Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. 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.

2.3 Overexcavation

In addition to removals and overexcavations 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 overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical

Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

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 the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

3.1 General

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant 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 the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 Oversize

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. 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 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import

If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

## 4.0 Fill Placement and Compaction

### 4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

### 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).

### 4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557). 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.

### 4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557.

### 4.5 Compaction Testing

Field-tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant'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).

4.6 Frequency of Compaction Testing

Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations

The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

## 5.0 Subdrain Installation

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

## 6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant 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, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

## 7.0 Trench Backfills

### 7.1 Safety

The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations.

### 7.2 Bedding and Backfill

All bedding and backfill of utility trenches shall be performed in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified. Backfill shall be placed and densified to a minimum of 90 percent of relative compaction from 1 foot above the top of the conduit to the surface.

The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

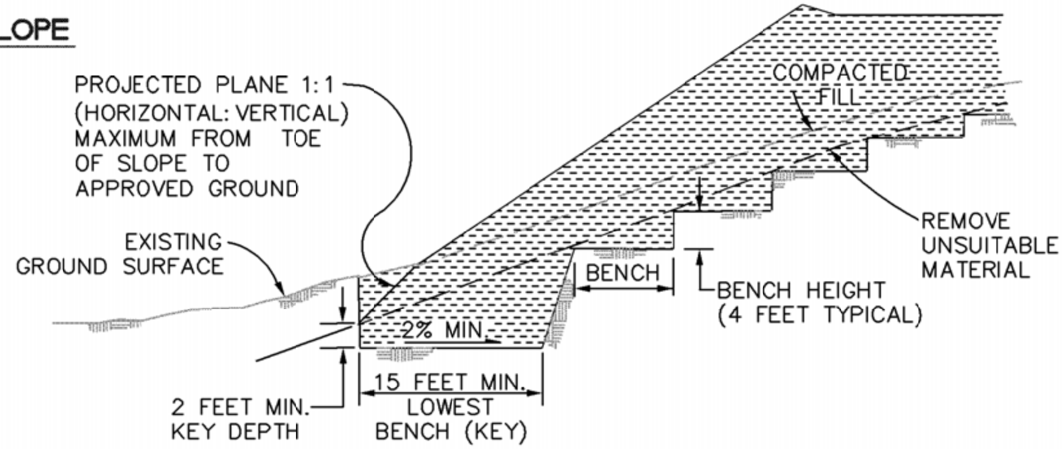
### 7.3 Lift Thickness

Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

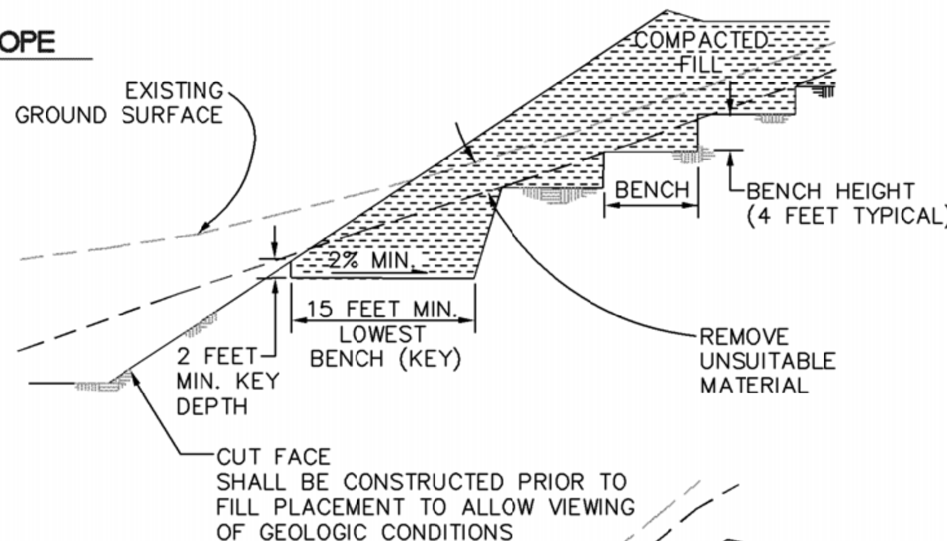
### 7.4 Observation and Testing

The densification of the bedding around the conduits shall be observed by the Geotechnical Consultant.

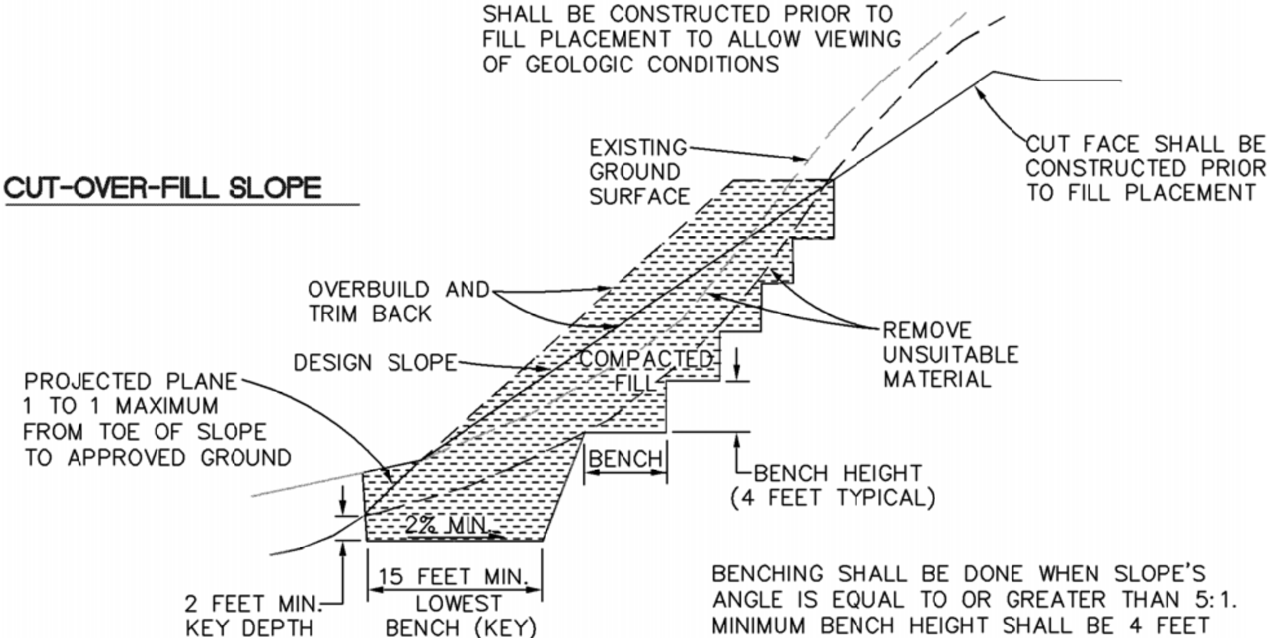
## FILL SLOPE




## FILL-OVER-CUT SLOPE



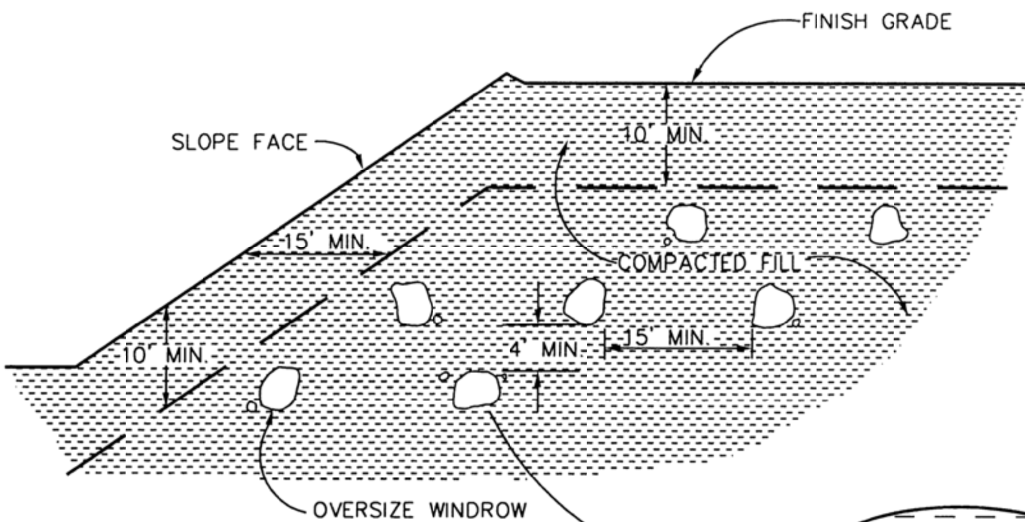

## CUT-OVER-FILL SLOPE

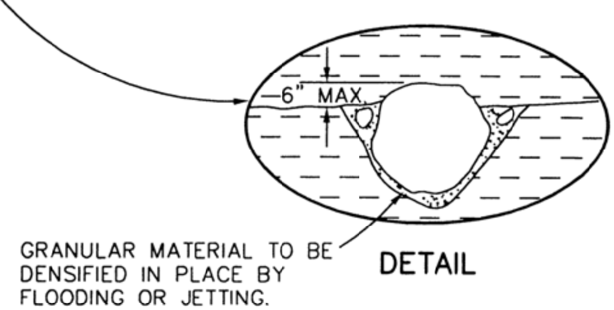



BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

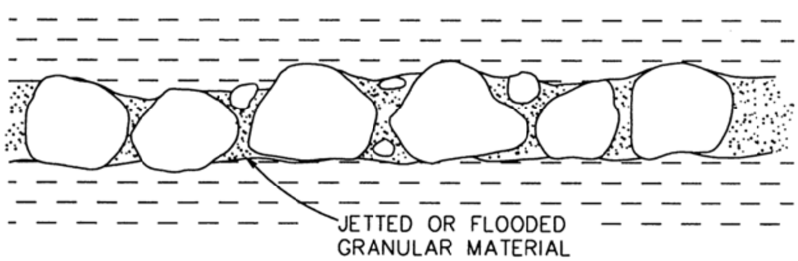
# KEYING AND BENCHING

GENERAL EARTHWORK AND GRADING SPECIFICATIONS
STANDARD DETAIL A



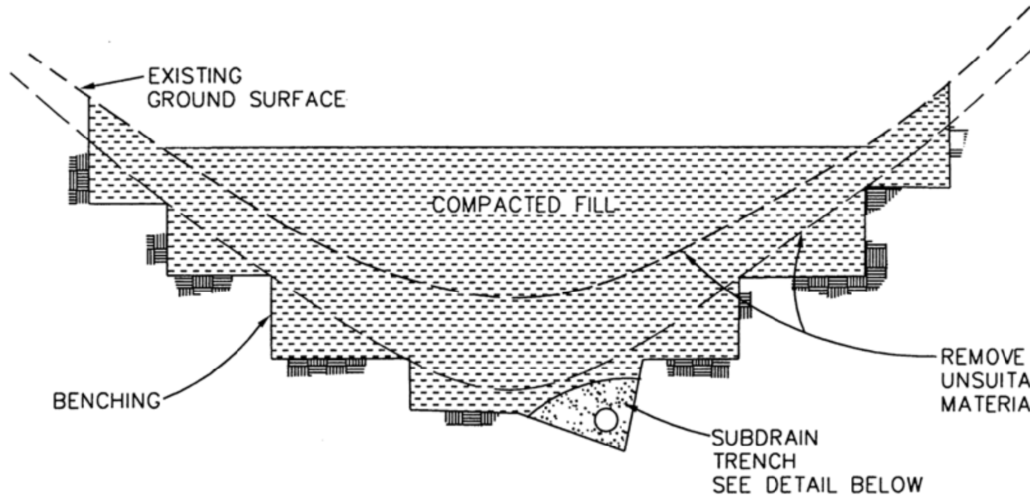

* OVERSIZE ROCK IS LARGER THAN 8 INCHES IN LARGEST DIMENSION.
* EXCAVATE A TRENCH IN THE COMPACTED FILL DEEP ENOUGH TO BURY ALL THE ROCK.
* BACKFILL WITH GRANULAR SOIL JETTED OR FLOODED IN PLACE TO FILL ALL THE VOIDS.
* DO NOT BURY ROCK WITHIN 10 FEET OF FINISH GRADE.
* WINDROW OF BURIED ROCK SHALL BE PARALLEL TO THE FINISHED SLOPE.

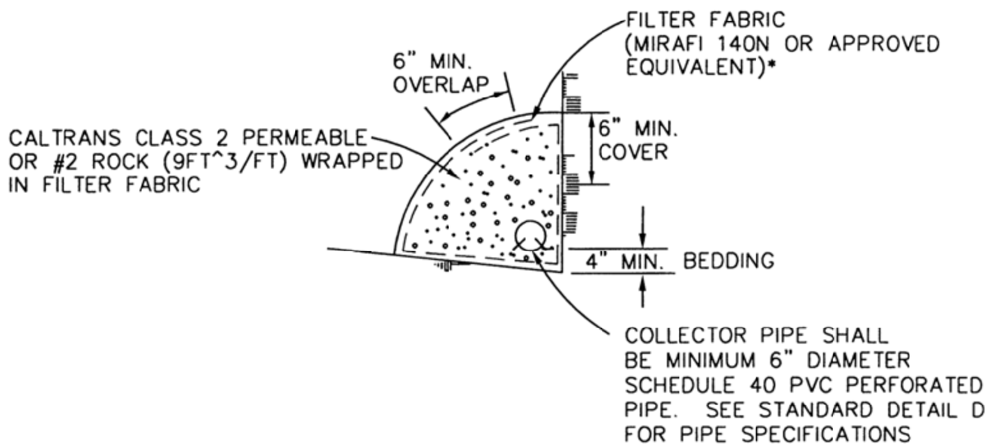



TYPICAL PROFILE ALONG WINDROW

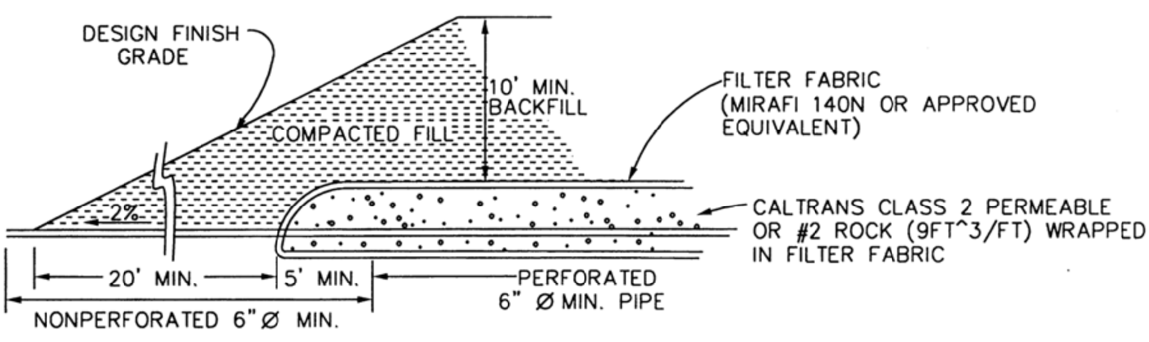
# OVERSIZE ROCK DISPOSAL

GENERAL EARTHWORK AND GRADING SPECIFICATIONS
STANDARD DETAIL B



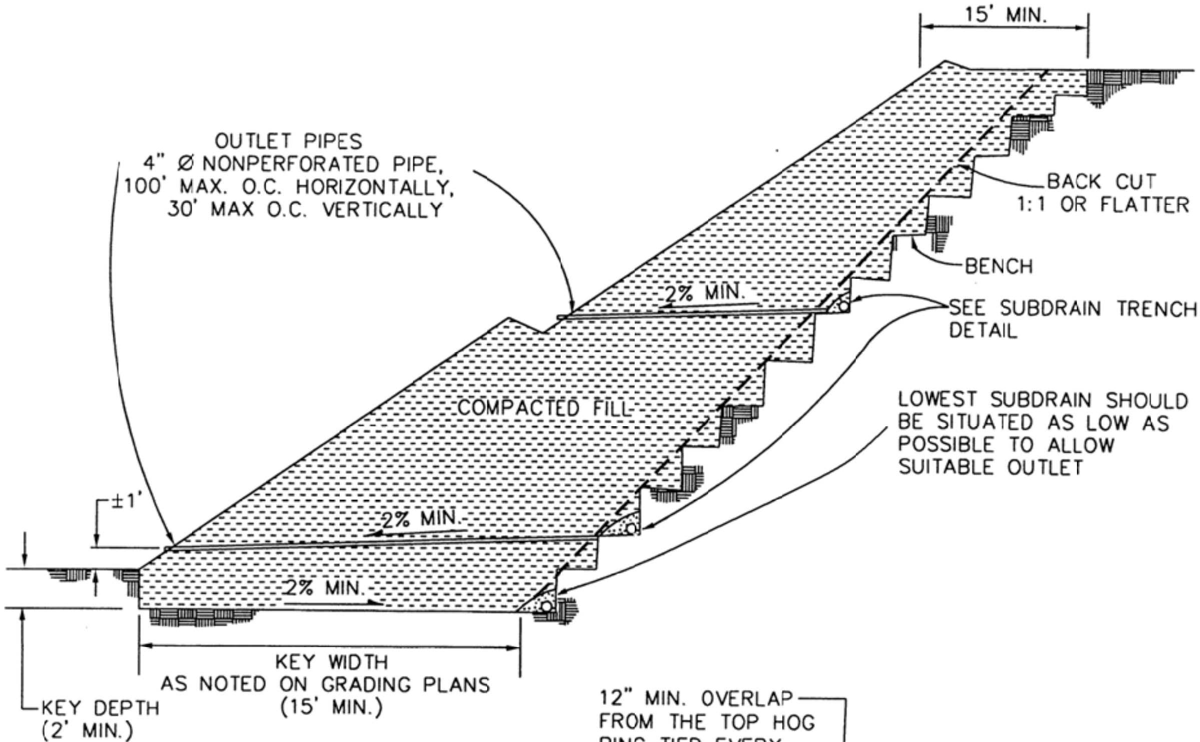



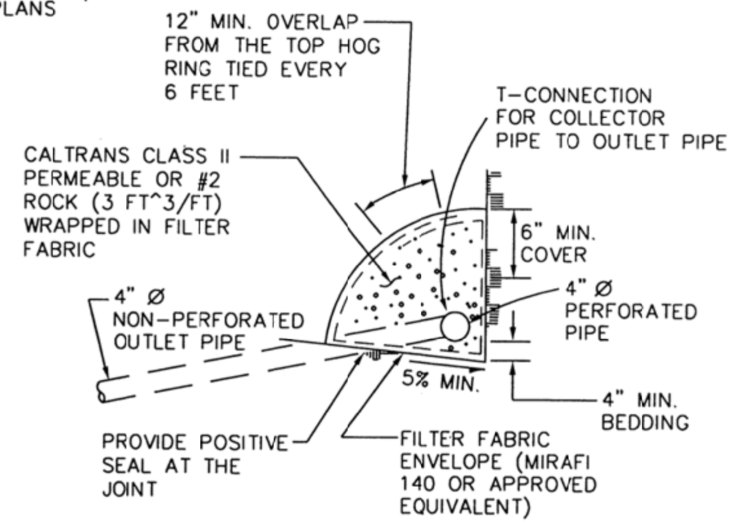

**SUBDRAIN DETAIL**




**DETAIL OF CANYON SUBDRAIN OUTLET**

| | |
|---|---|
| CANYON SUBDRAINS | GENERAL EARTHWORK AND<br>GRADING SPECIFICATIONS<br>STANDARD DETAIL C |



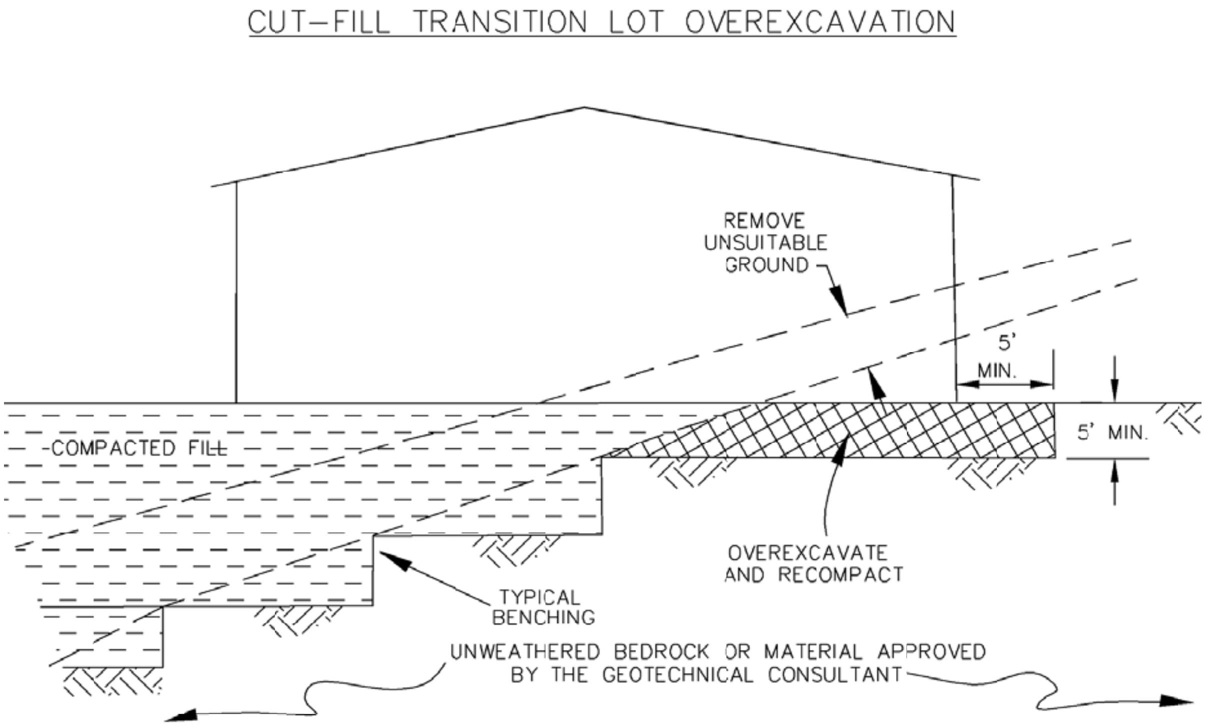

**SUBDRAIN TRENCH DETAIL**

SUBDRAIN INSTALLATION – subdrain collector pipe shall be installed with perforation down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" to 1/2" if drill holes are used. All subdrain pipes shall have a gradient of at least 2% towards the outlet.

SUBDRAIN PIPE – Subdrain pipe shall be ASTM D2751, SDR 23.5 or ASTM D1527, Schedule 40, or ASTM D3034, SDR 23.5, Schedule 40 Polyvinyl Chloride Plastic (PVC) pipe.

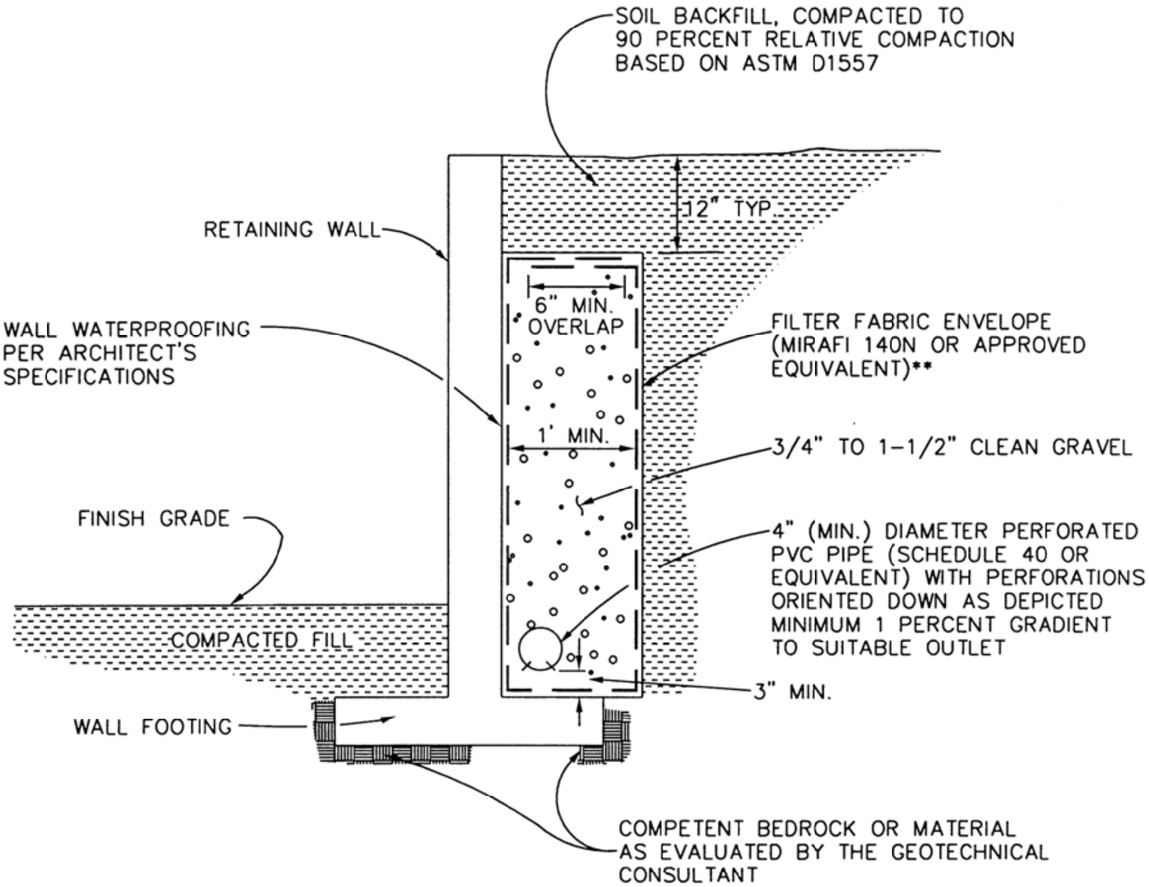
All outlet pipe shall be placed in a trench no wider than twice the subdrain pipe.

| BUTTRESS OR REPLACEMENT FILL SUBDRAINS | GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL D |
|---|---|



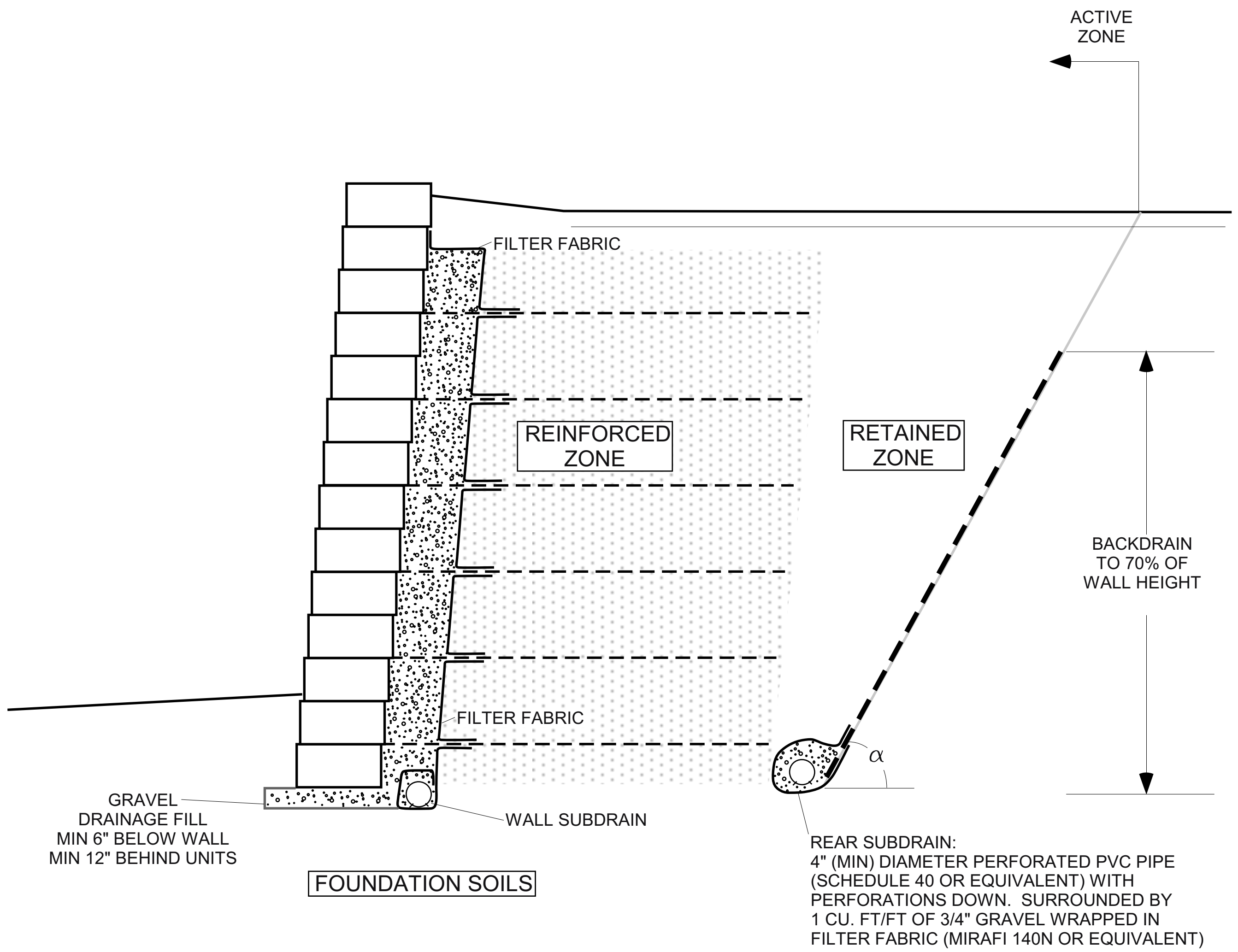

# TRANSITION LOT FILLS

GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL E




NOTE: UPON REVIEW BY THE GEOTECHNICAL CONSULTANT, COMPOSITE DRAINAGE PRODUCTS SUCH AS MIRADRAIN OR J-DRAIN MAY BE USED AS AN ALTERNATIVE TO GRAVEL OR CLASS 2 PERMEABLE MATERIAL. INSTALLATION SHOULD BE PERFORMED IN ACCORDANCE WITH MANUFACTURER'S SPECIFICATIONS.

| | |
|---|---|
| RETAINING WALL DRAINAGE | GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL F |




NOTES:

1) MATERIAL GRADATION AND PLASTICITY

REINFORCED ZONE:

| SIEVE SIZE | % PASSING |
|---|---|
| 1 INCH | 100 |
| NO. 4 | 20-100 |
| NO. 40 | 0-60 |
| NO. 200 | 0-35 |

FOR WALL HEIGHT < 10 FEET, PLASTICITY INDEX < 20
FOR WALL HEIGHT 10 TO 20 FEET, PLASTICITY INDEX < 10
FOR TIERED WALLS, USE COMBINED WALL HEIGHTS
WALL DESIGNER TO REQUEST SITE-SPECIFIC CRITERIA FOR WALL HEIGHT > 20 FEET

GRAVEL DRAINAGE FILL:

| SIEVE SIZE | % PASSING |
|---|---|
| 1 INCH | 100 |
| 3/4 INCH | 75-100 |
| NO. 4 | 0-60 |
| NO. 40 | 0-50 |
| NO. 200 | 0-5 |

2) CONTRACTOR TO USE SOILS WITHIN THE RETAINED AND REINFORCED ZONES THAT MEET THE STRENGTH REQUIREMENTS OF WALL DESIGN.

3) GEOGRID REINFORCEMENT TO BE DESIGNED BY WALL DESIGNER CONSIDERING INTERNAL, EXTERNAL, AND COMPOUND STABILITY.

3) GEOGRID TO BE PRETENSIONED DURING INSTALLATION.

4) IMPROVEMENTS WITHIN THE ACTIVE ZONE ARE SUSCEPTIBLE TO POST-CONSTRUCTION SETTLEMENT. ANGLE $\alpha = 45 + \phi/2$, WHERE $\phi$ IS THE FRICTION ANGLE OF THE MATERIAL IN THE RETAINED ZONE.

5) BACKDRAIN SHOULD CONSIST OF J-DRAIN 302 (OR EQUIVALENT) OR 6-INCH THICK DRAINAGE FILL WRAPPED IN FILTER FABRIC. PERCENT COVERAGE OF BACKDRAIN TO BE PER GEOTECHNICAL REVIEW.

# SEGMENTAL RETAINING WALLS

GENERAL EARTHWORK AND GRADING SPECIFICATIONS
STANDARD DETAIL G

# APPENDIX F

# GBA

# GEOTECHNICAL 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.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

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