APPENDIX F

Geotechnical Investigations

ADDENDUM - UPDATED
SEISMIC RECOMMENDATIONS
WAKELAND 4TH CORNER RESIDENTIAL
4390 UNIVERSITY AVENUE
SAN DIEGO, CALIFORNIA

Prepared for:

Wakeland Housing & Development Corporation, Inc.
1230 Columbia Street
San Diego California 92101

Project No. 11534.003

August 28, 2020

Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

August 28, 2020

Project No. 11534.003

Wakeland Housing & Development Corporation, Inc.
1230 Columbia Street
San Diego, California 92101

Attention: Ms. Kim Duran

Subject: Addendum - Updated Seismic Recommendations, Wakeland 4th Corner Residential Project, San Diego, California

In accordance with your request, we have prepared this addendum letter to provide supplemental recommendations to our original geotechnical report (Leighton, 2017) to address the seismic design and to provide updated infiltration letter (Leighton, 2020) for the proposed new building. The updated infiltration letter is included in Appendix B of this report. The original geotechnical investigation is included in Appendix C of this report. Specifically, this addendum letter has been prepared to present updated Mapped Spectral Acceleration Parameters in accordance with the 2019 California Building Code. Below is the updated Sections 4.4.3 and 4.4.4.

4.4.3 Site Class

A shear wave survey was recently performed to measure soil velocity using Microtremor Analytical Methods (MAM). The survey extrapolated the V_{s30} for the upper 100 feet with a value of 1,410 feet per second. Based on this survey and on our experience with similar sites in the project area we have characterized the site soil profile to be a Site Class C utilizing 2019 California Building Code (CBC) procedures. A copy of the Seismic Survey is provided in Appendix D.

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4.4.4 Building Code Mapped Spectral Acceleration Parameters

The effect of seismic shaking may be mitigated by adhering to the California Building Code and state-of-the-art seismic design practices of the Structural Engineers Association of California. Provided below in Table 1 are the spectral acceleration parameters for the project determined in accordance with the 2019 CBC (CBSC, 2019) and the SEA/OSHPD Web Application.

Table 1

2019 CBC Mapped Spectral Acceleration Parameters

Site Class	C		
Site Coefficients	F_a	=	1.2
	F_v	=	1.5
Mapped MCE Spectral Accelerations	S_S	=	1.067 g
	S_1	=	0.369g
Site Modified MCE Spectral Accelerations	S_{MS}	=	1.281g
	S_{M1}	=	0.554g
Design Spectral Accelerations	S_{DS}	=	0.854g
	S_{D1}	=	0.369g

Utilizing ASCE Standard 7-16, in accordance with Sections 11.8.2 and 11.8.3, the following additional parameters for the peak horizontal ground acceleration are associated with the Geometric Mean Maximum Considered Earthquake (MCE_G). The mapped MCE_G peak ground acceleration (PGA) is 0.473g for the site. For a Site Class C, the F_{pga} is 1.2 and the mapped peak ground acceleration adjusted for Site Class effects (PGA_m) is 0.567g for the site.

Since the mapped spectral response at 1-second period is less than 0.75g, then all structures subject to the criteria in Section 1613A.2.5 of the 2019 CBC are assigned Seismic Design Category D.

If you have any questions regarding this letter, please contact this office. We appreciate this opportunity to be of service.

Respectfully submitted,

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<u>Appendices</u>

Appendix A - References
Appendix B - Updated Infiltration Letter
Appendix C - Preliminary Geotechnical Investigation (Leighton and Associates, 2017)
Appendix D - Seismic Survey

Distribution: (1) Addressee, via email

Appendix A

References

APPENDIX A

REFERENCES

Applied Technology Council (ATC), 2020, Hazards by Locations Seismic Online Tool https://hazards.atcouncil.org/

California Building Standards Commission (CBSC), 2019, California Building Code, Volumes 1 and 2.

Leighton and Associates, 2020, Updated Infiltration Feasibility Letter, Fourth Corner Residential Project, San Diego, California 92101, Project No 11534.003, dated April 15, 2020.

_____________________, 2017, Preliminary Geotechnical Investigation, Fourth Corner Residential Project, Fairmount Avenue, San Diego, California 92101, Project No 11534.001, dated February 6, 2017.

Appendix B

Infiltration Feasibility Letter
(Leighton and Associates, 2020)

Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

June 5, 2020

Project No. 11534.003

To: Wakeland Housing and Development, Inc.
1230 Columbia Street
San Diego, CA 92101

Attention: Ms. Dani McMillin

Subject: Infiltration Feasibility Letter, Fourth Corner Residential Project, San Diego, California

As requested, we have prepared this letter to discuss the infiltration feasibility at the project site. Therefore, in general accordance with Section C.1.1 Infiltration Feasibility Condition Letter of the San Diego Storm Water Standards (City of San Diego, 2018), Leighton has prepared this summary letter discussing infiltration feasibility at the site. Items associated with C.1.1 of the City BMP Design Manual are included in italics and summarized below:

The phase of the project in which the geotechnical engineer first analyzed the site for infiltration feasibility.

The site was first analyzed for infiltration feasibility during the field investigation for the geotechnical report dated February 6, 2017 (Appendix A). At that time the site was not considered feasible for storm water infiltration.

Results of previous geotechnical analyses conducted in the project area, if any.

The results of the project geotechnical investigation, referenced in Appendix A, indicate that the site is underlain by undocumented fill soils (approximately 2 feet thick) apparently placed during the initial site development, were observed in our exploration locations across the site. Localized deeper unknown fills associated with past development may exist across the site. As encountered during our explorations, the fill soils were observed to generally consist of dark brown, moist, soft, high plasticity, sandy lean clay with variable amounts of gravel and cobble and light brownish gray, moist, loose to medium dense, silty sand. As observed in the off-site boring B-1 performed at

4089 Fairmount Avenue, we encountered undocumented fill to a depth of approximately 7 feet thick. The fill materials consist of light brownish gray, moist, loose to medium dense, silty sand. Pliocene-aged Normal Heights Mudstone was encountered underlying the undocumented fill and extended to depths of approximately 6 to 7 feet bgs at the subject site. The Normal Heights Mudstone, which caps the mesa, is generally composed of poorly consolidated claystone that is characteristically steel gray in color and highly cohesive. Where observed in our exploration, the Normal Heights Mudstone consists of very dark gray, moist, firm to very stiff, high plasticity, claystone with interbedded layers of gravel and cobble. Late Pleistocene-aged Very Old Paralic Deposits underlie the entire site. As encountered, these deposits consist primarily of light yellowish brown to yellowish brown, dense to very dense, moist, fine-grained, oxidized, clayey sandstone with gravel with interbedded layers of cobble conglomerate.

The development status of the site prior to the project application (i.e., new development with raw ungraded land, or redevelopment with existing graded conditions).

This is a redevelopment-type project. The subject site is a rectangular shaped parcel of land. Specifically, the proposed residential development will be located at 4021, 4029, 4035, and 4061 Fairmount Avenue in a previously developed area known as City Heights in the City of San Diego, California. The property at 4089 Fairmont Avenue was explored; however, it is our understanding that this property is not currently being proposed for redevelopment at this time. In general, the site is bounded by Fairmount Avenue to the west, an alleyway to the east and existing commercial developments to the north and south. Overall dimensions of the subject site are approximately 130 by 240 lineal feet. The site is currently occupied by asphalt paved parking lots, a two-story commercial building (i.e., United Women of East Africa Organization) and areas that are used for urban gardening. Other site improvements consist of underground utilities, concrete hardscaping, and perimeter security fences. Site topography is nearly level with surface elevations ranging from approximately 366 to 364 feet above mean sea level (msl) (i.e., drainage from the west to the east). The site was developed prior to the 1950's. There are no areas of exposed surface soils across the site where water infiltration might occur.

The history of design discussions for the project footprint, resulting in the final design determination.

Leighton was not involved in design discussions related to project footprint and final design determination. However, the footprint of the proposed building is a property line to property line footprint covering generally the central and northern portions of the City block (Figure 2).

Full/partial infiltration BMP standard setbacks to underground utilities, structures, retaining walls, fill slopes, and natural slopes applicable to the DMA that prevent full/partial infiltration.

Numerous existing underground utilities are located within 10 feet of the site within City of San Diego Right-of-Way. These utilities include settlement sensitive wet utilities such as storm drain and sewer lines. In addition, several dry utilities are located within 10 feet of the site which will be adversely impacted by infiltration of storm water.

The physical impairments (i.e., fire road egress, public safety considerations, etc.) that prevent full/partial infiltration.

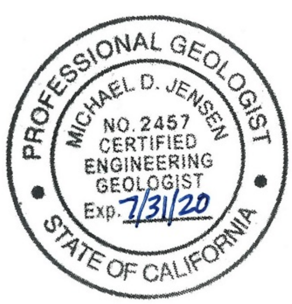
Physical impairments that prevent infiltration were not observed at the site.

Conclusion or recommendation from the geotechnical engineer regarding the DMA's infiltration condition.

As previously mentioned above, the site is underlain by approximately 2 feet of undocumented fill which in turn is underlain by Normal Heights Mudstone. BMPs located in these soil units can be problematic and may induce adverse soil movement. In addition, numerous existing underground utilities are located within 10 feet of the site. These utilities include settlement sensitive wet utilities such as storm drain and sewer lines. In addition, several dry utilities are located within 10 feet of the site which will be adversely impacted by infiltration of storm water.

It is therefore our opinion that storm water infiltration at the site is not feasible.

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



Respectfully submitted
LEIGHTON AND ASSOCIATES, INC.

Mike D. Jensen

Mike Jensen, CEG 2457
Associate Geologist

Distribution: (1) email
Attachments (1) Appendix A - References

APPENDIX A

REFERENCES

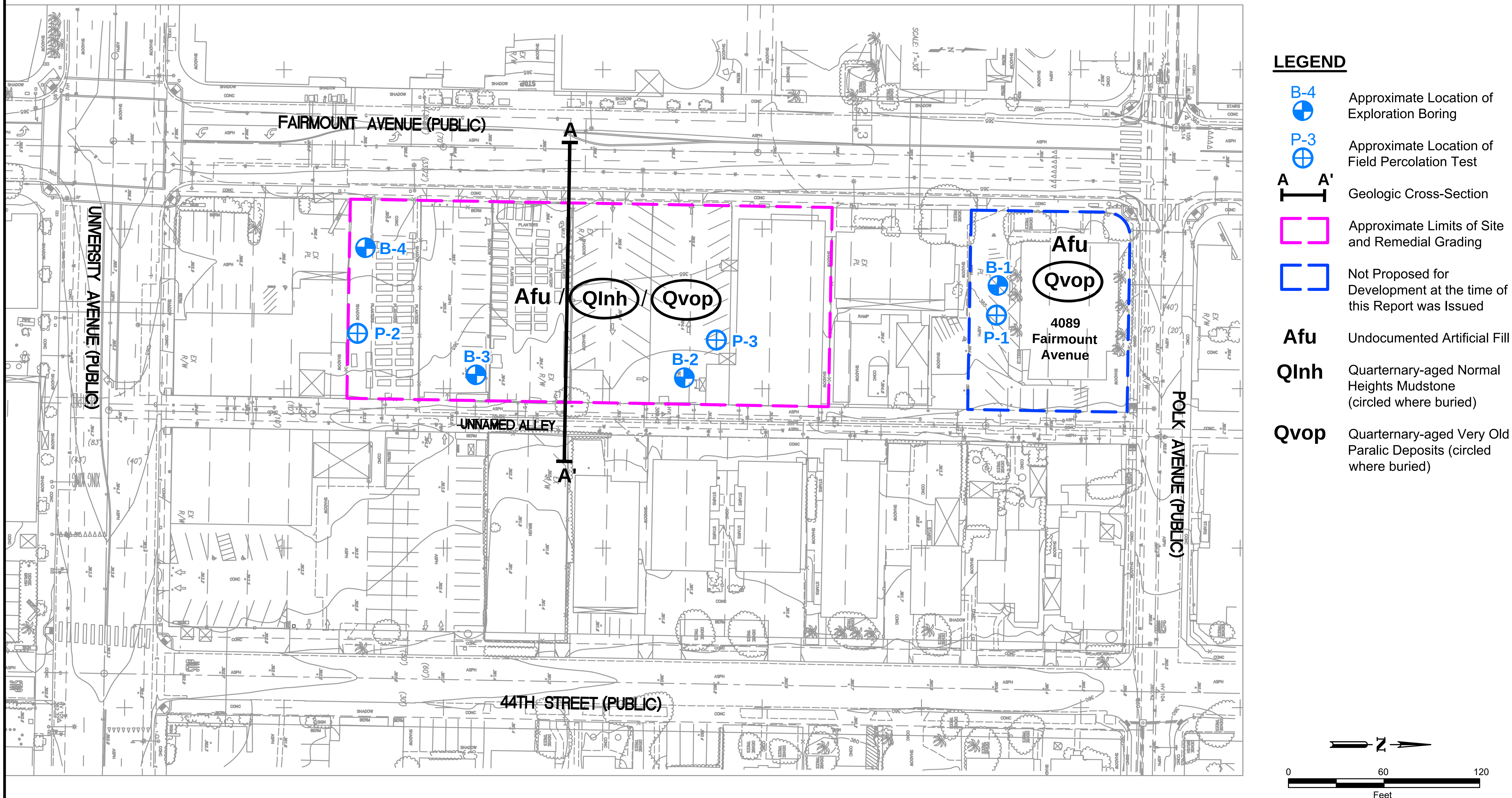
Chang Consultants,2020, Preliminary Drainage Report 4th Corner Apartments, San Diego, California, dated June 11, 2020.

Leighton and Associates, 2020, Updated Infiltration Feasibility Letter, Fourth Corner Residential Project, San Diego, California 92101, Project No 11534.003, dated April 15, 2020.

____________________, 2020, Addendum Geotechnical Investigation, Fourth Corner Residential Project, Fairmount Avenue, San Diego, California 92101, Project No 11534.003, dated April 13, 2020.

____________________, 2017, Preliminary Geotechnical Investigation, Fourth Corner Residential Project, Fairmount Avenue, San Diego, California 92101, Project No 11534.001, dated February 6, 2017.

Kettler Leweck, 2020, Preliminary Stormwater Management Plan Letter, San Diego, California, dated June 15, 2020.



Project: 11534.001	Eng/Geol: WDO/MJD
Scale: 1"=60'	Date: June 2020

Reference: Site Development Plan, Kettler Leweck, 4th Corner, 4089-4061-4035-4021 Fairmount Avenue, dated January 5, 2017.
Author: MAM

GEOTECHNICAL EXPLORATION MAP

4061, 4035, 4029, 4021 Fairmount Avenue
San Diego, California

Figure 2

Leighton

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Appendix C

Preliminary Geotechnical Investigation
(Leighton and Associates, 2017)

PRELIMINARY GEOTECHNICAL INVESTIGATION
FOURTH CORNER RESIDENTIAL PROJECT
FAIRMOUNT AVENUE
SAN DIEGO, CALIFORNIA

Prepared for:

City Heights Realty, LLC
7777 Fay Avenue, Suite 300
La Jolla, California 92037

Project No. 11534.001

February 6, 2017

Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

February 6, 2017

Project No. 11534.001

City Heights Realty, LLC
7777 Fay Avenue, Suite 300
La Jolla, California 92037

Attention: Mr. Mark Daitch

Subject: Preliminary Geotechnical Investigation
Fourth Corner Residential Project
Fairmount Avenue
San Diego, California

In accordance with your request and authorization, we have conducted a preliminary geotechnical evaluation of the subject property for the design and construction of a proposed five-story residential development with one level of on-grade parking located at 4021, 4029, 4035, 4061, and 4089 Fairmount Avenue in the City of San Diego, California. The property at 4089 Fairmont Avenue was explored; however, it is our understanding that this property is not currently being proposed for redevelopment at this time.

Based on the results of our study, it is our professional opinion that the site is suitable to receive the proposed improvements. The accompanying report presents a summary of our current investigation and provides geotechnical conclusions and recommendations relative to the proposed site development.

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,

LEIGHTON AND ASSOCIATES, INC.

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Leighton

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1.0 INTRODUCTION

We recommend that all individuals utilizing this report read the preceding information sheet prepared by GBC (the Geotechnical Business Council of the Geoprofessional Business Council) and the Limitations, Section 7.0, located at the end of this report.

1.1 Purpose and Scope

This report presents the results of our geotechnical investigation for the proposed residential development to be located at 4021, 4029, 4035, and 4061 Fairmount Avenue in a previously developed area known as City Heights in the City of San Diego, California (Figure 1). The property at 4089 Fairmont Avenue was explored; however, it is our understanding that this property is not currently being proposed for redevelopment at this time. The intent of this report is to provide specific geotechnical conclusions and recommendations for the currently proposed project.

1.2 Site Location and Description

The subject site is a rectangular shaped parcel of land located in a previously developed area known as City Heights within the City of San Diego (Figure 2, Geotechnical Exploration Map). In general, the site is bounded by Fairmount Avenue to the west, an alleyway to the east and existing commercial developments to the north and south. Overall dimensions of the subject site are approximately 130 by 240 lineal feet.

Currently the site is occupied by asphalt paved parking lots, a two-story commercial building (i.e., United Women of East Africa Organization) and areas that are used for urban gardening. Other site improvements consist of underground utilities, concrete hardscaping, and perimeter security fences. Site topography is nearly level with surface elevations ranging from approximately 366 to 364 feet above mean sea level (msl) (i.e., drainage from the west to the east).

Site Latitude and Longitude
32.7504º N
117.1005º W

1.3 Proposed Development

Based on our review of conceptual site plans (M.W. Steele Group, 2016) and our discussions with you, we understand that the proposed project will consist of a five-story residential structure. The first level will be a parking garage with approximately 90 parking spaces, and will cover the complete footprint of the proposed residential structure. The second level will consist of community multi-purpose / meeting rooms, a kitchen and dining area, an exterior patio area and perimeter residential units. Levels three through five will consist of residential units. Structural loads for the proposed building were not available for the preparation of this report. Associated improvements are anticipated to include underground utilities, landscaping, and hardscaping. Additional geotechnical analysis may be needed once structural loads for the building are known.

2.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

2.1 Site Investigation

Our subsurface exploration consisted of four (4) 8-inch small diameter geotechnical borings (B-1 through B-4) to approximately 12 to 14 feet below the existing ground surface (bgs). Note that boring B-1 was located at 4089 Fairmont Avenue, which is off-site and not currently apart of the proposed redevelopment at this time. It should also be noted that auger refusal was encountered in all geotechnical borings due to the presence of large cobbles. The purpose of our subsurface exploration was to evaluate the underlying stratigraphy, physical characteristics, and specific engineering properties of the soils within the area of the proposed improvements. All geotechnical borings were drilled using a heavy duty truck mounted hollow-stem auger drill rig. During the exploration operations, a geologist from our firm prepared geologic logs and collected bulk and relatively undisturbed samples for laboratory testing and evaluation.

Additionally, three (3) field percolation tests were performed for the evaluation of the infiltration characteristics. The in-situ field percolation testing was performed on January 13, 2017 and January 26, 2017 in general accordance with City of San Diego Storm Water Standards (City of San Diego, 2016), Section D.3.3.2 and the Riverside County Borehole Percolation method. The test boreholes were drilled using a heavy duty truck mounted hollow-stem auger drill rig to a depth of 4 feet bgs. The water levels during field percolation test were measured at 30 to 60 minute intervals using a water level sounder.

After logging and field testing, the borings and percolation test holes were backfilled with drill cuttings. The boring logs and field percolation test results are provided in Appendix B. The boring and field percolation test locations are depicted on Figure 2 (Geotechnical Exploration Map).

2.2 Laboratory Testing

Laboratory testing performed on representative soil samples obtained during the subsurface explorations expansion index, atterberg limits and geochemical analysis. A discussion of the laboratory tests performed and a summary of the laboratory test results are presented in Appendix C.

3.0 SUMMARY OF GEOTECHNICAL CONDITIONS

3.1 Geologic Setting

The project area is situated in the Peninsular Ranges Geomorphic Province. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California, and varies in width from approximately 30 to 100 miles (Norris and Webb, 1990). The province is characterized by mountainous terrain on the east composed mostly of Mesozoic igneous and metamorphic rocks, and relatively low-lying coastal terraces to the west underlain by late Cretaceous-age, Tertiary-age, and Quaternary-age sedimentary units. Most of the coastal region of the County of San Diego occurs within this province and is underlain by sedimentary units. The subject site is located within the coastal plain section of the Peninsular Range Geomorphic Province of California, which generally consists of subdued landforms underlain by sedimentary bedrock. Specifically, the site is located in an area underlain by generally shallow undocumented artificial fill, which in turn is underlain by the Quaternary-aged Normal Heights Mudstone and Quaternary-aged Very Old Paralic Deposits.

3.2 Site-Specific Geology

Based on our subsurface exploration and review of pertinent geologic literature and maps (Appendix A), the geologic units underlying the site consist of shallow undocumented artificial fill overlying the Quaternary-aged Normal Heights Mudstone and Quaternary-aged Very Old Paralic Deposits. The approximate areal distribution of the geologic units is depicted on the Geotechnical Exploration Map (Figure 2). A brief description of the geologic units encountered at the site is presented below. The geotechnical boring logs with detailed soils descriptions are presented in Appendix B.

3.2.1 Undocumented Fill – Afu

Based on our subsurface exploration, undocumented fill was encountered in the on-site geotechnical borings B-2 through B-4 with a thickness of approximately 2 bgs. Locally deeper undocumented fill depths should be anticipated considering the site has been development. The fill materials at the subject site consist of dark brown, moist, soft, high plasticity, sandy lean clay with variable amounts of gravel and cobble. It should be noted that the undocumented fill is an expansive clayey soil, which may not be suitable for reuse without mitigation.

As observed in the off-site boring B-1 performed at 4089 Fairmount Avenue, we encountered undocumented fill to a depth of approximately 7 feet thick. The fill materials consist of light brownish gray, moist, loose to medium dense, silty sand. The undocumented fill was apparently placed during previous grading operations for the existing developments.

3.2.2 Normal Heights Mudstone – Qlnh

Based on our subsurface exploration and review of the applicable geotechnical publications, Pliocene-aged Normal Heights Mudstone was encountered underlying the undocumented fill and extended to depths of approximately 6 to 7 feet bgs at the subject site. The Normal Heights Mudstone, which caps the mesa, is generally composed of poorly consolidated claystone that is characteristically steel gray in color and highly cohesive (Reed, 1991). Where observed in our exploration, the Normal Heights Mudstone consists of very dark gray, moist, firm to very stiff, high plasticity, claystone with interbedded layers of gravel and cobble. Based on laboratory expansion testing, the clayey soil has a “very high” expansion potential, which may not be suitable for reuse without mitigation.

3.2.3 Very Old Paralic Deposits – Qvop

Previously, the site was mapped as being underlain by the Linda Vista Formation (Kennedy, 1975). More recent mapping by Kennedy and Tan, 2008, has renamed the previously mapped geologic formation as Very Old Paralic Deposits. These middle to late Pleistocene-aged Very Old Paralic Deposits underlie the entire site, and extend to the maximum depth explored of approximately 14 feet bgs. As encountered, these deposits consist primarily of light yellowish brown to yellowish brown, dense to very dense, moist, fine-grained, oxidized, clayey sandstone with gravel with interbedded layers of cobble conglomerate. Based on our experience with similar sites, excavations within this unit may encounter zones of poorly graded cohesionless and friable sands that may cave or slough during unsupported site excavation. Note that we encountered auger refusal in all the geotechnical boring explorations due to the presence of large cobbles located within the Very Old Paralic Deposits. The cobbles located throughout the Very Old Paralic Deposits are 6 to 8 inches in diameter with isolated cobbles up to 1 foot in diameter anticipated.

3.3 Surface and Groundwater

No indication of surface water or evidence of surface ponding was encountered during our geotechnical investigation at the subject site. However, surface water may drain as sheet flow across the site during rainy periods.

Groundwater was not encountered during our geotechnical investigation at the subject site. Groundwater depths are anticipated to be greater than 100 feet below the ground surface. Based on our review of the conceptual plans and our experience with similar projects, groundwater is not expected to be a constraint to site development.

It should be noted that in the off-site boring B-1 performed at 4089 Fairmount Avenue, we encountered a perched groundwater conditions at a depth of 2 feet bgs. The perched groundwater and seepage conditions may fluctuate with seasonal variations and irrigation.

3.4 Engineering Characteristics of On-site Soils

Based on the results of our laboratory testing of representative on-site soils, and our professional experience on similar sites with similar soils conditions, the engineering characteristics of the on-site soils are discussed below.

3.4.1 Compressible Soils

The site is underlain by undocumented artificial fill materials and the weathered upper portions of the Normal Heights Mudstone which are considered compressible in their current state. Recommendations for remedial grading of these soils are provided in the following sections of this report.

3.4.2 Expansion Potential

As discussed above, the undocumented artificial fill and Normal Heights Mudstone materials at the subject site are anticipated to be highly expansive. Expansion index testing indicates that the near surface soils are greater than 130 (i.e., very high expansion potential). Geotechnical observations and additional laboratory testing of the excavation materials are recommended to determine the expansion potential of on-site soils used during grading.

3.4.3 Soil Corrosivity

A preliminary screening for corrosive soil was completed to evaluate their potential effect on concrete and ferrous metals. Specifically, laboratory testing was performed to evaluate pH, minimum electrical resistivity, and chloride and soluble sulfate content. The sample tested had measured pH value of 6.7, and a measured minimum electrical resistivity of 375 ohm-cm. Test results also indicated that the sample had a chloride content of 348 parts per million (ppm), and a soluble sulfate content of 500 ppm.

3.4.4 Excavation Characteristics

It is anticipated the on-site soils can be excavated with conventional heavy-duty construction equipment. Localized cemented zones, if encountered, may require heavy ripping or breaking. If oversize material (larger than 8 inches in maximum dimensions) is generated, it should be placed in non-structural areas or hauled off-site. Localized interbedded gravels and cobbles may be encountered within the Very Old Paralic Deposits. In addition, localized zones of friable sands also may occur within the Very Old Paralic Deposits. Beds of friable sands, gravel, and cobble may experience caving during unsupported excavation or drilling.

3.4.5 Infiltration

Percolation tests were performed in general accordance with the County of Riverside borehole percolation method and City of San Diego Storm Water Standards. 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 should be noted that we have used the following equation based upon the Porchet Method to convert measured percolation rates to infiltration rates in accordance with County of Riverside Standards (2011). 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:

$$I_t = \frac{\Delta H * 60 * r}{\Delta t(r+2H_{AVG})}$$

Where:

I_t = calculated infiltration rate, inches/hour

ΔH = change in head over the time interval, inches

Δt = time interval, minutes

r = radius of test hole

H_{AVG} = average head over the time interval, inches

The field percolation test locations are shown on Figure 2 (Geotechnical Exploration Map). Field data and calculated percolation rates for each field percolation test location is presented in Appendix B.

Table 1
Percolation and Infiltration Rates

Test No.	Depth (ft)	Soil Type	Measured Percolation Rate (mins/in)	Calculated Infiltration Rate (inches/hr)	Recommended Infiltration Rate w/ FS of 2 (inches/hr)
P-1*	4	Artificial Fill (Afu)	NP	N/A	N/A
P-2	4	Normal Heights Mudstone (Qlnh)	1,200	<0.01	<0.005
P-3	4	Normal Heights Mudstone (Qlnh)	1,500	<0.01	<0.005

* Off-site Test Location. NP – No percolation measured.

Based on the field percolation testing and the recommended calculated infiltration rates, the site is categorized as "No-Infiltration", as determined by the Storm Water Standards BMP Design Manual, San Diego Region, February 2016. The City of San Diego Infiltration Worksheet C.4-1, Categorization of Infiltration Feasibility Condition, has been completed and is presented in Appendix D. Note that the above percolation test results are representative of the tested locations and depths where they were performed. It should also be noted that percolation test field measurements are accurate to 0.01 feet. Varying subsurface conditions may exist outside of the test locations, which could alter the calculated percolation rate indicated below. In addition, 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.

It is possible that the long term rate of transmissivity of permeable soil strata may be lower than the values obtained by testing. Infiltration may be influenced by a combination of factors including but not limited to: a highly variable vertical permeability and limited lateral extent of permeable soil strata; a reduction of permeability rates over time due to silting of the soil pore spaces; and other unknown factors. 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.

4.0 SEISMICITY

4.1 Regional Tectonic Setting

The California Geologic Survey (CGS, 2007) define an active fault as a fault which has “had surface displacement within Holocene time (about the last 11,000 years).” The City of San Diego (1999) further defines a Potentially Active fault, as a fault that has had activity within the last 1.6 million years (Quaternary Period) and can be demonstrated to be inactive during the last 11,000 years (Holocene Epoch).

The site is located within the Peninsular Ranges Geomorphic Province, which is traversed by several major active faults. The Whittier-Elsinore, San Jacinto, and the San Andreas faults are major active fault systems located east of the site, and the Rose Canyon, Newport-Inglewood (offshore), and Coronado Bank are active faults located west to southwest of the site (Jennings, 2010). The primary seismic risk to the site area is the Rose Canyon fault zone located 4.4 miles west of the site (USGS, 2014).

The Rose Canyon fault zone consists predominantly of right-lateral strike-slip faults that extend south-southeast bisecting the San Diego metropolitan area. Various fault strands display strike-slip, normal, oblique, or reverse components of displacement. The Rose Canyon fault zone extends offshore at La Jolla and continues north-northwest subparallel to the coastline. The offshore segments are poorly constrained regarding location and character. South of downtown, the fault zone splits into several splays that underlie San Diego Bay, Coronado, and the ocean floor south of Coronado (Treiman, 1993 and 2000; Kennedy and Clarke, 1999). Portions of the fault zone in the Mount Soledad, Rose Canyon, and downtown San Diego areas have been designated by the State of California (CGS, 2003) as being Earthquake Fault Zones.

4.2 Local Faulting

Our review of available geologic literature (Appendix A) indicates that there are no known Active or Potentially Active faults transecting the site. The subject site is also not located within any State mapped Earthquake Fault Zones or County of San Diego mapped fault zones. The nearest active fault is the Rose Canyon fault zone located approximately 4.4 miles west of the site (USGS, 2014).

4.3 Seismicity

The site is considered to lie within a seismically active region, as is all of Southern California. As previously mentioned above, the Rose Canyon fault zone located approximately 4.4 miles west of the site is considered the ‘active’ fault having the most significant effect at the site from a design standpoint.

4.4 Seismic Hazards

Severe ground shaking is most likely to occur during an earthquake on one of the regional active faults in Southern California. The effect of seismic shaking may be mitigated by adhering to the California Building Code or state-of-the-art seismic design parameters of the Structural Engineers Association of California.

4.4.1 Shallow Ground Rupture

As mentioned above, no active or potentially active faults are mapped crossing or projecting toward the site. No faults were encountered during our fault study. Due to the absence of faults at the site, surface rupture from faulting is considered low. In addition, due to the lack of nearby slopes, ground cracking due to shaking from a seismic event is also considered low.

4.4.2 Mapped Fault Zones

The site is not located within a State mapped Earthquake Fault Zone (EFZ), nor is it located within a City of San Diego fault zone. As previously discussed, the subject site is not underlain by known active or potentially active faults.

4.4.3 Site Class

Utilizing 2016 California Building Code (CBC) procedures, we have characterized the site soil profile to be a Site Class D based on our experience with similar sites in the project area.

4.4.4 Building Code Mapped Spectral Acceleration Parameters

The effect of seismic shaking may be mitigated by adhering to the California Building Code and state-of-the-art seismic design practices of the Structural Engineers Association of California. Provided below in Table 2 are the spectral acceleration parameters for the project determined in accordance with the 2016 CBC (CBSC, 2016) and the USGS U.S. Seismic Design Maps Web Application (June, 2014).

Table 2
2016 CBC Mapped Spectral Acceleration Parameters

Site Class	D		
Site Coefficients	F_a	=	1.101
	F_v	=	1.640
Mapped MCE Spectral Accelerations	S_S	=	0.997g
	S_1	=	0.380g
Site Modified MCE Spectral Accelerations	S_{MS}	=	1.098g
	S_{M1}	=	0.623g
Design Spectral Accelerations	S_{DS}	=	0.732g
	S_{D1}	=	0.415g

Utilizing ASCE Standard 7-10, in accordance with Section 11.8.3, 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 0.416g for the site. For a Site Class D, the F_{pga} is 1.084 and the mapped peak ground acceleration adjusted for Site Class effects (PGA_m) is 0.451g for the site.

4.5 Secondary Seismic Hazards

In general, secondary seismic hazards can include soil liquefaction, seismically-induced settlement, lateral displacement, surface manifestations of liquefaction, landsliding, seiches, and tsunamis. The potential for secondary seismic hazards at the subject site is discussed below.

4.5.1 Liquefaction and Dynamic Settlement

Liquefaction and dynamic settlement of soils can be caused by strong vibratory motion due to earthquakes. Granular soils tend to densify when subjected to shear strains induced by ground shaking during earthquakes. Research and historical data indicate that loose granular soils underlain by a near surface groundwater table are most susceptible to liquefaction, while the most clayey materials are not susceptible to liquefaction. Liquefaction is characterized 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 at the ground surface by settlement and, possibly, sand boils where insufficient confining overburden is present over liquefied layers. Where sloping ground conditions are present, liquefaction-induced instability can result.

The site is underlain at depth by weakly to moderately cemented and moderately well indurated clayey sandstone with gravel and claystone. Since loose surficial fill are recommended for removal, the underlying dense character of the on-site formational deposits, and the lack of a shallow ground water table, it is our opinion that the potential for liquefaction and seismic related settlement across the site is low.

4.5.2 Lateral Spread

Empirical relationships have been derived (Youd et al., 1999) to estimate the magnitude of lateral spread due to liquefaction. These relationships include parameters such as earthquake magnitude, distance of the earthquake from the site, slope height and angle, the thickness of liquefiable soil, and gradation characteristics of the soil.

The susceptibility to earthquake-induced lateral spread is considered to be low for the site because of the low susceptibility to liquefaction and relatively level ground surface in the site vicinity.

4.5.3 Tsunamis and Seiches

Based on historical records, the orientation and distance of the San Diego coastline, the presence of the San Diego Bay, and the generally strike-slip character of off-shore faulting, it is our opinion that the potential for damage to occur at the site due to either a tsunami or seiche is nil. In addition, the site is not located within mapped tsunami inundation zone (CalEMA, 2009).

4.6 Landslides

Several formations within the San Diego region are particularly prone to landsliding. These formations generally have high clay content and mobilize when they become saturated with water. Other factors, such as steeply dipping bedding that project out of the face of the slope and/or the presence of fracture planes, will also increase the potential for landsliding.

No landslides or indications of deep-seated landsliding were indicated at the site during our field exploration or our review of available geologic literature, topographic maps, and stereoscopic aerial photographs. Furthermore, our field reconnaissance and the local geologic maps indicate the site is generally underlain by favorable oriented geologic structure, consisting of massively bedded sandstone. Therefore, the potential for significant landslides or large-scale slope instability at the site is considered nil.

4.7 Flood Hazard

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2012); the site is not located within a floodplain. Based on our review of topographic maps, the site is not located downstream of a dam or within a dam inundation area. Based on this review and our site reconnaissance, the potential for flooding of the site is considered low.

5.0 CONCLUSIONS

Based on the results of our geotechnical investigation of the site, it is our opinion that the proposed development is feasible from a geotechnical standpoint, provided the following conclusions and recommendations are incorporated into the project plans and specifications.

- As the site is located in the seismically active southern California area, all structures should be designed to tolerate the dynamic loading resulting from seismic ground motions;
- The site is not transected by Potentially Active or Active faults;
- The undocumented artificial fill and the weathered upper portions of the Normal Heights Mudstone are considered potentially compressible and generally unsuitable in their present state to support additional fill or structural loads;
- Based on laboratory testing, the undocumented fill and Normal Heights Mudstone possess a very high expansion potential. Remedial grading and/or structural floor slab-on-grade will be needed to mitigate highly expansive soil conditions at the subject site;
- The existing on-site soils are generally suitable for use as engineered fill, provided they are free of organic material, debris, and rock fragments larger than 6 inches in maximum dimension, and that they are mitigated for a very high expansion potential;
- If import soils are used for mitigation of expansive soils, the soils should be granular in nature, and have an expansion index less than 50 (per ASTM Test Method D4829) and have a low corrosion impact to the proposed improvements;
- Based on the results of our subsurface exploration, we anticipate that the on-site materials should be generally excavatable with conventional heavy-duty earthwork equipment. Localized cemented zones within the Very Old Paralic Deposits may be difficult to excavate and may require heavy ripping which can produce oversized rock fragments. Unknown objects such as buried concrete footings and debris left from previous site uses should be anticipated and are common on sites where previous structures existed;
- The groundwater should not be encountered during remedial grading activities. Although not encountered during our exploration, localized seepage along cemented zones and sand lenses within the Very Old Paralic Deposits may occur;

- Based on the results of our geotechnical evaluation, it is our opinion that the proposed building can be supported on conventional foundations.
- Although Leighton does not practice corrosion engineering, laboratory test results indicate the soils present on the site have a negligible potential for sulfate attack on normal concrete. The on-site soils are considered to have a very severe potential for corrosion to buried uncoated ferrous metal. A corrosion consultant should be consulted; and
- The site is proposed for remedial grading of the near surface soils. The new compacted artificial fill will likely consist of a mixture of soils ranging from silty sands to sandy clays that will have permeable and impermeable layers that can transmit and perch ground water in unpredictable ways. In addition, the underlying Normal Heights Mudstone has very low infiltration rates. Therefore, Low Impact Development (LID) measures may impact down gradient improvements and the use of some LID measures may not be appropriate for this project. Any proposed bioretention stormwater designs should be reviewed by geotechnical consultant. It is likely that as a No-Infiltration site, impermeable membrane liners may be needed to prevent lateral migration of storm water.

6.0 RECOMMENDATIONS

6.1 Earthwork

We anticipate that earthwork at the site will consist of site preparation and remedial grading. We recommend that earthwork on the site be performed in accordance with the following recommendations and the General Earthwork and Grading Specifications for Rough Grading included in Appendix E. In case of conflict, the following recommendations supersede those in Appendix E.

6.1.1 Site Preparation

Prior to grading, all areas to receive structural fill or engineered structures should be cleared of surface and subsurface obstructions, including any existing debris 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 above-optimum moisture conditions, and recompacted to at least 90 percent relative compaction (based on ASTM Test Method D1557).

6.1.2 Excavations and Oversize Material

Excavations of the on-site materials may generally be accomplished with conventional heavy-duty earthwork equipment. However, local heavy ripping or breaking may be required if cemented formational material is encountered. Excavation for utilities may also be difficult in some areas.

Due to the cohesive characteristics of the Normal Heights Mudstone and high-density characteristics of the Very Old Paralic Deposits, 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. Artificial fill soils present on site may cave during trenching operations. In accordance with OSHA requirements, excavations deeper than 5 feet should be shored or be laid back in accordance with Section 6.7 if workers are to enter such excavations.

6.2 Remedial Grading

Potentially compressible undocumented fill and the weathered upper portions of the Normal Heights Mudstone at the subject site may settle as a result of wetting or settle under the surcharge of engineered fill and/or structural loads supported on shallow foundations. In addition, we recommend mitigation of highly expansive soils at the subject site. To mitigate the existing compressible and expansive soil conditions, we recommended that one of the following grading options be selected.

6.2.1 Option 1: Imported Granular Cap

For this option, we recommend that the compressible undocumented fill and the upper weathered portions of the Normal Heights Mudstone be removed and replaced with an imported granular cap. The depth of the removal should be at least three (3) feet below the proposed subgrade elevation or one (1) foot below bottom of deepest foundation element, whichever is lower. Additionally, the removal should extend (5) feet horizontally outside the proposed building footprint or the sensitive improvements. The bottom of the removals should be evaluated by a Certified Engineering Geologist to confirm conditions are as anticipated.

After the removal bottom is approval, it should be scarified to a depth of 12 inches and moisture conditioned to a moisture content of 3 to 5 percent over the optimum content and compacted to at least 90 percent of the maximum laboratory dry density, as evaluated by ASTM D 1557. The resulting excavation may then be filled with import soils which should be granular in nature, and have an expansion index less than 50 (per ASTM Test Method D4829) and have a low corrosion impact to the proposed improvements. In addition, rocks greater than 3 inches in diameter should not be placed within 2 feet of finished grade. Although the optimum lift thickness for fill soils will be dependent on the type of compaction equipment utilized, fill should generally be placed in uniform lifts not exceeding approximately 8 inches in loose thickness.

Imported fill soil should be moisture conditioned to at least 2 percent above the optimum moisture content and compacted to 90 percent or more relative compaction, in accordance with ASTM D 1557. Placement and compaction of fill should be performed in general accordance with current City of San Diego grading ordinances, California Building Code and sound construction practices, these recommendations and the General Earthwork and Grading Specifications for Rough Grading presented in Appendix E.

6.2.2 Option 2: Reprocessing of On-site Soil with Structural Slab

For this option, we recommend that the compressible undocumented fill and the upper weathered portions of the Normal Heights Mudstone be removed and reprocessed for reuse as fill soil. Note that if this option is selected, we recommend using a structural slab for the parking garage. The structural slab should be designed by the project structure engineer. The removal should be a minimum depth of five (5) feet below the existing surface grade or proposed subgrade elevation, whichever is lower. Additionally, the removal(s) should extend (5) feet horizontally outside the proposed building footprint or the sensitive improvements. The bottom of the removals should be evaluated by a Certified Engineering Geologist to confirm conditions are as anticipated.

After the removal bottom is approval, it should be scarified to a depth of 12 inches and moisture conditioned to a moisture content of 3 to 5 percent over the optimum content and compacted to at least 90 percent of the maximum laboratory dry density, as evaluated by ASTM D 1557. Prior to placement of the on-site soils as compacted fill, it should be moisture conditioned to at least 5 percent above the optimum moisture content and compacted to between 90 and 93 percent relative compaction, in accordance with ASTM D 1557.

Placement and compaction of fill should be performed in general accordance with current City of San Diego grading ordinances, California Building Code and sound construction practices, these recommendations and the General Earthwork and Grading Specifications for Rough Grading presented in Appendix E.

6.3 Earthwork Shrinkage/Bulking

The volume change of excavated on-site materials upon recompaction as fill is expected to vary with material and location. Typically, the fill soils, Normal Heights Mudstone and Very Old Paralic Deposits vary significantly in natural and compacted density, and therefore, accurate earthwork shrinkage/bulking estimates cannot be determined. However, based on the results of our geotechnical analysis and our experience, a 5 percent shrinkage factor is considered appropriate for the undocumented fill. The Normal Heights Mudstone is anticipated to be 3 to 5 percent bulking.

6.4 Trench Backfill

Pipe bedding should consist of sand with a sand equivalent (SE) of not less than 30. Bedding should be extended the full width of the trench for the entire pipe zone, which is the zone from the bottom of the trench, to one foot above the top of the pipe. The sand should be brought up evenly on each side of the pipe to avoid unbalanced loads. On-site materials will probably not meet bedding requirements. Except for predominantly clayey soils, the on-site soils may be used as trench backfill above the pipe zone (i.e. in the trench zone) provided they are free of organic matter and have a maximum particle size of three inches. Compaction by jetting or flooding is not recommended.

6.5 Expansive Soils and Selective Grading

Based on our laboratory testing and observations, we anticipate that the undocumented artificial fill and Normal Height Mudstone materials possess a very high expansion potential (Appendix C). Options to mitigate the presence of the highly expansive materials are detailed in Sections 6.2.1 through 6.2.4 above. In addition, expansion testing should be performed on-site materials during grading operations to confirm their expansion potential.

6.6 Import Soils

If import soils are used, the soil should be granular in nature, and have an expansion index less than 50 (per ASTM Test Method D4829), and have a low corrosion impact to the proposed improvements. Beneath pavements, subgrade materials should possess an R-Value of 20, or greater. Import soils and/or the borrow site location should be evaluated by the geotechnical consultant prior to import.

6.7 Temporary Excavations

Sloping excavations may be utilized when adequate space allows. Based on the results of our evaluation, we provide the following recommendations for sloped excavations in fill soils, Normal Heights Mudstone or the Very Old Paralic Deposits without seepage conditions.

Table 3 Maximum Slope Ratios		
Excavation Depth (feet)	Maximum Slope Ratio In Fill Soils	Maximum Slope Ratio In Normal Heights Mudstone/ Very Old Paralic Deposits
0 to 5	1:1 (Horizontal to Vertical)	Vertical
5 to 10	1:1 (Horizontal to Vertical)	1:1 (Horizontal to Vertical)

The above values are based on the assumption that no surcharge loading or equipment is present within 10 feet of the top of slope. Care should be taken during design of excavations adjacent to the existing structures so that foundation support is preserved. A “competent person” should observe the slope on a daily basis for signs of instability.

6.8 Foundation and Slab Considerations

At the time of drafting this report, building loads for foundations were not known. However, based on our understanding of the project, conventional foundations (spread and continuous footings) are considered suitable for support of the proposed five-story structure as long as the grading recommendations as discussed in Section 6.2.1 or 6.2.2, and founded on compacted fill or competent undisturbed native soil. Foundations and slabs should be designed in accordance with structural considerations and the following recommendations.

6.8.1 Option 1: Imported Granular Cap

For Option 1, the proposed structure may be supported by conventional, continuous or isolated spread footings founded in dense compacted fill. Footings should extend a minimum of 24 inches beneath the lowest adjacent soil grade. At these depths, footings may be designed for a maximum allowable bearing pressure of 4,000 pounds per square foot (psf). The allowable bearing pressures may also 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. Footings and slabs should be designed in accordance with the structural engineer's requirements.

For the parking garage floor slab, we recommend using a conventional slab-on-grade floor with reinforcing bars. Slabs should be a minimum of 5 inches thick and reinforced with No. 3 rebars at 24 inches on center on center (each way). Slabs should have crack joints at spacings designed by the structural engineer. Columns should be structurally isolated from slabs. If applicable, slabs should also be designed for the anticipated traffic loading using a modulus of subgrade reaction of 140 pounds per cubic inch.

In areas with moisture sensitive flooring, the slab should be underlain by 2-inch layer of clean sand (S.E. greater than 30). A moisture barrier (10-mil non-recycled plastic sheeting) should be placed below the sand layer if reduction of moisture vapor up through the concrete slab is desired (such as below equipment, living/office areas, etc.), which is in turn underlain by an additional 2-inches of clean sand. All waterproofing and moisture vapor measures should be designed by the project architect.

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.

6.8.2 Option 2: Reprocessing of On-site Soil with Structural Slab

For Option 2, the proposed structure may be supported by conventional, continuous or isolated spread footings founded in dense compacted fill. Footings should extend a minimum of 36 inches beneath the lowest adjacent soil grade. At these depths, footings may be designed for a maximum allowable bearing pressure of 4,000 pounds per square foot (psf). The allowable bearing pressures may also 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. Footings and slabs should be designed in accordance with the structural engineer's requirements.

For the parking garage floor slab, we recommend using a structural slab-on-grade floor with reinforcing bars. Slabs should be a minimum of 6 inches thick and reinforced with No. 4 rebars at 18 inches on center on center (each way). Slabs should have crack joints at spacings designed by the structural engineer. Columns should be structurally isolated from slabs. If applicable, slabs should also be designed for the anticipated traffic loading using a modulus of subgrade reaction of 100 pounds per cubic inch.

In areas with moisture sensitive flooring, the slab should be underlain by 2-inch layer of clean sand (S.E. greater than 30). A moisture barrier (10-mil non-recycled plastic sheeting) should be placed below the sand layer if reduction of moisture vapor up through the concrete slab is desired (such as below equipment, living/office areas, etc.), which is in turn underlain by an additional 2-inches of clean sand. All waterproofing and moisture vapor measures should be designed by the project architect.

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.

6.8.3 Settlement

For conventional footings, the recommended allowable-bearing capacity is based on a maximum total and differential static settlement of 1 inch and ¾ inch, respectively. Since settlements are a function of footing size and contact bearing pressures, some differential settlement can be expected where a large differential loading condition exists.

6.8.4 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
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 to High	130 percent of the optimum moisture content to a minimum depth of 24 inches below slab subgrade
Very High	140 percent of the optimum moisture content to a minimum depth of 30 inches below slab subgrade

6.8.5 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 5 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 5
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.

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.

6.8.6 Lateral Earth Pressures and Retaining Wall Design

Should retaining walls be added to the project, Table 6 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 6
Static Equivalent Fluid Weight (pcf)

Conditions	Level	2:1 Slope
Active	35	55
At-Rest	55	65
Passive	350 (Maximum of 3 ksf)	150 (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 6.3.3.

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 8H 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.

6.9 Control of Groundwater and Surface Waters

Based on the results of our field percolation tests for the subject site, the site is classified as a “No-Infiltration” site. Specifically, across the site the reliable infiltration rate is below 0.01 inches per hour (see Section 3.4.5, Table 1) due to the clayey nature of the Normal Heights Mudstone and the very dense nature of the underlying Very Old Paralic Deposits. In general accordance with City of San Diego Storm Water Standards and the BMP Design Manual, we have provided a completed copy of the City of San Diego Infiltration Worksheet C.4-1 in Appendix D.

It should be noted that unlined bioswales, infiltration basins, and other unlined on-site detention and retention systems can potentially create adverse perched ground water conditions both on-site and off-site. However, Low Impact Development (LID) BMPs that contain and filter surface waters (flow-through planters and bioretention areas) are acceptable provided that they are completely lined with an impermeable liner and have subdrain systems that tie into an approved existing or proposed storm drain system.

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.

6.10 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 4 to 6 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. Due to the potential for high expansive soil present in portions of the site, we recommend the flatwork near curbs and the interior and exterior entryways for the inclusion of dowels between curbs and/or exterior flatwork.

6.11 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 City of San Diego Standard Specifications (Schedule J).

For areas subject to regular truck loading (i.e., trash truck apron), we recommend a full depth of Portland Cement Concrete (PCC) section of 7 inches with appropriate 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 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.

6.12 Geochemical Considerations

Concrete in direct contact with soil or water that contains a high concentration of soluble sulfates can be subject to chemical deterioration commonly known as "sulfate attack." Soluble sulfate results (Appendix C) indicate negligible soluble sulfate content. We recommend that concrete in contact with earth materials be designed in accordance with Section 4 of ACI 318-11 (ACI, 2011).

Based on the results of preliminary screening laboratory testing, the site soils have a generally very high corrosion potential to buried uncoated metal conduits (Caltrans, 2012). We recommend measures to mitigate corrosion be implemented during design and construction. Leighton does not practice corrosion engineering. Therefore, a corrosion engineer may be contacted for additional recommendations.

6.13 Construction Observation and Plan Reviews

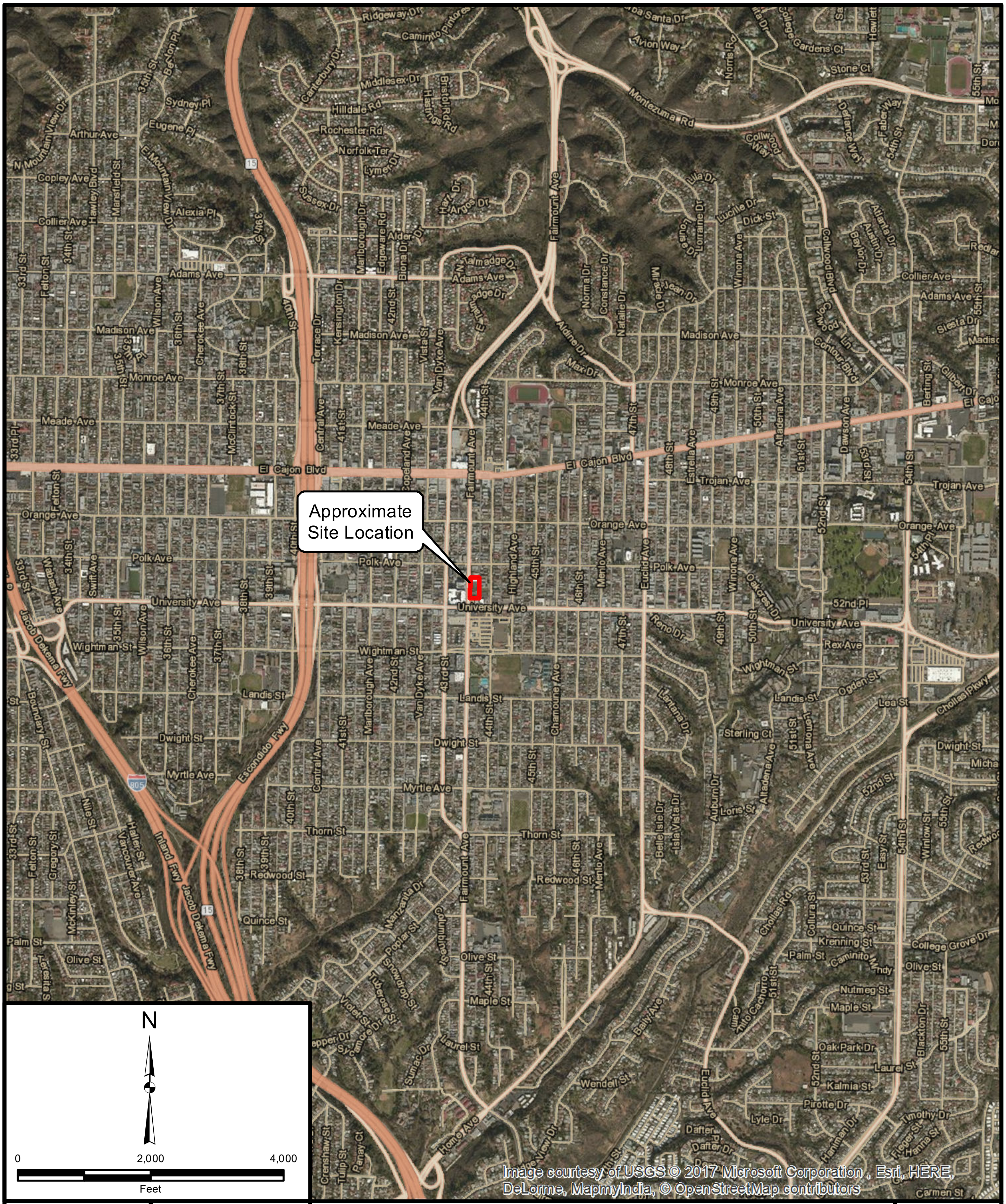
The recommendations provided in this report are based on preliminary design information and subsurface conditions disclosed by widely spaced borings. The interpolated subsurface conditions should be checked in the field during construction. Construction observation of all on-site excavations and field 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. In addition, during the installation of perimeter shoring systems, the City of San Diego requires that a geologist be on-site to log sidewalls for potential faults. In addition following completion of the temporary shoring, the City will require an "as-built" letter regarding observed geologic conditions prior to the approval of building inspection services.

Final project drawings should be checked by Leighton and Associates, Inc. before excavation to see that the recommendations provided in this report are incorporated in the project plans.

7.0 LIMITATIONS

The conclusions and recommendations presented 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: 11534.001	Eng/Geol:WDO/MDJ
Scale: 1 " = 2,000 '	Date: January 2017

Base Map: ESRI ArcGIS Online 2017
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

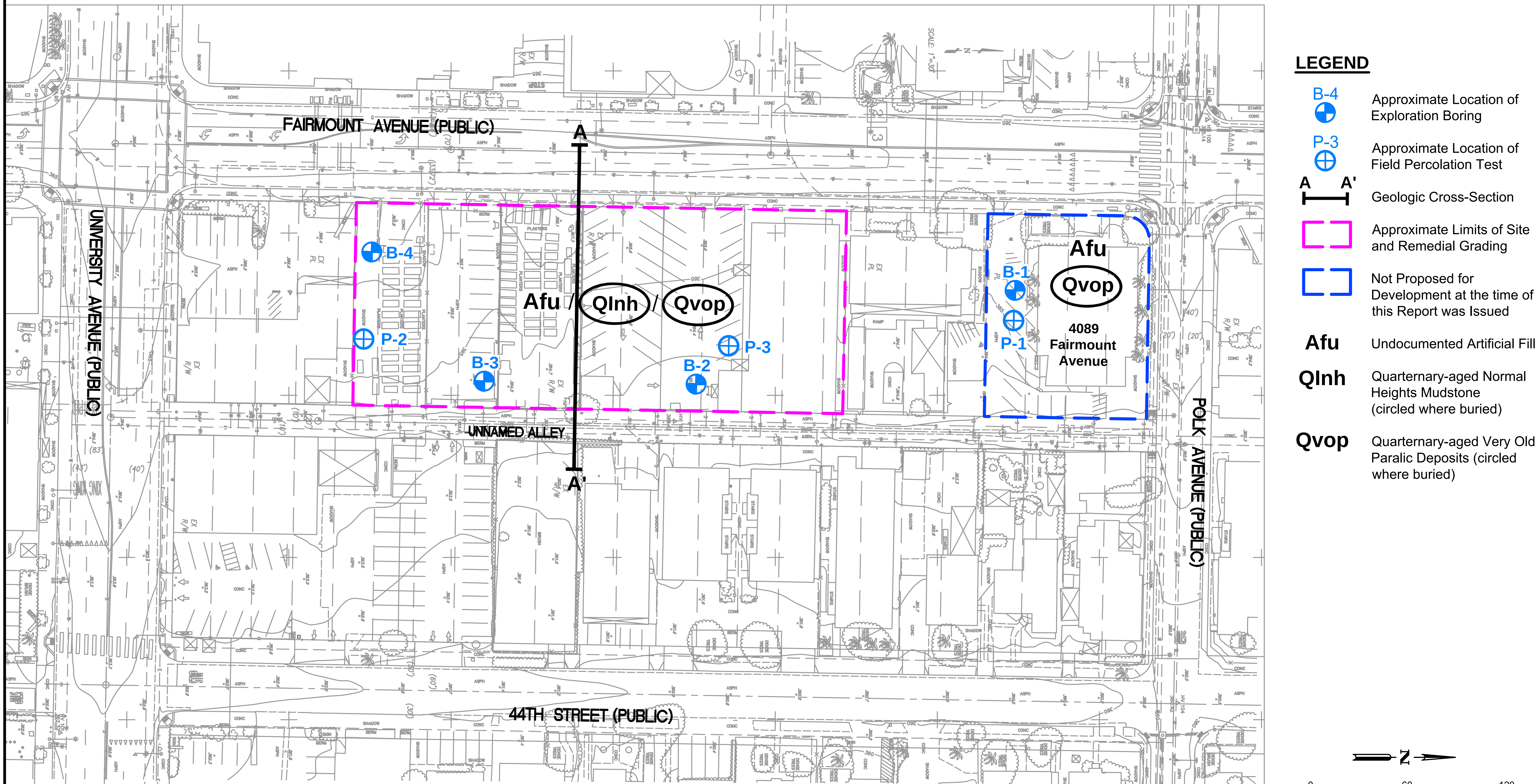
SITE LOCATION MAP

4061, 4035, 4029, 4021 Fairmount Avenue
San Diego, California

Figure 1

Leighton

Map Saved as V:\Drafting\11534\001\Maps\11534-001_F01_SLM_2017-01-23.mxd on 1/25/2017 1:12:40 PM



Project: 11534.001	Eng/Geol: WDO/MJD
Scale: 1"=60'	Date: January 2017

Reference: Site Development Plan, Kettler Leweck, 4th Corner, 4089-4061-4035-4021 Fairmount Avenue, dated January 5, 2017.
Author: MAM

GEOTECHNICAL EXPLORATION MAP

4061, 4035, 4029, 4021 Fairmount Avenue
San Diego, California

Figure 2

Leighton

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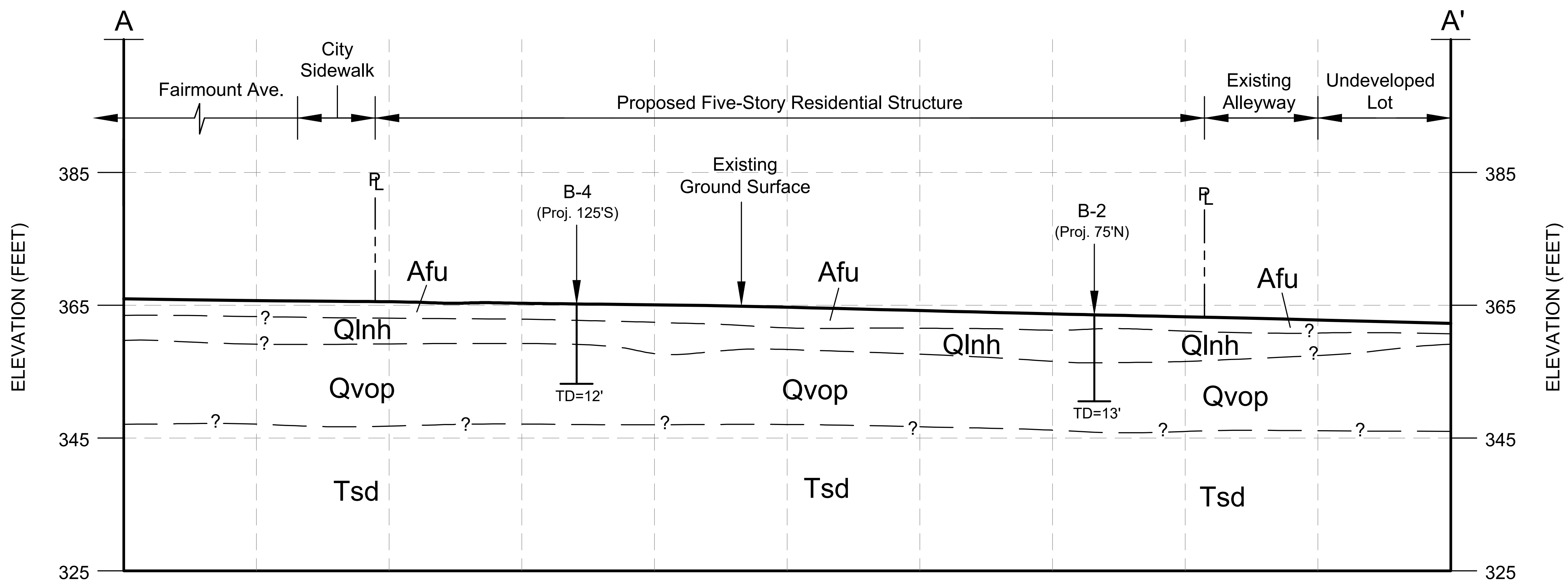

LEGEND

B-3 / TD=13'	Approximate Location of Exploratory Boring showing total depth	Afu	Undocumented Artificial Fill
		Qlnh	Quarternary-aged Normal Heights Mudstone
— — —	Approximate Geologic Contact	Qvop	Quarternary-aged Very Old Paralic Deposits
		Tsd	Tertiary-aged San Diego Formation

0 20 40
Feet

Project: 11534.001	Eng/Geol: WDO/MDJ
Scale: 1"=20'	Date: January 2017
Author: MAM	

GEOLOGIC CROSS-SECTION A-A'

4061, 4035, 4029, 4021 Fairmount Avenue
San Diego, California

Figure 3

Leighton

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Appendix A
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APPENDIX A

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REFERENCES (Continued)

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Appendix B
Boring Logs and Field Percolation Test Results

GEOTECHNICAL BORING LOG KEY

Project No.		**Date Drilled**	
Project	KEY TO BORING LOG GRAPHICS	**Logged By**	
Drilling Co.		**Hole Diameter**	
Drilling Method		**Ground Elevation**	
Location		**Sampled By**	

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
	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	
	20			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.)	
				S-1					Standard Penetration Test SPT (Sampler (2" O.D., 1.4" I.D.)	
	25			PUSH					Sampler Penetrates without Hammer Blow	
	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
- TR THERMAL RESISTIVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG B-1

Project No.	11534.001	**Date Drilled**	1-12-17
Project	4th Corner Residential Project	**Logged By**	RNB
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	CME-95 - 140lb - Autohammer - 30" Drop	**Ground Elevation**	366' msl
Location	See Figure 2	**Sampled By**	RNB

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								3" ASPHALT CONCRETE over 3" AGGREGATE BASE	
365								SM	ARTIFICIAL FILL (Afu) @ 6": Silty SAND, loose, light brownish gray (10 yr 6/2), moist, fine-grained, micaceous @ 2': Groundwater seepage encountered	
360	5			R-1	5 9 18				@ 5': Becomes medium dense and saturated	
								SC	VERY OLD PARALIC DEPOSITS (Qvop) @ 7': Clayey SANDSTONE with GRAVEL, dense, light yellowish brown (10 yr 6/4), moist, fine-grained, oxidation staining, micaceous @ 8': Cobble encountered	
355	10			S-1	50/2"					
				S-2	50/1"				@ 13': Cobble encountered	
350	15								**Auger Refusal on Cobble at 14 Feet bgs** **Groundwater seepage encountered at 2 Feet bgs** **Backfilled with soil cuttings on 1/12/17**	
345	20									
340	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 B-2

Project No.	11534.001	**Date Drilled**	1-12-17
Project	4th Corner Residential Project	**Logged By**	RNB
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	CME-95 - 140lb - Autohammer - 30" Drop	**Ground Elevation**	364' msl
Location	See Figure 2	**Sampled By**	RNB

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								3" ASPHALT CONCRETE over 3" AGGREGATE BASE	
				B-1 1'-5'				CL	ARTIFICIAL FILL (Afu) @ 6": Sandy lean CLAY, soft, dark brown (10 yr 3/3), moist, medium to high plasticity, trace organics	
								CL	NORMAL HEIGHTS MUDSTONE (Qlnh) @ 2': Lean CLAYSTONE, firm, very dark gray (7.5 yr 3/1), high plasticity, trace gravel	
360	5			R-1	39 33 42				@ 5': Becomes very stiff, gravel and cobble encountered	
355								SC	VERY OLD PARALIC DEPOSITS (Qvop) @ 7': Clayey SANDSTONE with GRAVEL, dense, light yellowish brown (10 yr 6/4), moist, fine-grained, oxidation staining, micaceous @ 8': Cobble encountered	
	10			R-2	50/2"					
				S-1	50/1"				@ 12': Cobble encountered	
350	15								**Auger Refusal on Cobble at 12 Feet bgs** **No groundwater encountered during exploration** **Backfilled with soil cuttings on 1/12/17**	
345	20									
340	25									
335	30									

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

TYPE OF TESTS:

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

GEOTECHNICAL BORING LOG B-3

Project No.	11534.001	**Date Drilled**	1-12-17
Project	4th Corner Residential Project	**Logged By**	RNB
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	CME-95 - 140lb - Autohammer - 30" Drop	**Ground Elevation**	364' msl
Location	See Figure 2	**Sampled By**	RNB

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								3" ASPHALT CONCRETE over 3" SAND BASE	
				B-1 1'-5'				CL	ARTIFICIAL FILL (Afu) @ 6": Sandy lean CLAY, soft, very dark grayish brown (10 yr 3/2), moist, high plasticity	
360								CL	NORMAL HEIGHTS MUDSTONE (Qlnh) @ 2': Lean CLAYSTONE, firm, very dark gray (7.5 yr 3/1), moist, high plasticity, trace gravel	
	5			R-1	39 50/3"				@ 5': Becomes very stiff, gravel and cobble encountered	CR, EI, AL
355				S-1	22 11 16			SC	VERY OLD PARALIC DEPOSITS (Qvop) @ 7': Clayey SANDSTONE with GRAVEL, dense, yellowish brown (10 yr 5/8, moist, fine-grained, micaceous, oxidation staining @ 8': Cobble encountered	
	10			S-2	7 15 50/6"					
350				S-3	50/1"				@ 13': Cobble encountered	
	15								**Auger Refusal on cobble at 13 Feet bgs** **No groundwater or seepage encountered** **Backfilled with soil cuttings on 1/12/17**	
345	20									
340	25									
335	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 B-4

Project No.	11534.001	**Date Drilled**	1-12-17
Project	4th Corner Residential Project	**Logged By**	RNB
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	CME-95 - 140lb - Autohammer - 30" Drop	**Ground Elevation**	366' msl
Location	See Figure 2	**Sampled By**	RNB

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
									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.	
365	0							CL	ARTIFICIAL FILL (Afu) @ 6": Sandy lean CLAY, soft, very dark grayish brown (10yr 3/2), moist, medium to high plasticity, trace gravel and cobble	
								CL	NORMAL HEIGHTS MUDSTONE (Qlnh) @ 2': Lean CLAYSTONE, firm, very dark gray (7.5 yr 3/1), moist, high plasticity, trace gravel	
	5			R-1	50/1"				@ 5': Becomes very stiff, gravel and cobble encountered	
360								SC	VERY OLD PARALIC DEPOSITS (Qvop) @ 6': Clayey SANDSTONE with GRAVEL, dense, yellowish brown (10 yr 5/8), moist, fine-grained, oxidation staining, micaceous @ 7': Cobble encountered	
355	10			S-1	50/2"					
				S-2	50/1"				@ 12': Cobble encountered	
									Auger Refusal on Cobble at 12 Feet bgs **No groundwater or seepage encountered during exploration** **Backfilled with soil cuttings on 1/12/17**	
350	15									
345	20									
340	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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FIELD PERCOLATION TEST DATA SHEET

Project Name:	4TH Corner Residential Project	Project No.:	11534.001
Proj. Address:	4021, 4029, 4035, 4061,4089, Fairmount Ave, San Diego, California		

SOIL TYPE / TEST LOCATION / BOREHOLE

Soil Type: Silty SAND (SM) - Artfifical Fill (Afu) Hole: P-1

Location: See Figure 2

Hole Dia: 8"

Depth: 4 Feet (bgs)

Tested by: RNB Pre-Saturation Date: 1-12-17 Test Date: 1-13-17

Notes: See Below

Time of Day	Interval / Notes	Initial Depth to Water (in.)	Final Depth of Water (in.)	Δ in Water Level (in.)	Percolation Rate (min/inch)
8:00	Start	24.00	-	-	-
8:30	30	24.00	24.00	0.00	NP
9:00	30	24.00	24.00	0.00	NP
10:00	60	24.00	24.00	0.00	NP
11:00	60	24.00	24.00	0.00	NP
12:00	60	24.00	24.00	0.00	NP
1:00	60	24.00	24.00	0.00	NP
2:00	60	24.00	24.00	0.00	NP

Notes: Perched groundwater conditions possibly related to a broken underground utility resulted in filling of water in the test hole to a depth of 24-inches below the existing ground surface. (NP-No Percolation Measured).

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FIELD PERCOLATION TEST DATA SHEET

Project Name:	4TH Corner Residential Project	Project No.:	11534.001
Proj. Address:	4021, 4029, 4035, 4061, 4089 Fairmount Ave, San Diego, California		

SOIL TYPE / TEST LOCATION / BOREHOLE

Soil Type: Fat CLAY (CL) - Normal Heights Mudstone (Qlnh) Hole: P-2

Location: See Figure 2

Hole Dia: 8"

Depth: 4 Feet (bgs)

Tested by: RNB Pre-Saturation Date: 1-12-17 Test Date: 1-13-17

Notes:

Time of Day	Interval / Notes	Initial Depth to Water (in.)	Final Depth of Water (in.)	Δ in Water Level (in.)	Percolation Rate (min/inch)
8:15	Start	36.00	-	-	-
8:45	30	36.00	36.09	0.09	333
9:15	30	36.09	36.18	0.09	333
9:45	60	36.18	36.24	0.06	1000
10:45	60	36.24	36.31	0.07	857
11:45	60	36.31	36.39	0.08	750
12:45	60	36.39	36.45	0.06	1000
1:45	60	36.45	36.51	0.06	1000
2:45	60/Fill H20	36.00	36.05	0.05	1200

Notes: Last 60 Minute Reading Used to Determine Field Percolation Rate

Percolation Rate = (1200 min/inch)

Leighton

FIELD PERCOLATION TEST DATA SHEET

Project Name:	4TH Corner Residential Project	Project No.:	11534.001
Proj. Address:	4021, 4029, 4035, 4061, 4089 Fairmount Ave, San Diego, California		

SOIL TYPE / TEST LOCATION / BOREHOLE

Soil Type: Fat CLAY (CL) - Normal Heights Mudstone (Qlnh) Hole: P-3

Location: See Figure 2

Hole Dia: 8"

Depth: 4 Feet (bgs)

Tested by: RNB Pre-Saturation Date: 1-25-17 Test Date: 1-26-17

Notes:

Time of Day	Interval / Notes	Initial Depth to Water (in.)	Final Depth of Water (in.)	Δ in Water Level (in.)	Percolation Rate (min/inch)
8:30	Start	36.00	-	-	-
9:00	30	36.00	36.08	0.08	375
9:30	30	36.08	36.15	0.07	429
10:00	60	36.15	36.23	0.08	750
11:00	60	36.23	36.28	0.05	1200
12:00	60/Fill H20	36.00	36.04	0.04	1500
1:00	60	36.04	36.09	0.05	1200
2:00	60	36.09	36.13	0.04	1500
3:00	60/Fill H20	36.00	36.04	0.04	1500

Notes: Last 60 Minute Reading Used to Determine Field Percolation Rate

Percolation Rate = (1500 min/inch)

Appendix C
Laboratory Testing Procedures and Test Results

APPENDIX C

Laboratory Testing Procedures and Test Results

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
B-3 @ 1 to 5 feet	Lean CLAY (CL)	>130	Very High

Atterberg Limits: The Atterberg Limits were determined in accordance with ASTM Test Method D4318 for engineering classification of a representative fine-grained material and the results are presented in the table below:

Boring	Depth (feet)	Plasticity Index	Liquid Limit	Plastic Limit	USCS Soil Classification
B-3	1-5	32	45	13	CL

Soluble Sulfates: The soluble sulfate content of a selected sample was determined by standard geochemical methods (Caltrans Test Method CT417). The test result is presented in the table below:

Sample Location	Sulfate Content (%)	Potential Degree of Sulfate Attack*
B-3 @ 1 to 5 feet	0.050	Negligible

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

APPENDIX C (Continued)

Chloride Content: Chloride content was tested in accordance with DOT Test Method No. 422. The results are presented below:

Sample Location	Chloride Content, ppm
B-3 @ 1 to 5 feet	348

Minimum Resistivity and pH Tests: Minimum resistivity and pH tests were performed in general accordance with California Test Method 643. The results are presented in the table below:

Sample Location	pH	Minimum Resistivity (ohms-cm)
B-3 @ 1 to 5 feet	6.7	375

Appendix D
City of San Diego Infiltration Worksheet C.4-1

Categorization of Infiltration Feasibility Condition		Worksheet C.4-1	
Part 1 - Full Infiltration Feasibility Screening Criteria 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: Based on our field percolation testing, the in-situ infiltration rates of the soils at the subject site are less than 0.5 inches per hour (Leighton, 2017). Specifically, the calculated infiltration rate via the Porchet Method and applied safety factor of 2 is less than 0.01 inches per hour across the site and therefore the site is considered appropriate for a "No-Infiltration" designation. 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: 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) and undocumented fill depths greater than 5 feet within the vicinity of the proposed infiltration site. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			

Worksheet C.4-1 Page 2 of 4			
Criteria	Screening Question	Yes	No
3	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.	X	
Provide basis: 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, groundwater depths are anticipated to be greater than 50 feet bgs. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
4	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.	X	
Provide basis: 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. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
Part 1 Result*	If all answers to rows 1 - 4 are "Yes" a full infiltration design is potentially feasible. The feasibility screening category is Full Infiltration 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	Go to Part 2	

*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 City Engineer to substantiate findings.

<table>
<tr><th colspan="4">Worksheet C.4-1 Page 3 of 4</th></tr>
<tr><td colspan="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?</td></tr>
<tr><td>Criteria</td><td>Screening Question</td><td>Yes</td><td>No</td></tr>
<tr><td>5</td><td>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.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis:

Based on our field percolation testing, the in-situ infiltration rates of the soils at the subject site are less than 0.5 inches per hour (Leighton, 2017). Specifically, the calculated infiltration rate via the Porchet Method and applied safety factor of 2 is less than 0.01 inches per hour across the site and therefore the site is considered appropriate for a “No-Infiltration” designation.

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>6</td><td>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.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:

If partial infiltration conditions (greater than 0.01 inches per hour) existed across the site, it may be possible that the risk of geotechnical hazards will not be increased by partial infiltration provided mitigation is performed for any underground utilities/structures, slopes (i.e., setbacks) and undocumented fill depths greater than 5 feet within the vicinity of the proposed infiltration site. Mitigation includes subsurface vertical barriers and subdrains to limit perched ground water mounding 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.</td></tr>
</table>

Worksheet C.4-1 Page 4 of 4			
Criteria	Screening Question	Yes	No
7	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.	X	
Provide basis: If partial infiltration conditions (greater than 0.01 inches per hour) existed across the site, it may be possible that 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, groundwater depths are anticipated to be greater than 50 feet bgs. 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.			
8	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.	X	
Provide basis: If partial infiltration conditions (greater than 0.01 inches per hour) existed across the site, 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. 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.			
Part 2 Result*	If all answers from row 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration. 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”.	“No-Infiltration” feasibility	

*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 City Engineer 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.

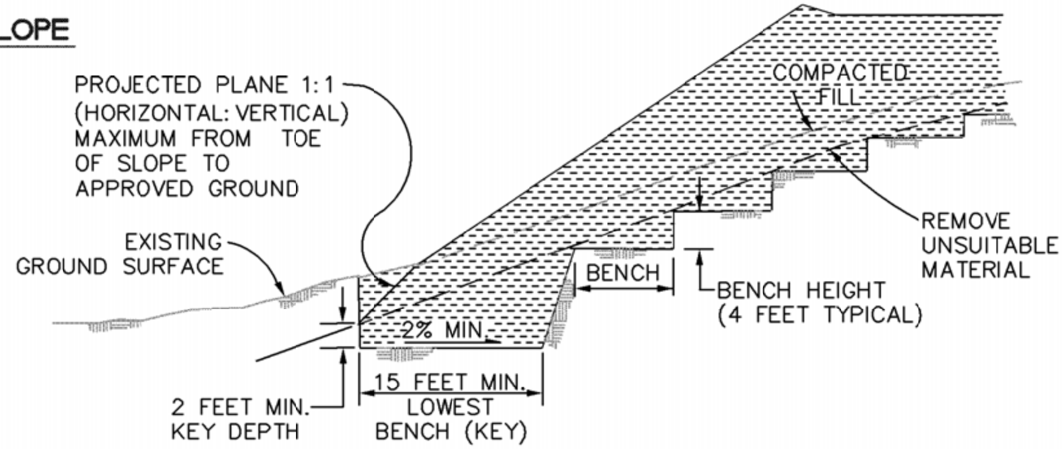
7.3 Lift Thickness

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.

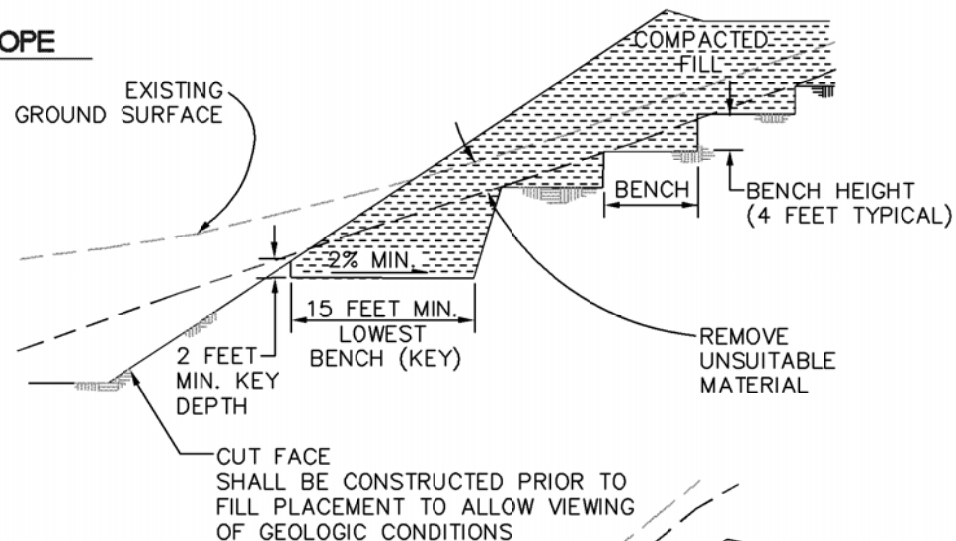
7.4 Observation and Testing

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

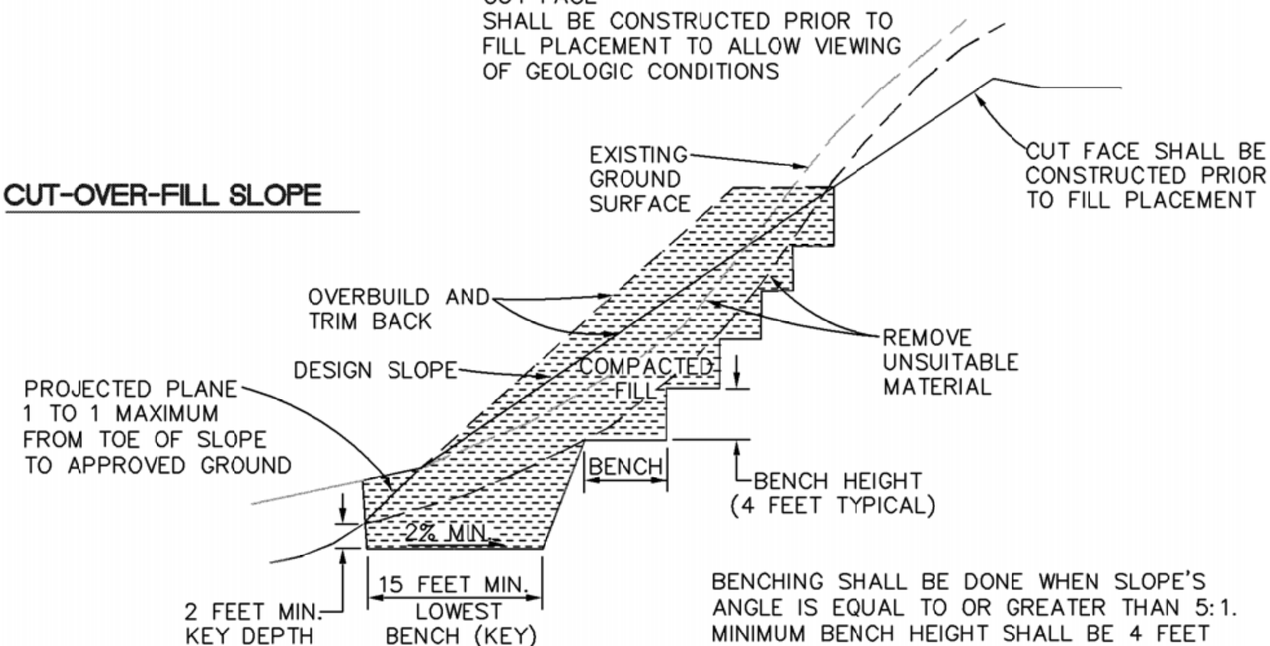
FILL SLOPE



FILL-OVER-CUT SLOPE

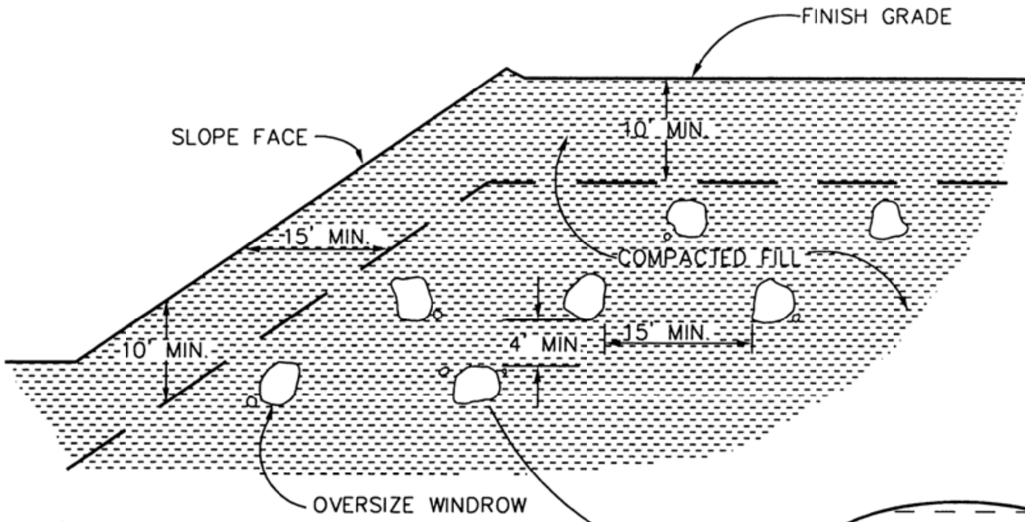


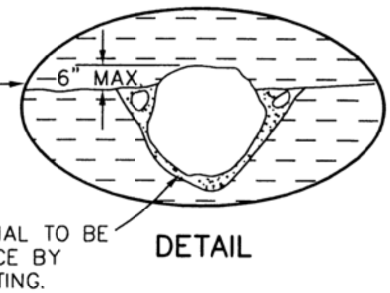
CUT-OVER-FILL SLOPE



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.

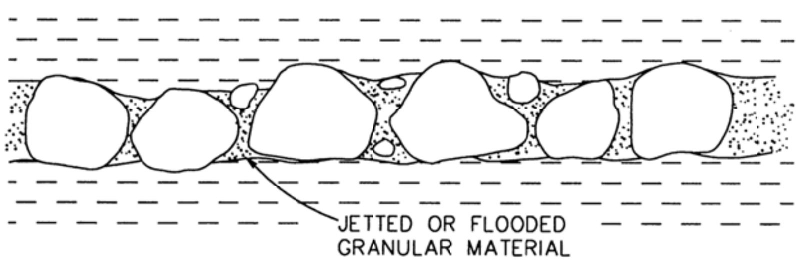
KEYING AND BENCHING	GENERAL EART WORK AND GRADING SPECIFICATIONS STANDARD DETAIL A





DETAIL

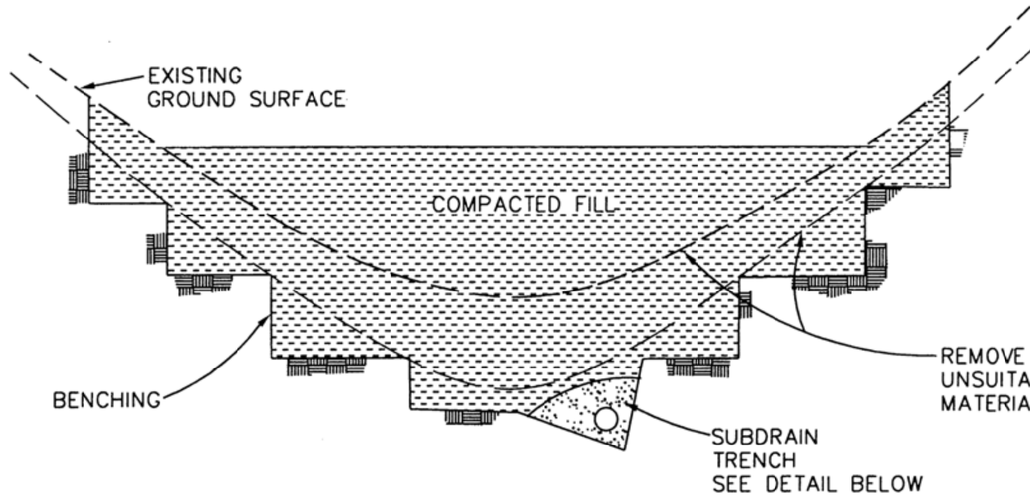
* OVERSIZE ROCK IS LARGER THAN 8 INCHES IN LARGEST DIMENSION.
* EXCAVATE A TRENCH IN THE COMPACTED FILL DEEP ENOUGH TO BURY ALL THE ROCK.
* BACKFILL WITH GRANULAR SOIL JETTED OR FLOODED IN PLACE TO FILL ALL THE VOIDS.
* DO NOT BURY ROCK WITHIN 10 FEET OF FINISH GRADE.
* WINDROW OF BURIED ROCK SHALL BE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

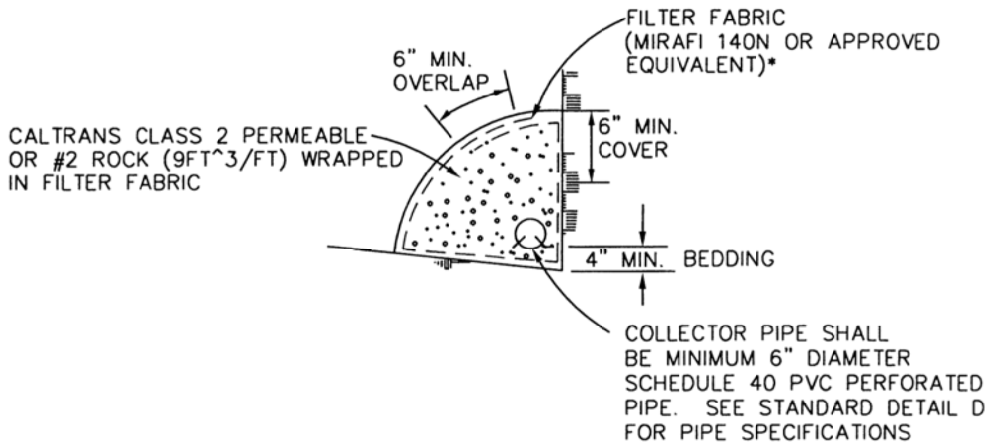


TYPICAL PROFILE ALONG WINDROW

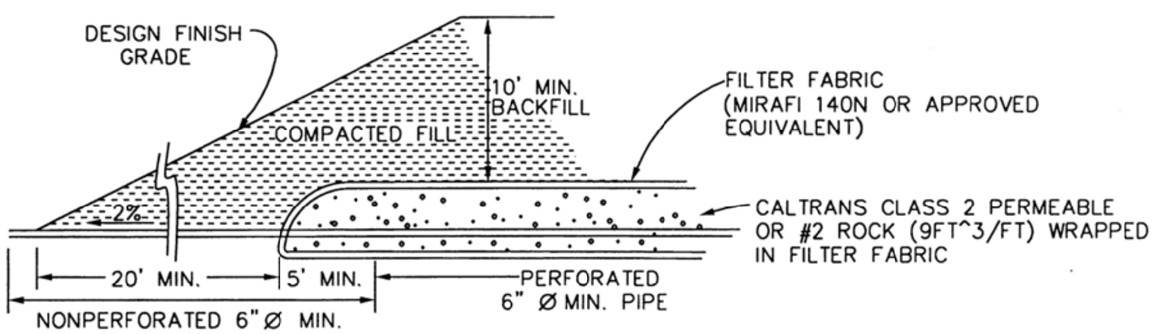
OVERSIZE ROCK DISPOSAL

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL B



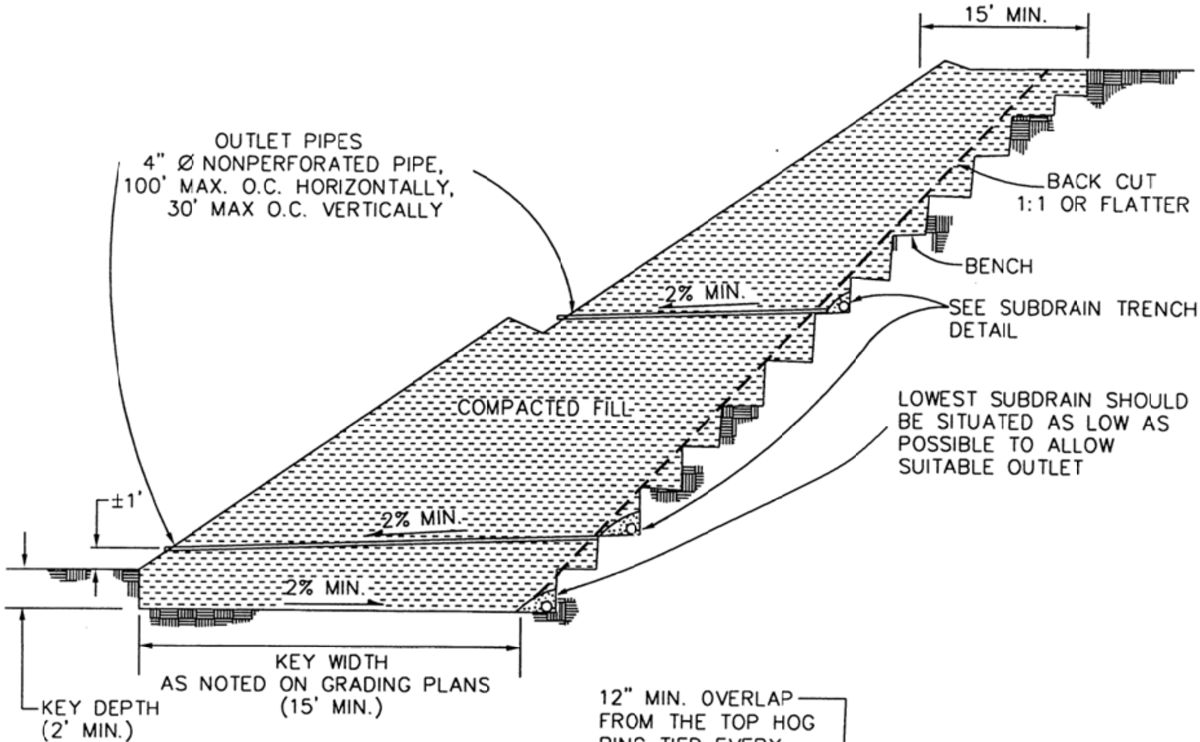


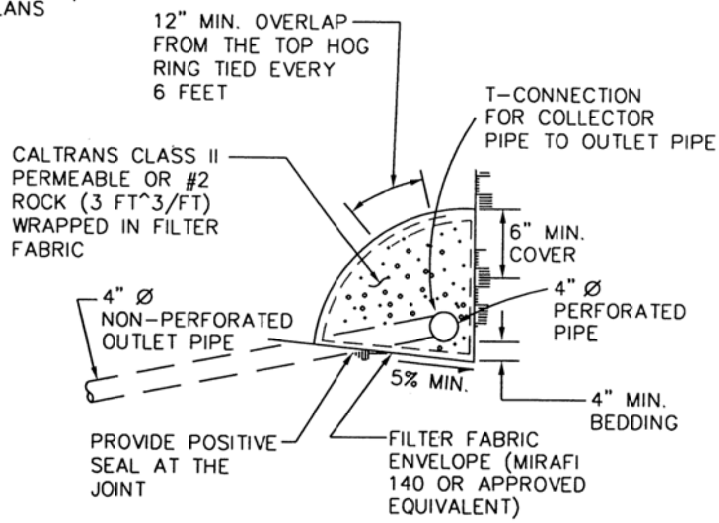
SUBDRAIN DETAIL



DETAIL OF CANYON SUBDRAIN OUTLET

CANYON SUBDRAINS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS 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.

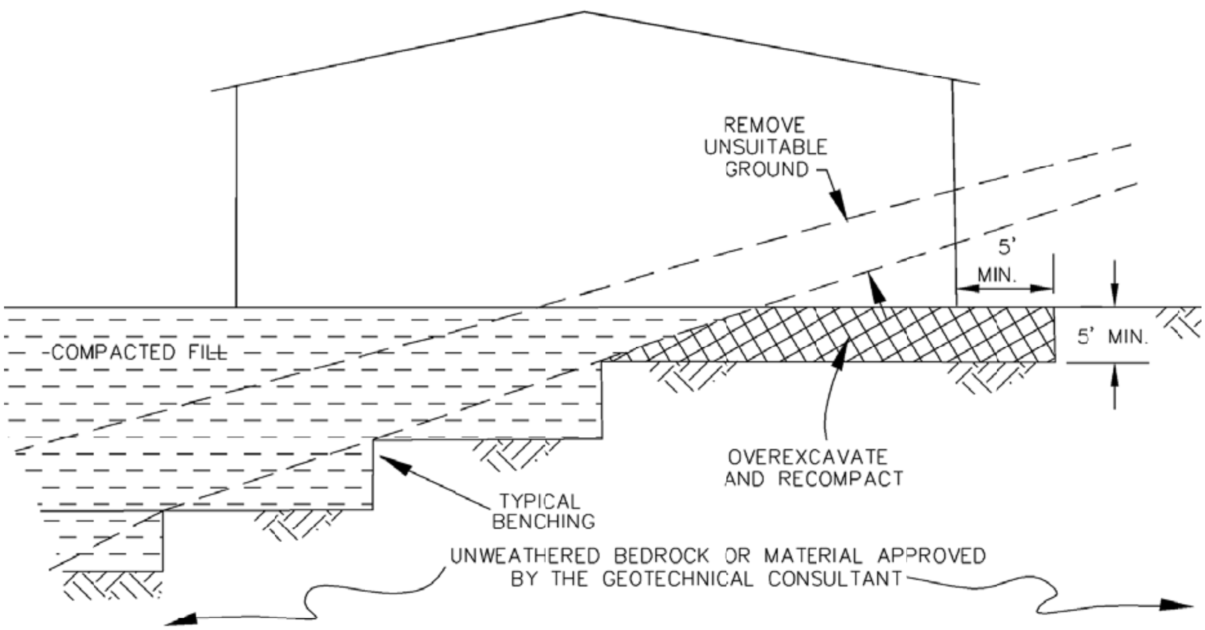
SUBDRAIN PIPE – Subdrain pipe shall be ASTM D2751, SDR 23.5 or ASTM D1527, Schedule 40, or ASTM D3034, SDR 23.5, Schedule 40 Polyvinyl Chloride Plastic (PVC) pipe.

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

BUTTRESS OR REPLACEMENT FILL SUBDRAINS

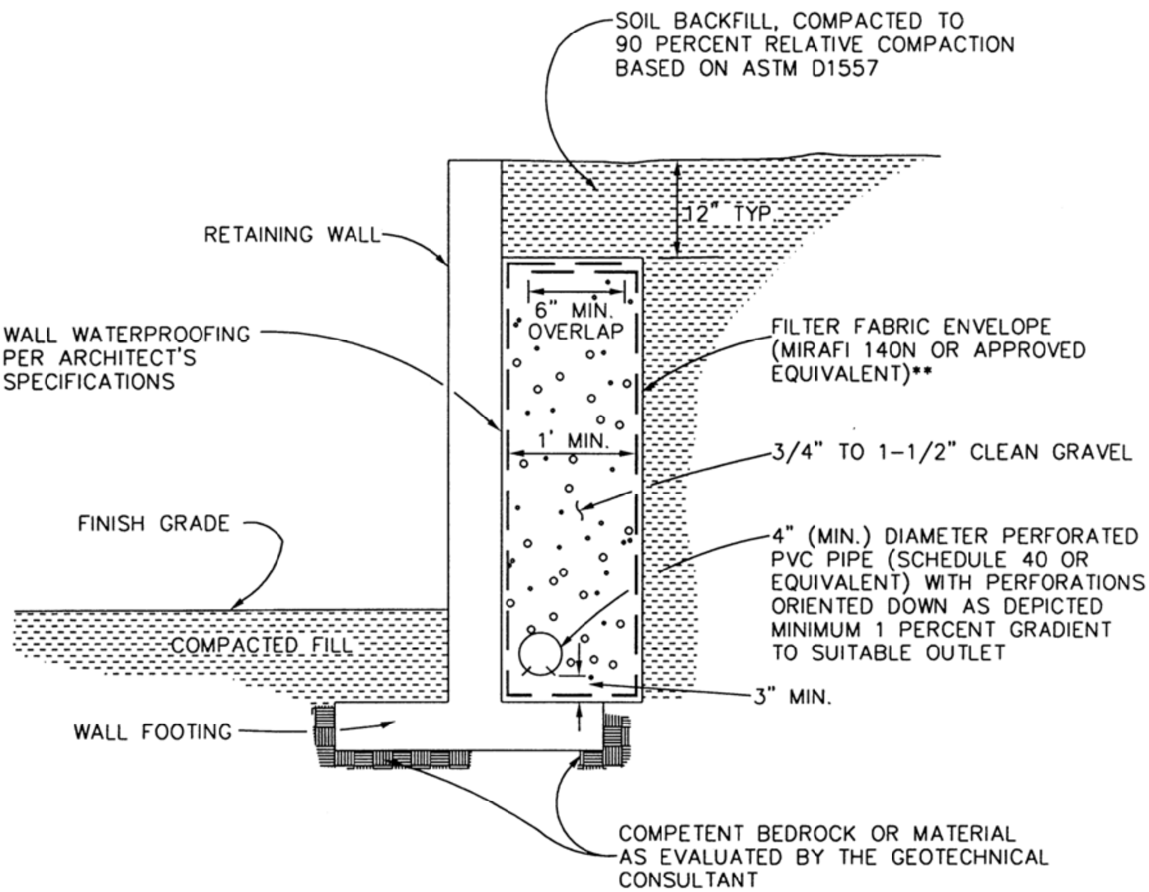
GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL D

CUT–FILL TRANSITION LOT OVEREXCAVATION



TRANSITION LOT FILLS

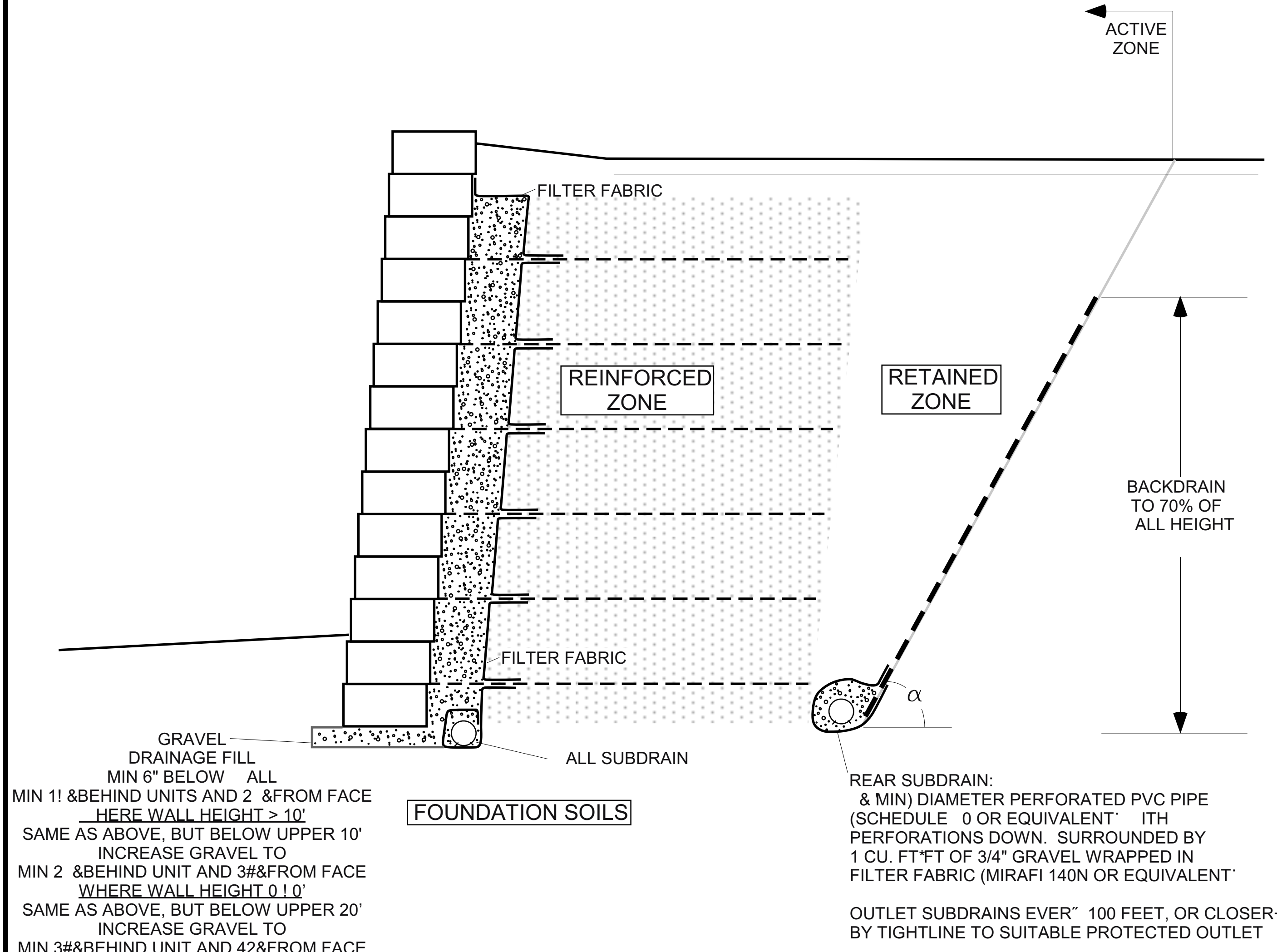
GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL E



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 ALL DRAINAGE

GENERAL EART ORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL F



NOTES˘

ˆ˙ MATERIAL GRADATION AND PLASTICIT˝

REINFORCED ZONE:

SIEVE SIZE	% PASSING
ˆ INCH	100
NO. 4	20-100
NO. 40	0-60
NO. 200	0-$%

FOR WALL HEIGHT / 10 FEET, PLASTICIT˝ INDEX / 20 AND LIQUID LIMIT / 0
FOR WALL HEIGHT 10 FEET OR TALLER, PLASTICIT˝ INDEX / #
FOR TIERED WALLS, USE COMBINED WALL HEIGHTS
FOR WALL HEIGHT > 20 FEET, REDUCE ALLOWABLE RANGE % PASSING NO. 200 SIEVE TO 0-15

GRAVEL DRAINAGE FILL:

SIEVE SIZE	% PASSING
ˆ INCH	100
$* INCH	75-100
NO. 4	0-60
NO. 40	0-50
NO. 200	0-%

! ˙ CONTRACTOR TO USE SOILS WITHIN THE RETAINED AND REINFORCED ZONES THAT MEET THE STRENGTH AND UNIT EIGHT REQUIREMENTS OF WALL DESIGN.

$˙ GEOGRID REINFORCEMENT TO BE DESIGNED BY WALL DESIGNER CONSIDERING INTERNAL+EXTERNAL+AND COMPOUND STABILIT˝˝

$˙ GEOGRID TO BE PRETENSIONED DURING INSTALLATION.

) IMPROVEMENTS ITHIN THE ACTIVE ZONE ARE SUSCEPTIBLE TO POST-CONSTRUCTION SETTLEMENT. ANGLE α = %- Φ*!+ HERE Φ IS THE FRICTION ANGLE OF THE MATERIAL IN THE RETAINED ZONE.

%˙ 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 ALLS

GENERAL EARTH ORK AND GRADING SPECIFICATIONS
STANDARD DETAIL G

REVISED 11/16

Appendix D

Seismic Survey

Subsurface Surveys & Associates, Inc.
2075 Corte Del Nogal, Suite W   Carlsbad, CA 92011
Phone: (760) 476-0492   Fax: (760) 476-0493

Leighton and Associates, Inc.
3934 Murphy Canyon Road, B205
San Diego, CA 92123

July 16, 2020

Attn: Mike Jensen

Re: Seismic Survey Summary Report
Wakeland Union Residential Project

Subsurface Surveys has completed a seismic shear wave survey at 4035 Fairmount Ave in City Heights, California. The main objective was to measure the shear wave velocity (Vs) of soil and bedrock to a depth of 100 feet, if possible. This information is to be used for soil classification and engineering design.

The field work was conducted on July 8, 2020. One traverse was recorded at a location selected by Leighton. A survey location map is provided on Figure 1 that shows the position and orientation of the traverse.

GEOLOGIC SETTING

A review of the "Geologic Map of the Oceanside 30' x 60' quadrangle", (Department of Conservation, 2005) indicates the survey area is underlain by Very old paralic deposits, Unit 8 which are middle to early Pleistocene age. The deposits are composed mainly of siltstone, sandstone and conglomerate.

SEISMIC METHODS

MASW – The MASW (Multi-channel Analysis of Surface Waves) seismic method uses low frequency surface waves, commonly referred to as "ground roll", to extract shear wave (S-wave) velocity data verses depth in the subsurface. This information is commonly used for soil classification, tower and foundation design, and seismic response spectra calculations.

This method does not measure shear wave velocity directly. Rather, dispersion curves are extracted from the raw multi-channel field records and inverted to produce a one-dimensional S-wave velocity profile.

MAM - The Microtremor Analytical Method uses ambient low frequency seismic noise (vibrations) generated mostly by surrounding car, truck, and bus traffic. This approach also uses the induced ground roll to extract shear wave velocity.

EQUIPMENT AND FIELD PROCEDURES

Seismic data were recorded with a Seistronix RAS-24 digital seismograph and a 24 channel cable system with 4.5 Hz vertical geophones placed at 10 foot intervals. Three shotpoints were used, two off-end (15-foot offset) and one at the middle of the spread.. Energy was generated by sledge hammer impacts on a metal plate. Each record was made by stacking 2 to 3 hammer hits. A recording length of 1000 milliseconds was used to completely envelop the surface waves.

The MAM survey used the same layout and configuration, but recorded 20 separate random recordings, each 32 seconds in length. This ensured that seismic waves were sampled from a variety of different directions, a requirement for this technique.

DATA REDUCTION AND VELOCITY DETERMINATION

Surface wave records were processed with SeisImager/SW software from Geometrics Inc. The software calculates shear wave velocity by mathematical inversion of the dispersive phase velocity of surface waves. In this application, Raleigh waves (also referred to as "ground roll") are the main surface waves of interest. A summary of the steps involved is provided below.

Standard time-distance field records are converted from time domain to frequency domain using a phase velocity- frequency transformation. This yields dispersion curve plots of frequency (Hz) verses phase velocity in (ft/sec). A wave equation module then performs inversion modeling to produce graphs of shear wave velocity verse depth. The resultant Vs curve is used to calculate the Uniform Building Code (UBC) Vs100 ft (30 m) value for soil classification. Vs100 ft represents the average shear wave velocity between 0-100 feet depth beneath the traverse.

SUMMARY OF RESULTS

Inversion modeling results are displayed on x-y graphs of shear wave velocity verses depth (see Figures 2 and 3).

Results indicate the MASW records were significantly affected by persistent traffic noise from Fairmount Ave. This masked the lower amplitude surface wave arrivals towards the far end of the line and limited the survey depth to about 70 feet.

However, the steady background of traffic noise from all directions enhanced the MAM data set, both in quality and depth. The modeling results on Figure 3 show shear wave velocity to a depth of 150 feet.

The calculated Vs100 ft values are provided below:

MASW extrapolated Vs 100 ft = 1409.4 ft/s
MAM Vs 100 ft = 1411.5 ft/s

This velocity is within the Class C soil range of 1200-2500 ft/sec using the Uniform Building Code (1997) guidelines.

All data acquired during this survey is considered confidential and is available for review by your staff at any time. We appreciate the opportunity to participate in this project.

Please call if there are any questions.

PAWalen

Phillip A. Walen
Senior Geophysicist
CA Registration No. GP917

Seismic Survey Location Map

4035 Fairmount Ave, San Diego, California

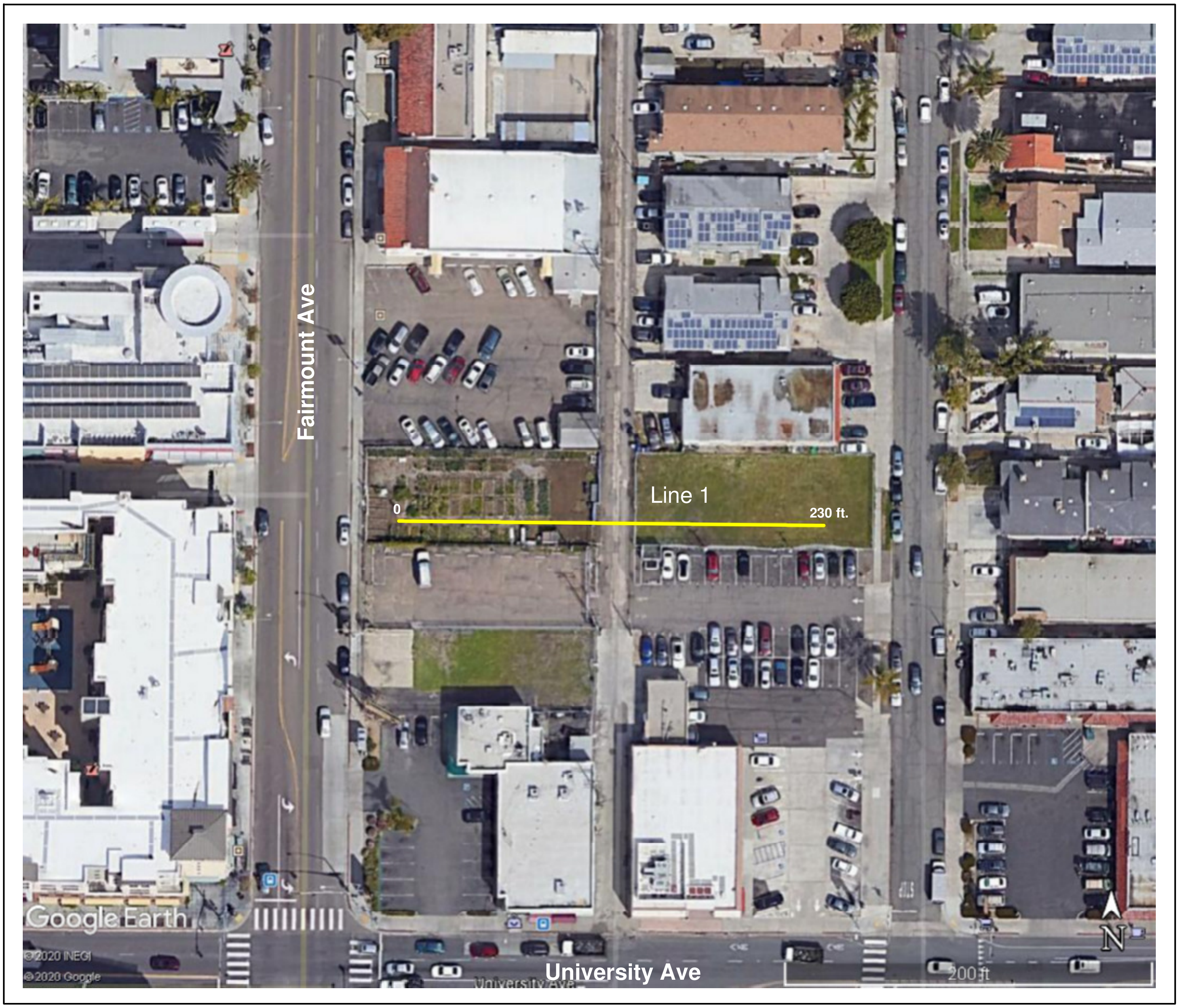


Figure 1

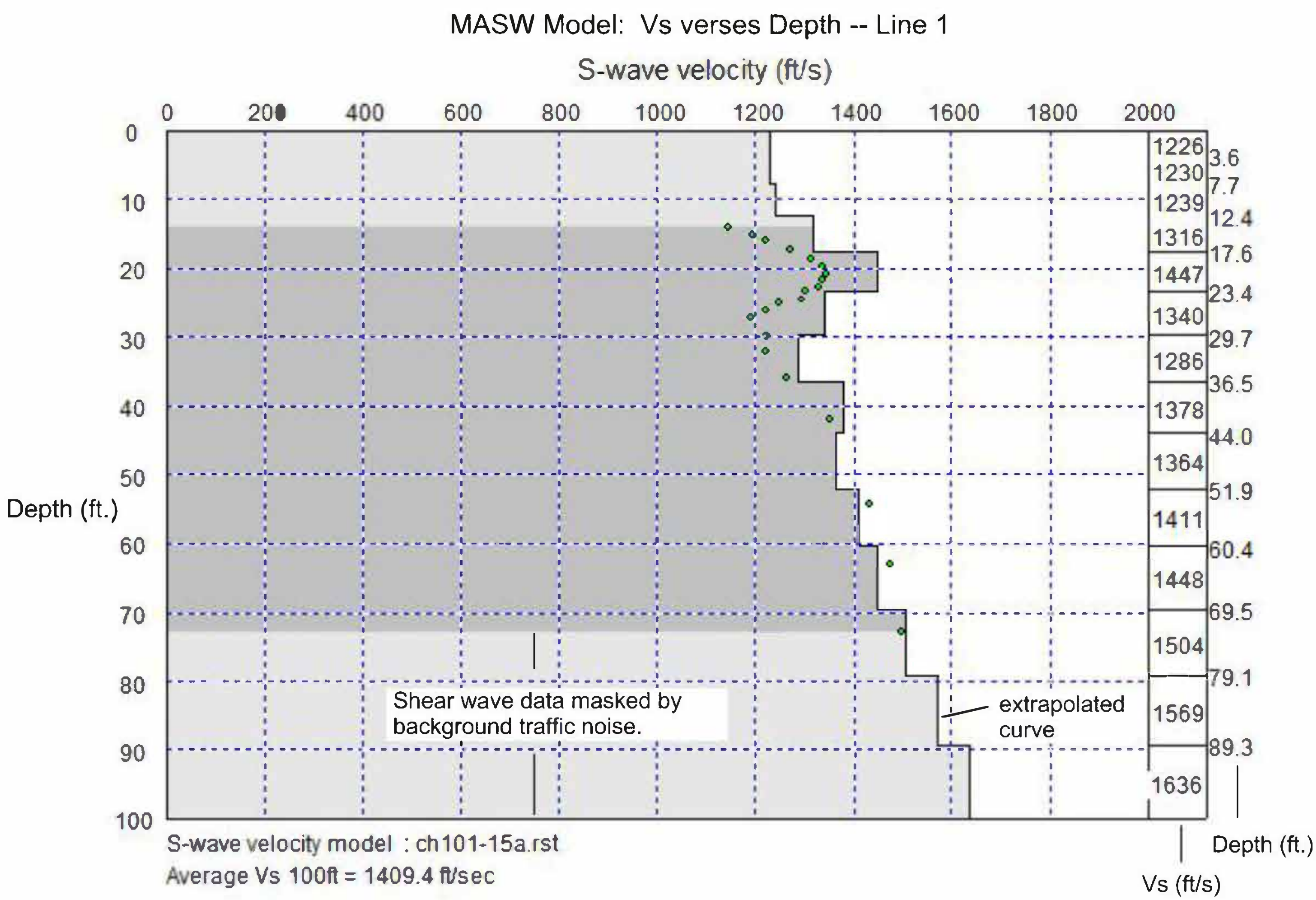

MASW Model: Vs verses Depth -- Line 1
S-wave velocity (ft/s)
0
200
400
600
800
1000
1200
1400
1600
1800
2000
Depth (ft.)
0
10
20
30
40
50
60
70
80
90
100
1226
1230
1239
1316
1447
1340
1286
1378
1364
1411
1448
1504
1569
1636
3.6
7.7
12.4
17.6
23.4
29.7
36.5
44.0
51.9
60.4
69.5
79.1
89.3
Shear wave data masked by background traffic noise.
extrapolated curve
S-wave velocity model : ch101-15a.rst
Average Vs 100ft = 1409.4 ft/sec
Depth (ft.)
Vs (ft/s)

Figure 2

MAM Model: Vs verses Depth -- Line 1

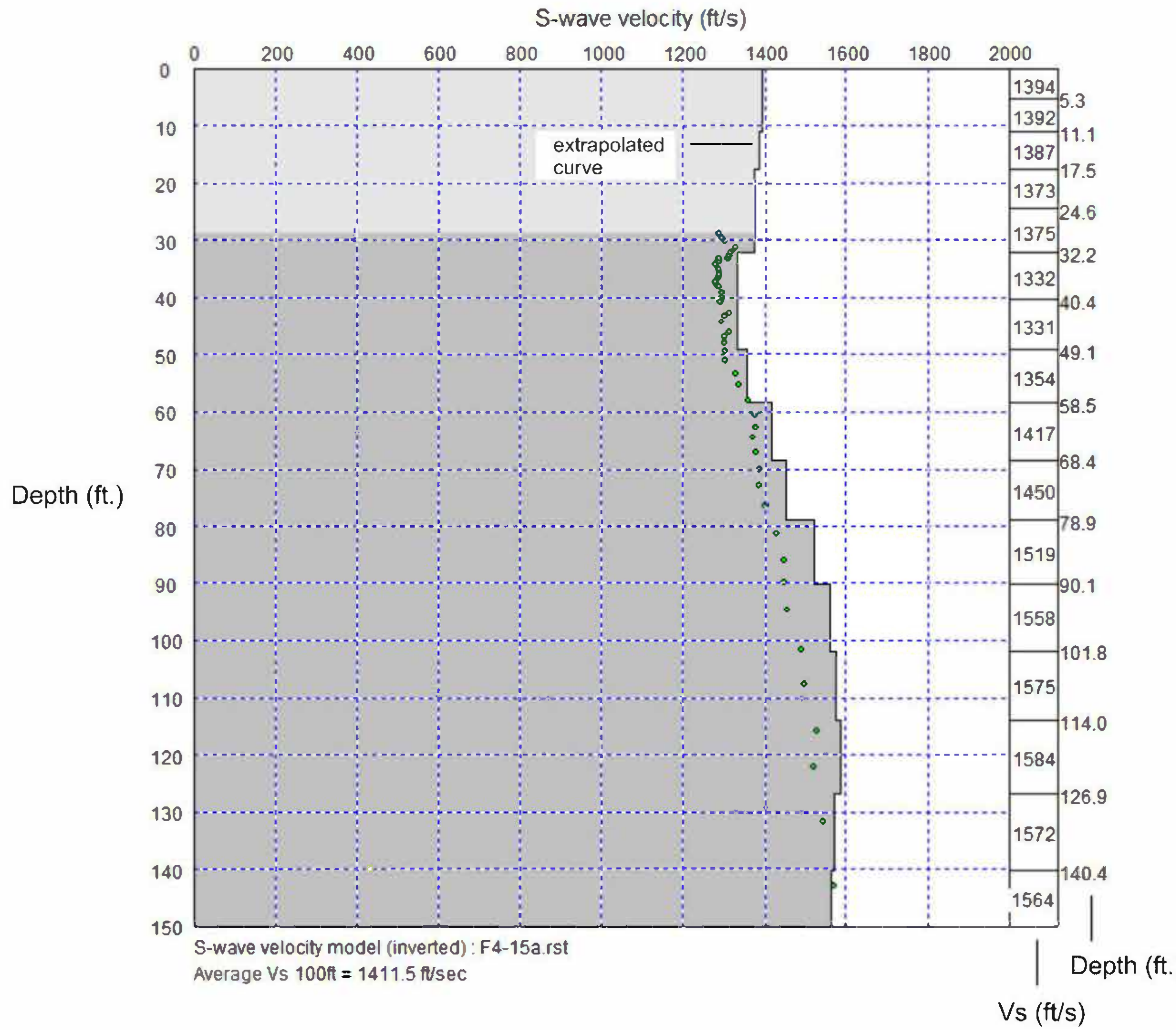


Figure 3