ATTENTION:

These geotechnical documents have been reviewed by BDR-Geology and have a green "Geology Record Document" stamp on each document. These documents with the green "Geology Record Document" stamp MUST be submitted at permit issuance. Your permit will not be issued without these stamped documents.

If these record documents are lost, misplaced, or recycled, original quality replacement copies of all documents previously reviewed by BDR-Geology will need to be submitted with a complete building plan set for additional BDR-Geology review which may delay permit issuance and incur additional fees.

Note that Geology Record Documents routed to plan pickup may be recycled if not picked up within 30 days.

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

CITY OF SAN DIEGO
GEOLOGY
RECORD DOCUMENT
PTS PROJECT NO.: 644432
DATE: 12/6/19

August 28, 2019

Project No. 11931.002

Aaron Magagna
3639 Midway Drive, Suite B #132
San Diego, California 92110

Attention: Mr. Aaron Magagna

Subject: Geotechnical Response to City of San Diego
Cycle 7 BDR-Geology; L64A-003A
Federal Blvd M.O.
City of San Diego, PTS No. 644432

As requested, we have prepared a response to the City of San Diego review comments for the proposed Federal Blvd M.O. Project located northeast of the intersection of Federal Blvd and Winnett Street in the City of San Diego, California. The San Diego County Assessor's Office designates the subject property as Assessor Parcel Number (APN) 543-020-04-00. The comments include 7 BDR-Geology Comments, PTS No. 644432, dated August 20, 2019. For clarity, the City of San Diego comments are italicized and numbered in accordance with the order presented on the comment sheet. It should also be noted that comments addressed below are specific to the geologic aspects of the project and other comments for other disciplines are not addressed in this letter.

Cycle 3 BDR-Geology Comments, PTS No. 607976

2 *Submit an addendum geotechnical report or update letter that specifically addresses the current building plan set and the following:*

Response

Please consider this response to comments as an addendum to the referenced report. Based on our review of the most recent site plans (PDC, 2019), we understand that the project now includes the following changes since the submittal of our referenced geotechnical report:

- The proposed commercial building will consist of a slab-on-grade two-story building which will be built with typical wood frame construction.

3 *Submit digital copies (on CD/DVD/or USB data storage device) of the referenced and requested geotechnical reports for our records.*

Response

The referenced and requested geotechnical reports have been put on a USB data storage device and will be to the City of San Diego Geology Department for their records.

4 *Provide an updated/geotechnical map that shows the distribution of fill and geologic units, location of exploratory excavations, and current proposed construction on a topographic base map.*

Response

See attached Figure 2 (Geotechnical Map).

5 *The project's geotechnical consultant must review the building plans and indicate if the plans are in accordance with their recommendations. Please provide additional analysis and/or recommendations if necessary.*

Response

A review of the project building plan set, including the structural/foundation plans has been completed. The review letter is attached at the rear of this response.

6 *NOTE – Strom Water Requirements for the proposed project will be evaluated by LDR-Engineering Building review. Priority Development Projects (PDPs) may require an investigation of storm water infiltration feasibility in accordance with the Storm Water Standards (including Appendix C and D). Check with your LDR-Engineering Building reviewer on requirements. LDR-Engineering Building may determine that BDR-Geology review of a storm water infiltration evaluation is required.*

<u>Response</u>

We understand that Strom Water Requirements for the proposed project will be evaluated by LDR-Engineering Building review. In addition, we have performed an investigation of storm water infiltration feasibility in accordance with the Storm Water Standards (including Appendix C and D). The results of this investigation can be found in the referenced project geotechnical report.

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 CONSULTING, INC.

PROFESSIONAL GEOLOGIST
ROY N. BUTZ
NO. 8942
3-31-21
STATE OF CALIFORNIA

Roy N. Butz, PG 8942
Senior Project Geologist

Attachments: Appendix A – References
Appendix B – Plan Review Letter
Figure 2 – Geotechnical Map

Distribution: (1) Addressee via email

APPENDIX A
REFERENCES

City of San Diego, Cycle 7 BDR-Geology Comments, PTS No. 644432, dated August 20, 2019.

Leighton Consulting, Inc., 2018, Geotechnical Investigation, Federal Blvd Retail Building, Assessor Parcel Number 543-020-04-00, San Diego, California 92114, Project No. 11931.001, dated March 9, 2018.

Pacific Design Concepts, LLC, 2019, Building Plans, Federal Blvd Marijuana Outlet, dated July 21, 2019.

Projection Engineering, Inc., 2019, Grading Plans, Federal Blvd Marijuana Outlet, dated July 24, 2018.

TJ Engineering, 2019, Structural/Foundation Plans, Federal Blvd Marijuana Outlet, dated July 21, 2019.

APPENDIX B
Plan Review Letter

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

August 28, 2019

Project No. 11931.002

Aaron Magagna
3639 Midway Drive, Suite B #132
San Diego, California 92110

Attention: Mr. Aaron Magagna

Subject: Plan and Specification Review
Federal Blvd M.O.
(APN) 543-020-04-00
San Diego, California

In accordance with your request, we have performed a geotechnical review of the referenced Federal Blvd M.O. Project plans and specifications. It should be noted that our review was limited to the geotechnical aspects of the project and was performed to identify potential conflicts with the intent of the referenced project geotechnical document (Leighton, 2018). Based on our review, it is our professional opinion that the plans and specifications were prepared in substantial conformance with the geotechnical document.

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

Respectfully submitted,

LEIGHTON CONSULTING, INC.

PROFESSIONAL GEOLOGIST
ROY N. BUTZ
NO. 8942
3-31-21
STATE OF CALIFORNIA

Roy N. Butz, PG 8942
Senior Project Geologist

Distribution: Addressee via email

Attachments: Appendix A – References

<u>Appendix A</u>

References

Leighton Consulting, Inc., 2018, Geotechnical Investigation, Federal Blvd Retail Building, Assessor Parcel Number 543-020-04-00, San Diego, California 92114, Project No. 11931.001, dated March 9, 2018.

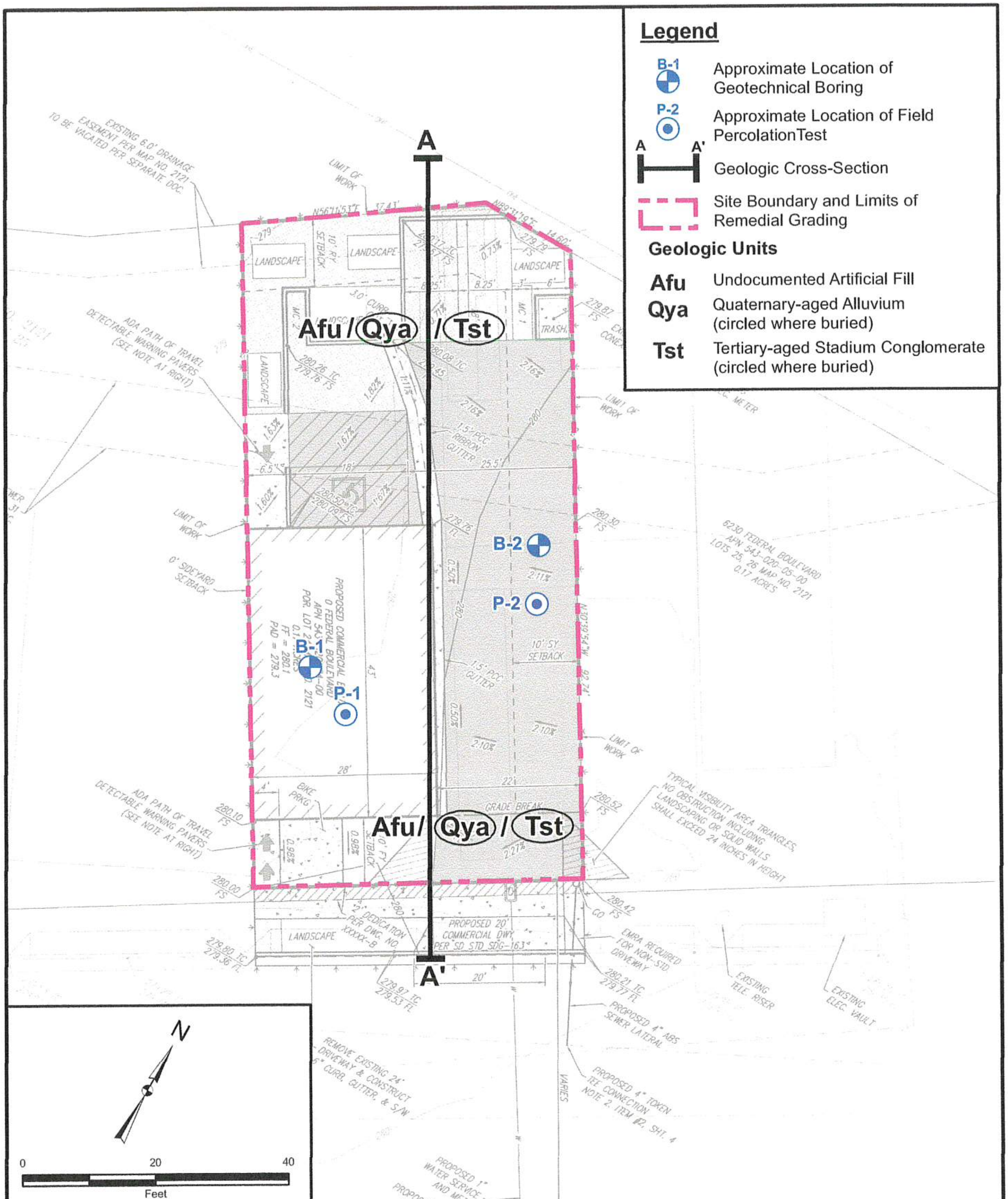
Pacific Design Concepts, LLC, 2019, Building Plans, Federal Blvd Marijuana Outlet, dated July 21, 2019.

Projection Engineering, Inc., 2019, Grading Plans, Federal Blvd Marijuana Outlet, dated July 24, 2018.

TJ Engineering, 2019, Structural/Foundation Plans, Federal Blvd Marijuana Outlet, dated July 21, 2019.

FIGURE 2

Geotechnical Map



Project: 11931.002	Eng/Geol: SAC/RNB/MDJ
Scale: 1 " = 20 '	Date: August 2019

Base Map: Preliminary Grading Plan by Paul Fisher Engineering.

Author: Leighton Geomatics (mmurphy)

GEOTECHNICAL MAP

Federal Blvd M.O.
San Diego, California

Figure 2

Leighton

Map Saved as V:\Drafting\11931\002\Maps\11931-002_F02_GM_2019-08-28.mxd on 8/28/2019 9:20:43 AM

GEOTECHNICAL INVESTIGATION
FEDERAL BLVD RETAIL BUILDING
ASSESSOR PARCEL NUMBER 543-020-04-00
SAN DIEGO, CALIFORNIA 92114

Prepared for:

Aaron Magagna

3639 Midway Drive, Suite B #132
San Diego, California 92110

CITY OF SAN DIEGO
GEOLOGY
RECORD DOCUMENT
PTS PROJECT NO.: 644432
DATE: 12/6/19

Project No. 11931.001

March 9, 2018

Leighton Consulting, Inc.

A LEIGHTON GROUP COMPANY

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

March 9, 2018

Project No. 11931.001

Aaron Magagna
3639 Midway Drive, Suite B #132
San Diego, California 92110

Attention: Mr. Aaron Magagna

Subject: Geotechnical Investigation
Federal Blvd Retail Building
APN: 543-020-04-00
San Diego, California 92114

In accordance with your request and authorization, we have conducted a geotechnical investigation of the subject property for the design and construction of a proposed two-story commercial development and associated improvements. The accompanying report presents a summary of our current investigation and provides geotechnical conclusions and recommendations relative to the proposed site development.

Based on the results of our current geotechnical study, it is our professional opinion that the site is suitable for the proposed improvements provided the recommendations contained in this report are implemented during design and construction. The accompanying report presents a summary of our current exploration 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 CONSULTING, INC.

REGISTERED PROFESSIONAL ENGINEER
SEAN A. COLORADO
No. 2507
Exp. 12/31/19
GEOTECHNICAL
STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST
ROY N. BUTZ
NO. 8942
3-31-19
STATE OF CALIFORNIA

Sean Colorado, GE 2507
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Roy Butz, PG 8942
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Extension: 8489, rbutz@leightongroup.com

PROFESSIONAL GEOLOGIST
MICHAEL D. JENSEN
NO. 2457
CERTIFIED ENGINEERING GEOLOGIST
Exp. 7/31/18
STATE OF CALIFORNIA

Mike Jensen, CEG 2457
Senior Project Geologist
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Distribution: (1) Addressee via e-mail
(3) Strom Entitlement-Permitting Project Management, LLC

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Leighton

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Important Information about This

Geotechnical-Engineering Report

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

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

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

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

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

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer about Change

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

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

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

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

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

This Report May Not Be Reliable

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

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

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

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

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

This Report's Recommendations Are Confirmation-Dependent

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

This Report Could Be Misinterpreted

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

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

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

Give Constructors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

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

GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

1.0 INTRODUCTION

We recommend that all individuals utilizing this report read the preceding information sheet prepared by the Geoprofessional Business Association (GBA) 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 commercial development of the property located northeast of the intersection of Federal Blvd and Winnett Street in the City of San Diego, California 92114. The San Diego County Assessor's Office designates the subject property as Assessor Parcel Number (APN) 543-020-04-00 (Figure 1). This report presents geotechnical conclusions and recommendations for the currently proposed project. Our scope of services included:

- Review of available pertinent, published and unpublished geotechnical literature and maps. References cited are listed in Appendix A;
- Field reconnaissance of the existing on-site geotechnical conditions;
- Subsurface exploration consisting of the excavation, logging, and sampling of two small-diameter hollow-stem borings. Additionally, two field percolation tests were performed at the site as part of the subsurface exploration. The exploration logs and field percolation test data sheets are presented in Appendix B. Approximate exploration and field percolation test locations are shown on the Geotechnical Map (Figure 2);
- Laboratory testing of representative soil samples obtained from the subsurface exploration program. Results of these tests are presented in Appendix C;
- Assessment of geologic hazards;
- Development of seismic design parameters based on the 2016 California Building Code (CBC);
- Compilation and analysis of the geotechnical data obtained from field exploration and laboratory testing; and
- Preparation of this report presenting our findings, conclusions, and recommendations with respect to the proposed design, site grading and general construction considerations.

1.2 Site Location and Description

The subject site is a nearly rectangular shaped parcel located directly northeast of the intersection of Federal Blvd and Winnett Street in the City of San Diego, California, and encompasses approximately 4,948 square feet of land (Figure 2, Geotechnical Map). In general, the site is bounded by Federal Blvd to the south and three separate commercial properties to the north, east, and west.

Currently, the site is undeveloped and used as a construction and equipment storage yard. In addition, the site is secured around its perimeter by a chain link security fence. Site topography is nearly level with a surface elevation of approximately 282 feet above mean sea level (msl).

Site Latitude and Longitude
32.72930° N
-117.06351° W

1.3 Proposed Development

Based on our review of the conceptual site plans (PDC, 2018), we understand that the project will consist of the design and construction of a two-story commercial building and associated improvements which will be used as a retail building. In addition, the commercial building will consist of one level of typical wood frame construction over a concrete podium which will be utilized for ground level parking. Improvements at the site will consist of associated driveways, utilities, landscape, and hardscape.

2.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

2.1 Subsurface Exploration

Our exploration consisted of excavating two 8-inch-diameter geotechnical borings (B-1 and B-2) to depths of between approximately 9 to 19½ feet below the existing ground surface (bgs). All geotechnical borings were drilled using a heavy-duty truck-mounted hollow-stem auger drill rig. Auger refusal was encountered within boring B-2 at depth of 9 feet bgs due to the presence of large cobbles located within the Alluvium. Additionally, two field percolation tests were performed at the site as part of the subsurface exploration. The field percolation test locations were advanced using a heavy-duty truck-mounted hollow-stem auger drill rig to a depth of 4 feet bgs. 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.

In-situ field percolation testing was performed on March 2, 2018 in general accordance with City of San Diego Storm Water Standards (City of San Diego, 2016), Section D.3.3.2. The level of introduced water in each field percolation test location was measured at 30 minute intervals using a water level sounder until readings where generally steady.

During the exploration operations, a geologist from our firm prepared geologic logs and collected soil samples for laboratory testing and evaluation. Disturbed standard penetration test (SPT) soil sampling using a 140-pound automatic-trip hammer free falling 30-inches was performed in accordance with ASTM D1586. However, it should be noted that gravels and cobbles impeded any significant penetration during sampling. After logging and field testing, the borings and field percolation test holes were backfilled with a mixture of soil cuttings. The boring logs and field percolation test data sheets are provided in Appendix B, and laboratory test results are included in Appendix C. In addition, geotechnical exploration locations and field percolation test locations are shown on Figure 2.

2.2 Laboratory Testing

Laboratory testing performed on representative soil samples obtained during the subsurface explorations included grain size analysis, expansion index, 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 coastal region 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 rock. The regional geologic setting is depicted on Figure 3.

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 surficial units of undocumented artificial fill materials overlying Quaternary-age Alluvium and the Tertiary-aged Stadium Conglomerate. The approximate areal distribution of the geologic units is depicted on the Geotechnical Map (Figure 2). The approximate vertical distribution of geologic units underlying the site are shown on geologic Cross-Section A-A' (Figure 4). A brief description of the geologic units encountered at the site is presented below. The geotechnical exploration logs with detailed soils descriptions are presented in Appendix B.

3.2.1 Undocumented Artificial Fill – Afu

Based on our subsurface exploration, undocumented fill soils were encountered in both geotechnical exploration locations with a thickness varying from approximately 3½ to 4 feet bgs. However, it should be noted that deeper fill may be encountered between exploration locations. As encountered during our subsurface exploration the fill soils generally consist of loose, dark brown, moist, clayey sand with trace gravel. An as-graded report was not available for our review, and it is assumed that no engineering observations of these fill soils were provided at the time of grading. Therefore, these fills are considered undocumented and may settle under the placement of additional fill and building loads.

3.2.2 Young Alluvium - Qya

Underlying the existing undocumented artificial fill soils, Quaternary-aged Alluvium was encountered in both of our geotechnical borings. During our drilling exploration, this material generally consisted of firm, moist, very dark brown, sandy clay and medium dense to dense, brown, moist, clayey gravel with regions of interbedded cobble. The cobble located throughout the Alluvium is 6 to 8 inches in diameter with isolated cobbles up to 1 foot in diameter should be anticipated. In addition, it should be noted that auger refusal was encountered on large cobbles at a depth of 9 feet bgs in boring B-2.

3.2.3 Stadium Conglomerate - Tst

Tertiary-aged Stadium Conglomerate was encountered underlying the Alluvium within boring B-1 at a depth of 11 feet bgs and extended to the maximum depth explored of 19½ feet bgs. The Stadium Conglomerate unit consists of very dense, yellowish brown, moist, cobble conglomerate with a medium- to coarse-grained sandstone matrix.

3.3 Surface Water and Groundwater

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

Groundwater was not encountered during our subsurface exploration at the site. It should be noted that groundwater levels may fluctuate with seasonal variations and irrigation and local perched groundwater conditions may exist within cemented layers of the Alluvium and at the Stadium Conglomerate geologic contact. Nevertheless, based on the above information, we do not anticipate groundwater will be a constraint to the construction of the proposed building or associated improvements.

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 and the weathered upper portions of the Alluvium which are considered compressible. All the undocumented fill and portions of the Alluvium are expected to be removed during excavation operations for the proposed commercial development at the site. Recommendations for remedial grading of these soils are provided in the recommendations section of this report.

3.4.2 Expansion Potential

The majority of the onsite material is expected to have a low to medium expansion potential. However, higher expansive soils may be encountered during the grading of the site for proposed improvements. If highly expansive clay material are encountered during grading, these materials should be exported offsite. Geotechnical observation and/or laboratory testing of the onsite soils during grading is recommended to determine the actual expansion potential of soil for reuse and its suitability.

3.4.3 Soil Corrosivity

During our investigation, a preliminary screening of one representative on-site soil sample was performed to evaluate its potential corrosive effect on concrete and ferrous metals. Laboratory testing on the representative soil sample that was obtained during our subsurface exploration evaluated pH, minimum electrical resistivity, and chloride and soluble sulfate content. The sample tested had a measured pH of 8.5 and a measured minimum electrical resistivity of 900 ohm-cm. The test result also indicated that the sample had a chloride content of 48 parts per million (ppm) and a soluble sulfate content of 300 ppm (<0.1%).

3.4.4 Excavation Characteristics

It is anticipated the onsite soils can be excavated with conventional heavy-duty construction equipment. Localized cemented zones, if encountered, may require heavy ripping. 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 Alluvium. In addition, localized zones of friable sands also may occur within the Alluvium. Beds of friable sands, gravel, and cobble may experience caving during unsupported excavations or drilling.

3.4.5 Infiltration

Field 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. We have used the following equation based upon the Porchet Method to convert measured percolation rates to infiltration rates in accordance with City of San Diego Standards (City of San Diego, 2016). In addition, we have included a factor of safety of 2 for the evaluation of existing site conditions. The storm water design factor of safety should be determined by civil engineer and reviewed by geotechnical consultant. Also, additional field percolation tests may be required within storm water retention areas once final locations are determined by the civil engineer.

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

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. Field data and measured 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 (min/in)	Calculated Infiltration Rate (inch/hr)	Calculated Infiltration Rate w/ FS of 2 (inch/hr)
P-1	4	Alluvium (Qya)	125	0.07	0.035
P-2	4	Alluvium (Qya)	36	0.25	0.125

Based on the clayey nature of the onsite soils (EI=85) and the shallow depth of the well cemented and impermeable Stadium Conglomerate, storm water infiltration at the site may create adverse effects such as groundwater mounding and/or ponding of water near structures or pavement. Therefore, the site is categorized as "No Infiltration", as determined by the storm water Model 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.

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 is considered to move both 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; a limited thickness of permeable Alluvium; and other unknown factors. Accordingly, the possibility of future surface ponding of water, as well as, shallow groundwater impacts on subterranean structures or improvements should be anticipated.

4.0 SEISMIC AND GEOLOGIC HAZARDS

4.1 Regional Tectonic Setting

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. The Rose Canyon, Newport-Inglewood (offshore), and Coronado Bank are active faults are located west to southwest of the site (Jennings, 2010). The primary seismic risk to the site area is the Rose Canyon fault zone located 5.9 miles west of the site (USGS, 2014b).

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 San Diego, 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 Holocene-active or pre-Holocene faults transecting the site. The site is also not located within any State mapped Earthquake Fault Zones or City of San Diego mapped fault zones. The nearest active fault is the Rose Canyon fault zone located approximately 5.9 miles west of the site (USGS, 2014b). A strand of the La Nacion fault as depicted on the City of San Diego's Seismic Safety Study Geologic Hazard Maps is located approximately 1,200 feet west of the site (City of San Diego, 2008).

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 5.9 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 previously discussed, no active faults are mapped transecting or projecting toward the site. Therefore, surface rupture hazard due to faulting is considered very low. Ground cracking due to shaking from a seismic event is not considered a significant hazard either, since the site is not located near slopes.

4.4.2 Mapped Fault Zones

The site is not located within a State mapped Earthquake Fault Zone (EFZ). 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 Site Class D based on our experience with similar sites in the project area and the results of our subsurface evaluation.

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, 2017) 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	=	1.125
	F_v	=	1.684
Mapped MCE Spectral Accelerations	S_S	=	0.937g
	S_1	=	0.358g
Site Modified MCE Spectral Accelerations	S_{MS}	=	1.055g
	S_{M1}	=	0.603g
Design Spectral Accelerations	S_{DS}	=	0.703g
	S_{D1}	=	0.402g

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 (MCE_G). The mapped MCE_G peak ground acceleration (PGA) is 0.379g for the site. For a Site Class D, the F_{PGA} is 1.121 and the mapped peak ground acceleration adjusted for Site Class effects (PGA_M) is 0.425g 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. Both research and historical data indicate that loose, saturated, granular soils are susceptible to liquefaction and dynamic settlement. Liquefaction is typified by a loss of shear strength in the affected soil layer, thereby causing the soil to behave as a viscous liquid. This effect may be manifested by excessive settlements and sand boils at the ground surface.

The site is located within in Geologic Hazard Category (GHC) 32 as shown on the City of San Diego's Seismic Safety Study Geologic Hazard Maps (City of San Diego, 2008). GHC 32 is characterized as having a potential susceptibility to liquefaction and ground failure. However, due to the presence of shallow loose undocumented artificial fill which overlies medium dense to dense clayey in nature Alluvium and very dense Stadium Conglomerate, and the absence of groundwater, the potential for liquefaction at the site is generally considered to be low. Also, considering planned grading and foundation design measures, dynamic settlement potential is considered to be negligible.

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. Based on the low susceptibility to liquefaction, the possibility of earthquake-induced lateral spread is considered to be low for the site.

4.5.3 Tsunamis and Seiches

Based upon the California Emergency Management Agency Tsunami Inundation Map (CalEMA, 2009), the site is not located within a tsunami inundation area. In addition, based on the generally strike-slip character of off-shore faulting and proposed elevation of the site with respect to sea level, the possibility of seiches and/or tsunamis is considered to be low.

4.6 Flood Hazard

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2012); the site is located within the 500 year flood plain (Figure 5). The civil engineer should consider this in site planning. In addition, the site is not located downstream of a dam or within a dam inundation area based on our review of topographic maps.

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;
- Holocene-active or pre-Holocene faults do not transect the site. Holocene-active faults do not project toward the site. The closest Holocene-active fault is the Rose Canyon fault zone located approximately 5.9 miles to the west;
- The peak horizontal ground acceleration associated with the Maximum Considered Earthquake Ground Motion is 0.425g;
- The potential for liquefaction and seismic settlement at the site is considered to be low;
- The existing onsite soils were found to have a low to medium potential for expansion, but highly expansive soils may be present within the Alluvium;
- The existing undocumented fill and weathered upper portions of the Alluvium are potentially compressible and not suitable for supporting foundations or additional fill, and need to be removed;
- The existing onsite soils are generally suitable for use as engineered fill provided they have an expansion index less than 70 and are free of organic material, debris, and rock fragments larger than 8 inches in maximum dimension. If more expansive soils are encountered during grading, selective grading may need to be performed and the more expansive soils should be exported from the site;
- Based on the results of our subsurface exploration, we anticipate that the onsite materials should be generally excavatable with conventional heavy-duty earthwork equipment. Localized cemented zones within the Alluvium may be difficult to excavate and may require heavy ripping, which can produce oversized rock fragments. In addition, the Alluvium contains cobbles that range in size from 6 to 8 inches in

diameter with isolated cobbles to up to 1 foot in diameter expected. Alluvial materials may need to be screened of plus sized material to be suitable for reuse as engineered fill. Unknown objects such as buried concrete footings and debris left from previous site uses should be anticipated and are common on sites where previous development existed;

- Based on the results of our geotechnical evaluation, it is our opinion that the proposed site improvements can be supported on conventional reinforced concrete foundations founded on new compacted fill soils;

- The static groundwater table should not be encountered during remedial grading activities. Although not encountered during our exploration, localized seepage along cemented zones, sand lenses within the Alluvium, and the contact with the Stadium Conglomerate may occur;

- The site is proposed for remedial grading of the near-surface fill and upper portions of the Alluvium. The new compacted artificial fill will likely consist of a mixture of soils ranging from clayey sands and gravels that will have permeable and impermeable layers that can transmit and perch groundwater in unpredictable ways. 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 storm water retention and detention designs should be reviewed by geotechnical consultant and additional percolation tests may be required at retention areas once final locations are determined by the civil engineer; and

- 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. However, the onsite soils are also considered to be very corrosive to buried uncoated ferrous metals. A corrosion consultant may be consulted to provide additional information.

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, engineered structures, and pavements should be cleared of surface and subsurface obstructions, including any existing debris and undocumented fill, old slabs, loose, compressible, or unsuitable soils, and stripped of vegetation. Removed vegetation and debris should be properly disposed off-site.

6.1.2 Excavations and Oversize Material

Excavations of the onsite materials may generally be accomplished with conventional heavy-duty earthwork equipment. However, local heavy ripping may be required if cemented zones within the Alluvium is encountered. Excavation for utilities may also be difficult in some areas. Where soils are found to have greater than 30 percent oversize particles retained on the ¾-inch sieve, corrections using ASTM D4718 are no longer considered valid. Where materials exceed the oversize fraction allowed by ASTM D4718, use of a test fill that contains oversize fraction within the allowable limits of the standard can be compacted and tested to develop a field method to obtain the specified compaction. That method should then be applied to subsequent layers that exceed the maximum allowable oversize percentage.

Due to the variable amount of oversized cobble in the Alluvium, a screening process of the removed Alluvium materials may need to be utilized to remove the plus sized cobble prior to its use as engineered fill. From a geotechnical perspective, it is considered acceptable to reuse the gravelly materials with a maximum dimension of 8 inches or less within the limits of the proposed improvements. Placement of fills with oversize materials shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. However, oversize material larger than 3 inches should not be placed within 1 vertical feet of finish pad grade, pavement and/or hardscape subgrade, and within 2 feet of future utilities or underground construction. Cobble, or other irreducible material with a maximum dimension greater than 8 inches, should not be placed within engineered fill areas and should be disposed of offsite.

Due to the general density and soil type characteristics of the Alluvium, temporary shallow excavations less than 5 feet in depth with vertical sides should remain stable for the period required to construct utilities, provided the trenches are free of adverse geologic conditions. Overlying artificial fill soils and beds of friable sands within the Alluvium present at the site may ravel during trenching operations. In accordance with OSHA requirements, excavations should be shored or be laid back in accordance with Section 6.2 if workers are to enter such excavations.

6.1.3 Removal of Compressible Soils

Undocumented fill and the upper weathered portions of the Alluvium at the site may settle as a result of wetting or settle under the surcharge of engineered fill and/or structure loads supported on shallow foundations.

All undocumented fill at the site should be completely removed. In addition, weathered portions of the Alluvium, if encountered beneath settlement-sensitive improvements and foundations (i.e. proposed structures), should be removed. Also, we recommended that at least 2 feet of compacted fill be placed below the proposed building foundations. Based on the results of our subsurface exploration, we anticipate removal depths between 4 to 5 feet will be necessary within the building pad areas for the improvements. The lateral limits of the bottom of the remedial removals should extend to the outside of the building/structure footprint a distance equal to the excavation depth or 5 feet, whichever is greater. If lateral removal limits cannot be accomplished, deepened footings may be required. The bottom of all removals should be evaluated by a Certified Engineering Geologist to confirm conditions are as anticipated.

In areas of proposed pavements, hardscape and landscaping features, removals should be performed to a depth of at least 2 feet below proposed finish grade. Isolated deeper removals may be necessary. The lateral limits of the removals should extend at least 3 feet beyond the limits of the proposed improvements, where possible. The bottom of all removals should be evaluated by a Certified Engineering Geologist to confirm conditions are as anticipated.

6.1.4 Engineered Fill

In areas proposed to receive engineered fill, the existing upper 8 inches of subgrade soils should be scarified then moisture conditioned to a moisture content of 3 to 5 percent above the optimum moisture content and compacted to 90 percent or more of the maximum laboratory dry density, as evaluated by ASTM D 1557. Soil materials utilized as fill should have an expansion index less than 70 and be free rock fragments larger than 8 inches in maximum dimension, organic materials, and deleterious debris. Oversize material larger than 3 inches shall not be placed within 1 vertical feet of finish pad grade, pavement and/or hardscape subgrade, and within 2 feet of future utilities or underground construction. Cobble, or other irreducible material with a maximum dimension greater than 8 inches, shall not be placed within engineered fill areas and should be disposed of offsite. Fill should be moisture conditioned to at least 3 to 5 percent above the optimum moisture content and compacted to 90 percent or more relative compaction, in accordance with ASTM D 1557. 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.

In pavement roadway areas, the upper 12 inches of subgrade soils should be scarified, then moisture conditioned to a moisture content at least 2 percent above the optimum content and compacted to 95 percent or more of the maximum laboratory dry density, as evaluated by ASTM D 1557.

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

6.1.5 Earthwork Shrinkage/Bulking

The volume change of excavated onsite materials upon recompaction as fill is expected to vary with material and location. Typically, the fill soils and the Alluvium 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 to 8 percent shrinkage factor is considered appropriate for the artificial fill and a 3 percent shrinkage is considered appropriate for the Alluvium (not accounting for screened out cobbles).

6.1.6 Import Soils

If import soils are necessary to replace the onsite soil materials which have been deemed unacceptable to be used as engineered fill, these soils should be granular in nature, have an expansion index less than 50 (per ASTM Test Method D4829), and have a low corrosion impact to the proposed improvements. Import soils and/or the borrow site location should be evaluated by the geotechnical consultant prior to import. The contractor should provide evidence that all import materials comply with Department of Toxic Substances Control guidelines for import materials.

6.1.7 Expansive Soils and Selective Grading

Based on our laboratory testing and observations, we anticipate the onsite soil materials possess a low to medium expansion potential (Appendix C). However, should highly expansive materials be encountered, selective grading may need to be performed and the highly expansive soils should be exported offsite. In addition, to accommodate conventional foundation design, all engineered fill materials placed within the building pads and 5 feet outside the limits of the building foundations, should have an expansion index less than 70. Also, materials with an expansion index less than 50 should be placed within the upper 2 feet of subgrade below paving and hardscape areas. Testing of pad and subgrade fill materials should be performed to confirm expansion potential according to ASTM D4829.

6.1.8 Utility Trench Excavation and Backfill

All excavation work should comply with the current requirements of OSHA. Trenches (either open or backfilled) which parallel structures, pavements, or flatwork should be planned so that they do not extend below a plane having a downward slope of one vertical and two horizontal from a line nine inches above the bottom edge of footings, pavements, or flatwork. Also, no parallel trenches should be closer than 1.5 feet from the closest edge of footings, pavements, or flatwork. Should it be necessary to locate parallel trenches which do not meet the criteria recommended above for footings at conventional depth, we recommend that the footing depths be increased until the criteria are met. A check should be made by the civil designer to verify that all trenches comply with the setback recommendations of this paragraph. If there are special cases where these requirements are not practical, the civil designer should communicate with the project geotechnical engineer and architect on a case-by-case basis.

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. Onsite materials will probably not meet bedding requirements. Except for predominantly clayey soils, the onsite soils may be used as trench backfill above the pipe 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.

6.2 Temporary Excavations

Sloping excavations may be utilized when adequate space allows. Based on the results of our update evaluation, we provide the following recommendations for sloped excavations in fill soils or competent Alluvium without seepage conditions.

Table 3 Temporary Slope Ratios		
Excavation Depth (feet)	Maximum Slope Ratio In Fill Soils	Maximum Slope Ratio In Alluvium
0 to 5	1:1 (Horizontal to Vertical)	1:1 (Horizontal to Vertical)

The above values are based on the assumption that no surcharge loading or equipment will be placed within 10 feet of the top of slope. Care should be taken during excavation adjacent to the existing structures so that undermining does not occur. A "competent person" should observe the slope on a daily basis for signs of instability.

6.3 Surface Drainage and Erosion

Surface drainage should be controlled at all times and carefully taken into consideration during precise grading, landscaping, and construction of site improvements. The proposed development should have appropriate drainage systems to collect roof runoff. Positive surface drainage should be provided to direct surface water away from the structures toward the street or suitable drainage facilities. Planters should be designed with provisions for drainage to the storm drain. Ponding of water adjacent to structures or pavements should be avoided.

The impact of heavy irrigation or inadequate runoff gradient can create perched water conditions, resulting in seepage or shallow ground water conditions where previously none existed. Maintaining adequate surface drainage and controlled irrigation will significantly reduce the potential for nuisance-type moisture problems. To reduce differential earth movements such as heaving and shrinkage due to change in moisture content of foundation soils, which may cause distress to structures and improvements, moisture content of the soils surrounding the improvements should be kept as relatively constant as possible.

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.

6.4 Foundation and Slab Considerations

The proposed structure may be constructed with conventional foundations. Foundations and slabs should be designed in accordance with structural considerations and the following recommendations. These recommendations assume that the soils placed within the building pads and 5 feet outside the limits of the building foundations have an expansion index less than 70. If more expansive materials are encountered and selective grading cannot be accomplished, revised foundation recommendations may be necessary. The foundation recommendations below assume that the building foundation will be underlain by properly compacted fill.

6.4.1 Shallow Spread Footing Considerations

The proposed structure may be supported by conventional, continuous or isolated spread footings. 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 3,000 pounds per square foot (psf) if founded in dense compacted fill soils. 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 should be designed in accordance with the structural engineer's requirements.

6.4.2 Slab Design

The slab-on-grade should be at least 5 inches thick and be reinforced with No. 4 rebars 18 inches on center each way (minimum) placed at mid-height in the slab. We recommend control joints be provided across the slab at appropriate intervals as designed by the project architect.

For slab areas where vapor control is appropriate, a minimum 15-mil vapor barrier should be provided between the underslab and gravel capillary break. The vapor barrier should have a permeance of less than 0.01 perms across the entire slab area in the final constructed condition. Measures to protect the barrier should be implemented throughout the installation and slab construction process to prevent damage (ASTM E1643). Vapor barrier materials should conform to ASTM E1745 Class A. The gravel capillary break should consist of a layer of uniform 3/8-inch to 1/2-inch gravel that is at least 4-inches thick. The mix design of the slab concrete should be proportioned to control bleeding, shrinkage and curling. The project architect should provide waterproofing and underslab insulation designs where appropriate.

Note that moisture barriers can retard, but not eliminate moisture vapor movement from the underlying soils up through the slabs. We recommend that the floor covering/insulation installer test the moisture vapor flux rate prior to attempting applications of the flooring/insulation. "Breathable" floor coverings should be considered if the vapor flux rates are high. A slip-sheet or equivalent should be utilized above the concrete slab if crack-sensitive floor coverings (such as ceramic tiles, etc.) are to be placed directly on the concrete slab. Additional guidance is provided in ACI Publications 302.1R-15 Guide for Concrete Floor and Slab Construction and 302.2R-06 Guide for Concrete Slab that Receive Moisture-Sensitive Floor Materials.

The potential for slab cracking may be reduced by careful control of water/cement ratios. The contractor should take appropriate curing precautions during the pouring of concrete in hot weather to minimize cracking of the slabs. We recommend that a slipsheet (or equivalent) be utilized if grouted tile, marble tile, or other crack-sensitive floor covering is planned directly on concrete slabs. All slabs should be designed in accordance with structural considerations. If heavy vehicle or equipment loading is proposed for the slabs, greater thickness and increased reinforcing may be required. The additional measures should be designed by the structural engineer using a modulus of subgrade reaction of 125 pounds per cubic inch. Additional moisture/waterproofing measures that may be needed to accomplish desired serviceability of the building finishes and should be designed by the project architect.

6.4.3 <u>Settlement</u>

For conventional footings, the recommended allowable-bearing capacity is based on a maximum total and differential static settlement of 3/4 inch and 1/2 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. However, for most cases, differential settlements are considered unlikely to exceed 1/4 inch.

6.4.4 Moisture Conditioning

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

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

Table 4
Presoaking Recommendations Based on Finish Grade Soil Expansion Potential

Expansion Potential	Presoaking Recommendations
Low	120 percent of the optimum moisture content to a minimum depth of 12 inches below slab subgrade
Medium	130 percent of the optimum moisture content to a minimum depth of 18 inches below slab subgrade

6.5 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) indicated 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.

6.6 Preliminary Pavement Design Considerations

Based on our experience at similar sites, we have assumed an R-value of 5 for preliminary pavement design. Actual subgrade R-value results should be verified during grading and an adjustment made to the base thicknesses as appropriate. If materials with lower R-value are placed as subgrade in proposed pavement areas, increased base thickness will be necessary.

6.6.1 Flexible Vehicular Pavement Section

It is our understanding that two types of vehicular traffic are to be considered for pavement design; those are auto parking and auto driveway. Table 5 below provides the traffic indices we have considered in our analysis.

Table 5
Design Traffic Index Values

Traffic	Traffic Index
Auto Parking	4.5
Auto Driveway	5.0

Flexible pavement sections have been evaluated in general accordance with the Caltrans method for flexible pavement design and are summarized below in Table 6.

Table 6
AC over Aggregate Base Pavement Sections

Traffic	TI	AC (in)	Aggregate Base (in)
Auto Parking	4.5	4	6
Auto Driveway	5.0	4	8

6.6.2 Rigid Vehicular Pavement Section

Where Portland Cement Concrete pavements are planned, Table 7 presents PCC pavements sections considering an R-Value of 5.

Table 7
PCC Pavement Sections

Traffic	TI	PCC (in)	Aggregate Base (in)
Auto Parking	4.5	6	--
Auto Driveway	5.0	7	--

Pavement materials should conform to and be placed in accordance with Greenbook Specifications. Per City of San Diego Standard Drawing SDG-113, concrete should be Class 560-B-3250 with a modulus of rupture of at least 600 psi. Regular crack control joints should be provided for PCC pavement to mitigate the potential for adverse cracking.

For trash truck aprons, we recommend a full depth of Portland Cement Concrete section of 7 inches with No. 4 bars at 24 inches on center, each way steel and crack-control joints as designed by the project civil or structural engineer. We recommend that jointed sections be as nearly square as possible.

If pavement areas are adjacent to heavily watered landscaping areas, we recommend some measures 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 if asphalt pavement is used for drainage of surface waters.

6.6.3 Pavement Section Materials

Prior to placement of the aggregate base materials, the upper 12 inches of subgrade soils (including beneath the curb and gutter and 6-inches behind the curb and gutter) should be scarified, moisture-conditioned (or dried back) as necessary to at least 2 percent above optimum moisture content and compacted to a minimum 95 percent relative compaction based on ASTM Test Method D1557. Aggregate base should be compacted to a minimum 95 percent relative compaction in accordance with ASTM Test Method D1557. Flexible pavement should be constructed in accordance with current Greenbook Specifications.

Actual pavement recommendations should be based on R-value tests performed on bulk samples of the soils that are exposed at the finished subgrade elevations across the site at the completion of the mass grading operations.

6.7 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. For all concrete flatwork, the upper 12 inches of subgrade soils should be moisture conditioned to at least 3 to 5 percent above optimum moisture content and compacted to at least 90 percent relative compaction based on ASTM Test Method D1557 prior to the concrete placement.

6.8 Control of Groundwater and Surface Waters

The measured percolation and calculated infiltration rates presented in Section 3.4.5 may be used for the planning level screening phase of design. Once the locations of proposed infiltration facilities/systems are known, additional percolation testing may be needed to verify values provided in this report for use in the design phase. During the design phase, it should be noted that an elevated factor or safety may also be used by designers in lieu of additional field testing. Based on our professional experience, sites having such low infiltration rates are best suited for Low Impact Development (LID) BMPs that contain and filter surface waters by the use of flow-through planters, retention, and detention areas which are fully lined with an impermeable liner and have subdrain systems that ties into an approved existing or proposed storm drain system. It should be noted that shallow bioswales, infiltration basins, and other unlined onsite detention and retention systems utilized in areas having 0.01 to 0.5 inches per hour infiltration rates can potentially create perched groundwater conditions and surface seepage conditions off-site, if not mitigated during BMP design.

Foundations, slopes and subsurface improvements (e.g., retaining walls and basements) located adjacent to proposed infiltration systems should be evaluated to ensure that they may not be adversely impacted from infiltration of surface water. Where setbacks cannot be attained, a 30-mil impermeable liner should be placed on the sides and bottom of the infiltration basins. Table 8 below, summarizes preliminary setback recommendations for the project.

Table 8
Stormwater Infiltration System Setbacks
(Measured from bottom of infiltration device)

Setback	Distance
Any Foundation, Retaining Wall, Basement Wall, or Utility Trench	10 horizontal feet from the face of the improvement
Face of any slope	H/2, 10 feet minimum (H is height of slope)

Surface drainage should be controlled at all times and carefully taken into consideration during precise grading, landscaping, and construction of site improvements. Positive drainage (e.g., roof gutters, downspouts, area drains, etc.) should be provided to direct surface water away from structures and improvements and towards the street or suitable drainage devices. Ponding of water adjacent to structures or pavements should be avoided. Roof gutters, downspouts, and area drains should be aligned so as to transport surface water to a minimum distance of 5 feet away from structures. The performance of structural foundations is dependent upon maintaining adequate surface drainage. Where possible, 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 if feasible. 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.

6.9 Construction Observation

The recommendations provided in this report are based on preliminary design information and subsurface conditions disclosed by widely spaced excavations. The interpolated subsurface conditions should be checked by Leighton in the field during construction. Construction observation of all onsite excavations and field density testing of all compacted fill should be performed by a representative of this office. We recommend that all excavations be mapped by the geotechnical consultant during grading to determine if any potentially adverse geologic conditions exist at the site.

6.10 Plan Review

Final project grading and foundation plans should be reviewed by Leighton as part of the design development process to ensure that recommendations in this report are incorporated in project plans.

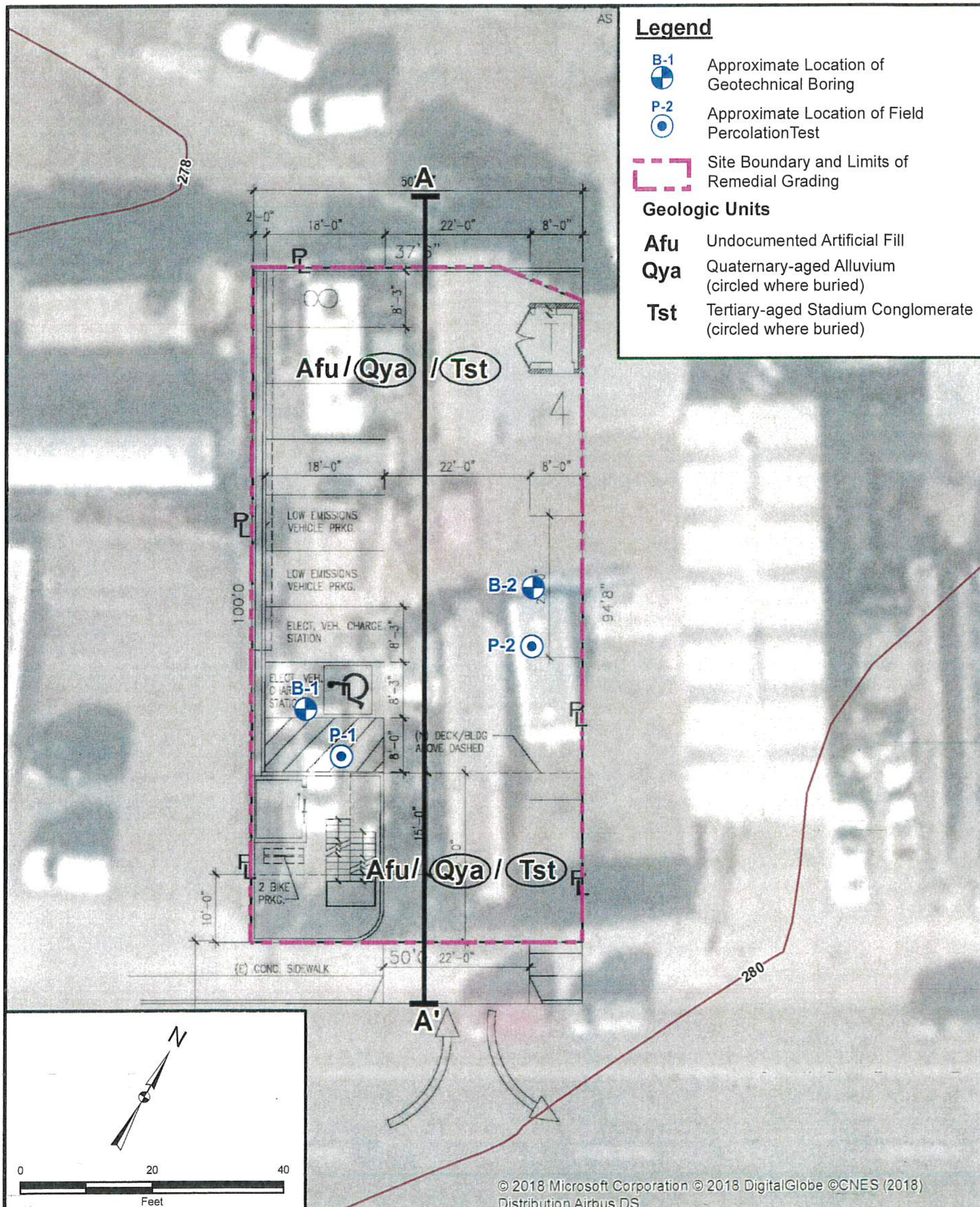
7.0 LIMITATIONS

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: 11931.001	Eng/Geol: SAC/RNB/MDJ	**SITE LOCATION MAP** Federal M.R.D. San Diego, California	Figure 1
Scale: 1 " = 1,000 '	Date: March 2018		Leighton
Base Map: Bing Maps 2018			
Author: Leighton Geomatics (mmurphy)			

Map Saved as P:\Drafting\11931\001\Maps\11931-001_F01_SLM_2018-03-05.mxd on 3/7/2018 8:58:37 AM



Project: 11931.001	Eng/Geol: SAC/RNB/MDJ	
Scale: 1 " = 20 '	Date: March 2018	

Base Map: Site Plan & Property Info by Pacific Design Concepts, Inc, 2/15/2018.

Author: Leighton Geomatics (mmurphy)

GEOTECHNICAL MAP

Federal M.R.D.
San Diego, California

Figure 2

Leighton

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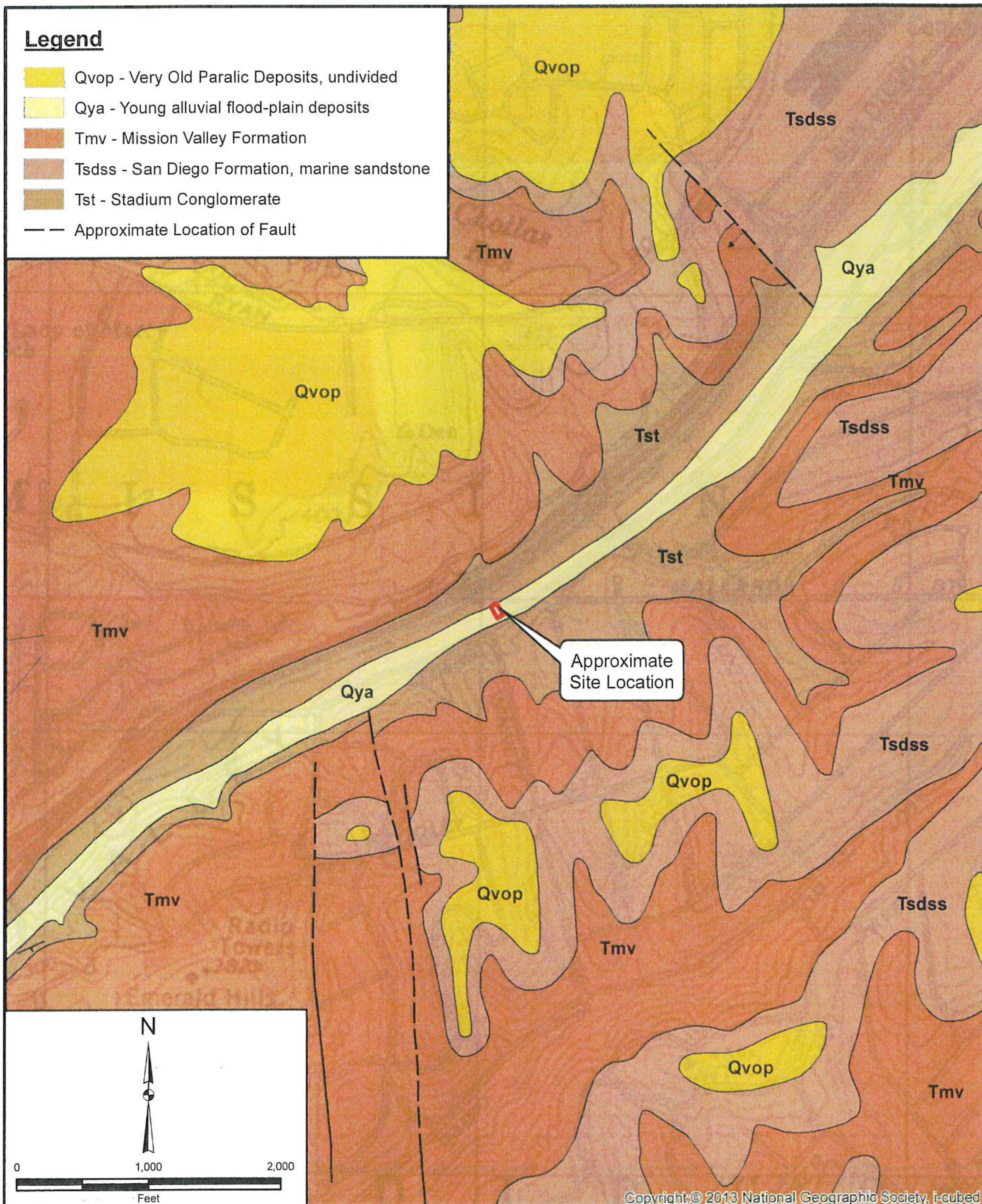

Project: 11931.001	Eng/Geol: SAC/RNB/MDJ
Scale: 1 " = 1,000 '	Date: March 2018

Base Map: Geologic map of the San Diego 30'x60' quadrangle, California, compiled by Michael P. Kennedy and Siang S. Tan, 2008.
Author: Leighton Geomatics (mmurphy)

REGIONAL GEOLOGY MAP

Federal M.R.D.
San Diego, California

Figure 3

Leighton

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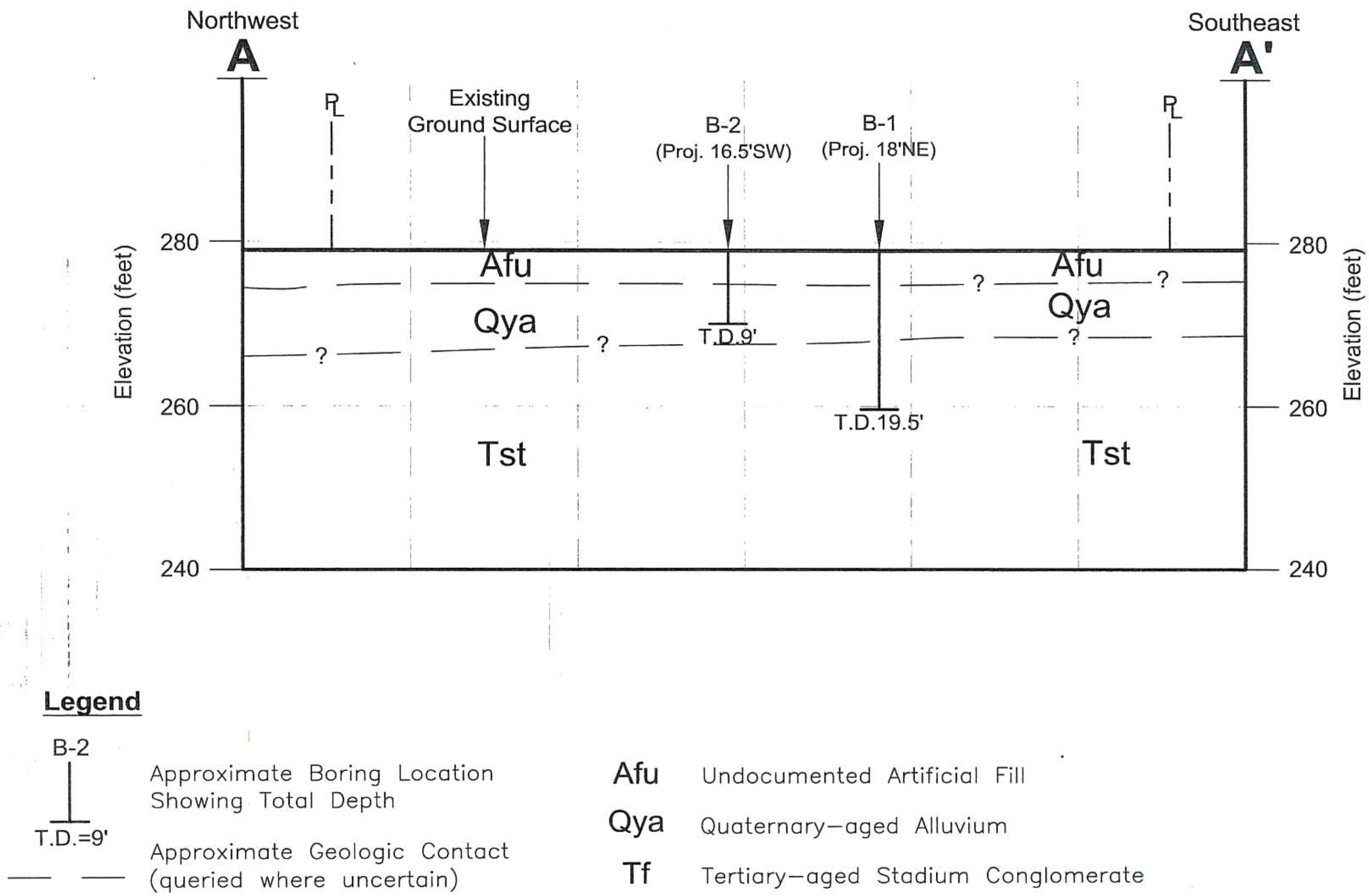

Project: 11931.001	Eng/Geol: SAC/RNB/MDJ
Scale: 1"=20'	Date: March 2018
Reference:	
Author: MAM	

CROSS-SECTION A-A'

Federal M.R.D.
San Diego, California

Figure 4

Leighton

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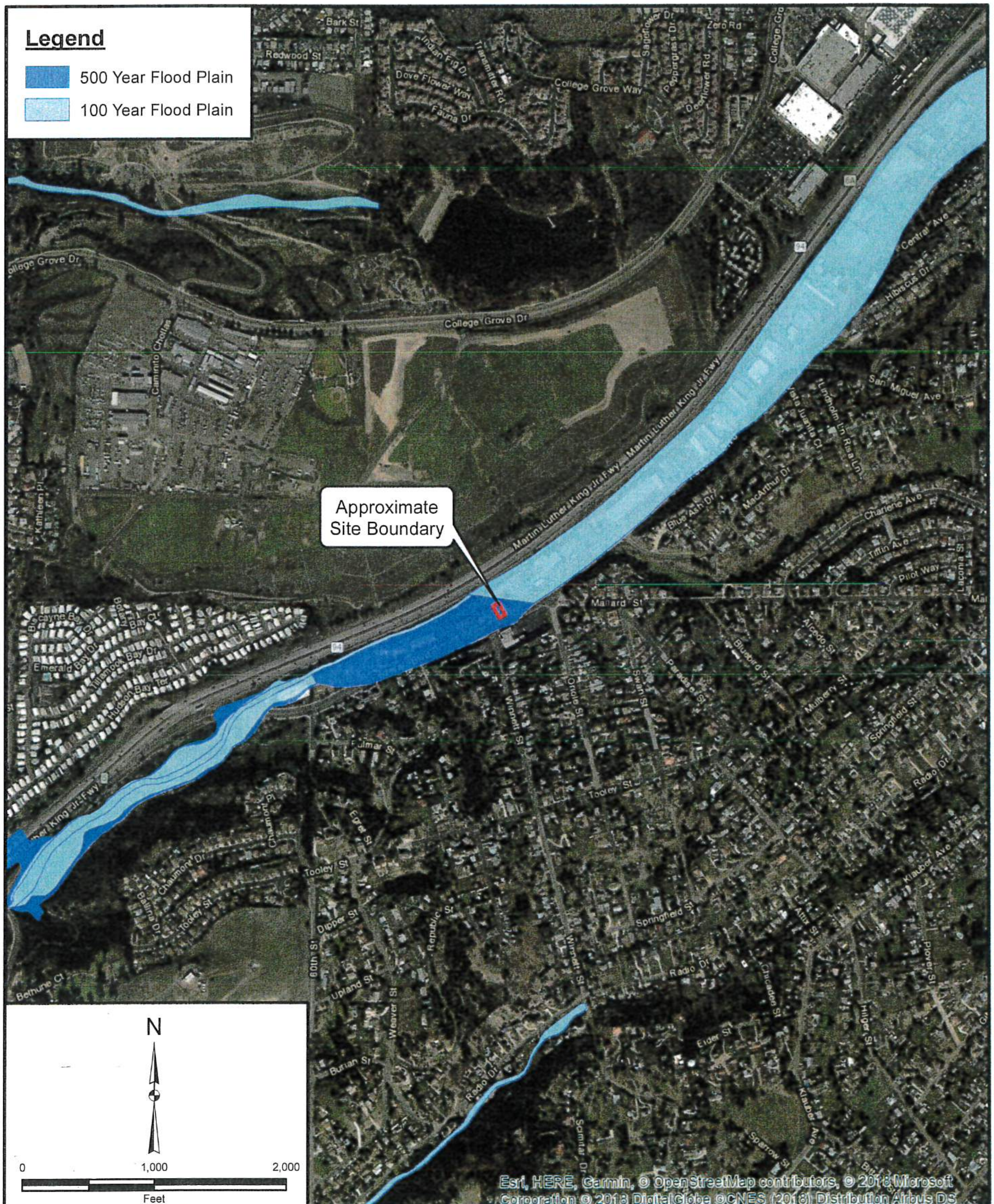

Project: 11931.001	Eng/Geol: SAC/RNB/MDJ	**FLOOD HAZARD ZONE MAP**	Figure 5
Scale: 1 " = 1,000 '	Date: March 2018	Federal M.R.D.	
Base Map: Bing Maps 2018		San Diego, California	
Author: Leighton Geomatics (mmurphy)			Leighton

Map Saved as P:\Drafting\11931\001\Maps\11931-001_F04_FHZM_2018-03-05.mxd on 3/7/2018 8:57:40 AM

Appendix A
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APPENDIX A

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APPENDIX A (Continued)

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Appendix B
Boring Logs & Field Percolation Test Results

GEOTECHNICAL BORING LOG KEY

Date ____________ Sheet 1 of 1

Project KEY TO BORING LOG GRAPHICS Project No. ____________

Drilling Co. ____________ Type of Rig ____________

Hole Diameter ____________ Drive Weight ____________ Drop "

Elevation Top of Elevation ' Location ____________

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

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

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

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GEOTECHNICAL BORING LOG B-1

Project No.	11931.001	**Date Drilled**	3-1-18
Project	Federal M.R.D.	**Logged By**	RNB
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	CME-95 - 140lb - Autohammer - 30" Drop	**Ground Elevation**	279' 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 *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								3" ASPHALT CONCRETE over 4" AGGREGATE BASE	
				B-1 1'-5'				SC	ARTIFICIAL FILL (Afu) @ 6"-4': Clayey SAND, loose, dark brown (7.5 yr 3/3), low plasticity, trace brick fragments, trace well-rounded gravel	EI, CR, SA
275								CL	ALLUVIUM (Qya) @ 4'-5': Sandy CLAY, firm, very dark brown (7.5 yr 2.5/3), moist, high plasticity	
	5			S-1	9 13 15			GC	@ 5': Clayey GRAVEL with SAND, medium dense, strong brown (7.5 yr 4/6), moist, gravel, well-rounded, trace oxidation staining, trace cobble @ 7': Becomes medium dense to dense	
270										
	10			S-2	21 50/6"					
								GM	STADIUM CONGLOMERATE (Tst) @ 11': Cobble CONGLOMERATE, very dense, yellowish brown (10 yr 5/8), moist, medium- to coarse-grained SAND matrix, gravel and cobble well-rounded	
265										
	15			S-3	22 50/6"					
260				S-4	50/6"					
	20								**Bottom of Boring at 19.5 Feet (bgs)** **No groundwater or seepage encountered** **Backfilled with soil cuttings on 3/1/18**	
255										
	25									
250										
	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.	11931.001	**Date Drilled**	3-1-18
Project	Federal M.R.D.	**Logged By**	RNB
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	CME-95 - 140lb - Autohammer - 30" Drop	**Ground Elevation**	279' 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** *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			B-1 1'-5'				SC	ARTIFICIAL FILL (Afu) @ 0-3.5': Clayey SAND, loose, dark brown (7,5 yr 3/3), moist, low to medium plasticity, trace well-rounded gravel	SA
275	5			S-1	20 26 18			GC	ALLUVIUM (Qya) @ 3.5': Clayey GRAVEL with SAND, medium dense, strong brown (7.5 yr 4/6), moist, gravel well-rounded, low to medium plasticity, trace cobble, trace oxidation staining @ 5': Becomes dense @ 8': Cobble layer encountered	
270	10			S-2	50/1"				**Auger Refusal on Cobble at 9 Feet (bgs)** **No groundwater or seepage encountered** **Backfilled with soil cuttings on 3/1/18**	
265	15									
260	20									
255	25									
250	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: Federal M.R.D. Project No.: 11931.001

Proj. Address: Federal Blvd & Winnett Street

SOIL TYPE / TEST LOCATION / BOREHOLE

Soil Type: Clayey GRAVEL w SAND Hole #: P-1

Location: See Figure 2

Hole Dia: 8"

Depth 4'

Tested by: Pre-Saturation Date: 3.1.18 Test Date: 3.2.18

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

Time of Day	Interval / Notes	Initial Depth to Water (ft)	Final Depth of Water (ft)	Δ in Water Level (ft)	Percolation Rate (min/inch)
10:04	Start	3	-	-	-
10:23	19	3	3.36	0.36	4.40
10:53	30	3.36	3.62	0.26	9.62
11:23	30	3.62	3.84	0.22	11.36
11:23	Fill	3	-	-	-
11:53	30	3	3.05	0.05	50.00
12:23	30	3.05	3.06	0.01	250.00
12:53	30	3.06	3.15	0.09	27.78
1:23	30	3.15	3.16	0.01	250.00
1:53	30	3.16	3.17	0.01	250.00
2:23	30	3.17	3.19	0.02	125.00
2:53	30	3.19	3.22	0.03	83.33
3:23	Fill	3	-	-	-
3:53	30	3	3.02	0.02	125.00

Notes: Final Field Percolation Rate = 125 min/in

Leighton

FIELD PERCOLATION TEST DATA SHEET

Project Name:	Federal M.R.D.	Project No.:	11931.001
Proj. Address:	Federal Blvd & Winnett Street		

SOIL TYPE / TEST LOCATION / BOREHOLE

Soil Type: Clayey GRAVEL w SAND — Hole #: P-2

Location: See Figure 2

Hole Dia: 8"

Depth 4'

Tested by: Pre-Saturation Date: 3.1.18 — Test Date: 3.2.18

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

Time of Day	Interval / Notes	Initial Depth to Water (ft)	Final Depth of Water (ft)	Δ in Water Level (ft)	Percolation Rate (min/inch)
9:45	Start	3	-	-	-
10:07	22	3	3.35	0.35	5.24
10:25	18	3.35	3.5	0.15	10.00
10:25	Fill	3	-	-	-
10:55	30	3	3.48	0.48	5.21
11:25	30	3.48	3.55	0.07	35.71
11:25	Fill	3	-	-	-
11:55	30	3	3.4	0.4	6.25
12:25	30	3.4	3.51	0.11	22.73
12:55	30	3.51	3.58	0.07	35.71
12:55	Fill	3	-	-	-
1:25	30	3	3.44	0.44	5.68
1:55	30	3.44	3.54	0.1	25.00
2:25	30	3.54	3.63	0.09	27.78
2:55	Fill	3	-	-	-
3:25	30	3	3.07	0.07	35.71

Notes: Final Field Percolation Rate = 36 min/in

Appendix C
Laboratory Testing Procedures and Test Results

APPENDIX C

Laboratory Testing Procedures and Test Results

Particle/Grain Size Analysis: Particle size analysis was performed by mechanical sieving and wash sieving methods according to ASTM 06913. Plots of sieve results are provided on the figures in this appendix.

Expansion Index Test: The expansion potential of a 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 then loaded to an equivalent 144 psf surcharge and was inundated with water until volumetric equilibrium is reached. The results of this test is presented in the table below:

Sample Location	Sample Description	Expansion Index	Expansion Potential
B-1 @ 1 to 5 feet	Clayey SAND	85	Medium

Soluble Sulfate: 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-1 @ 1 to 5 feet	0.03	Negligible

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

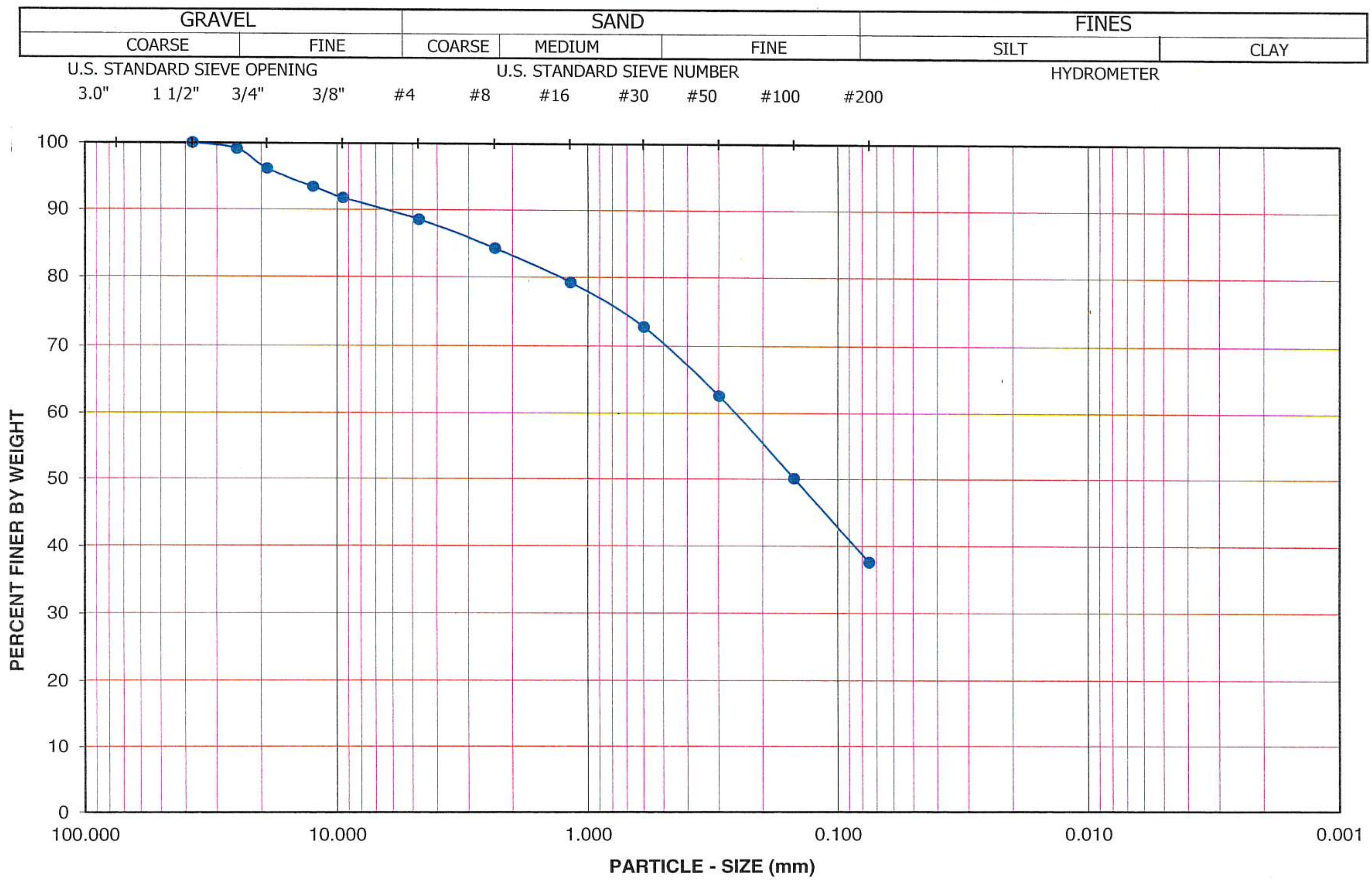
Chloride Content: Chloride content was tested in accordance with DOT Test Method No. 422. The result is presented below:

Sample Location	Chloride Content, ppm
B-1 @ 0 to 5 feet	48

APPENDIX C (Continued)

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

Sample Location	pH	Minimum Resistivity (ohms-cm)
B-1 @ 0 to 5 feet	8.5	900



Project Name: Federal Blvd/MRD

Project No.: 11931.001

Boring No.: B-1 | Sample No.: B-1

Depth (feet): 1.0 - 5.0 | Soil Type : SC

Soil Identification: Clayey Sand (SC), Dark Brown.

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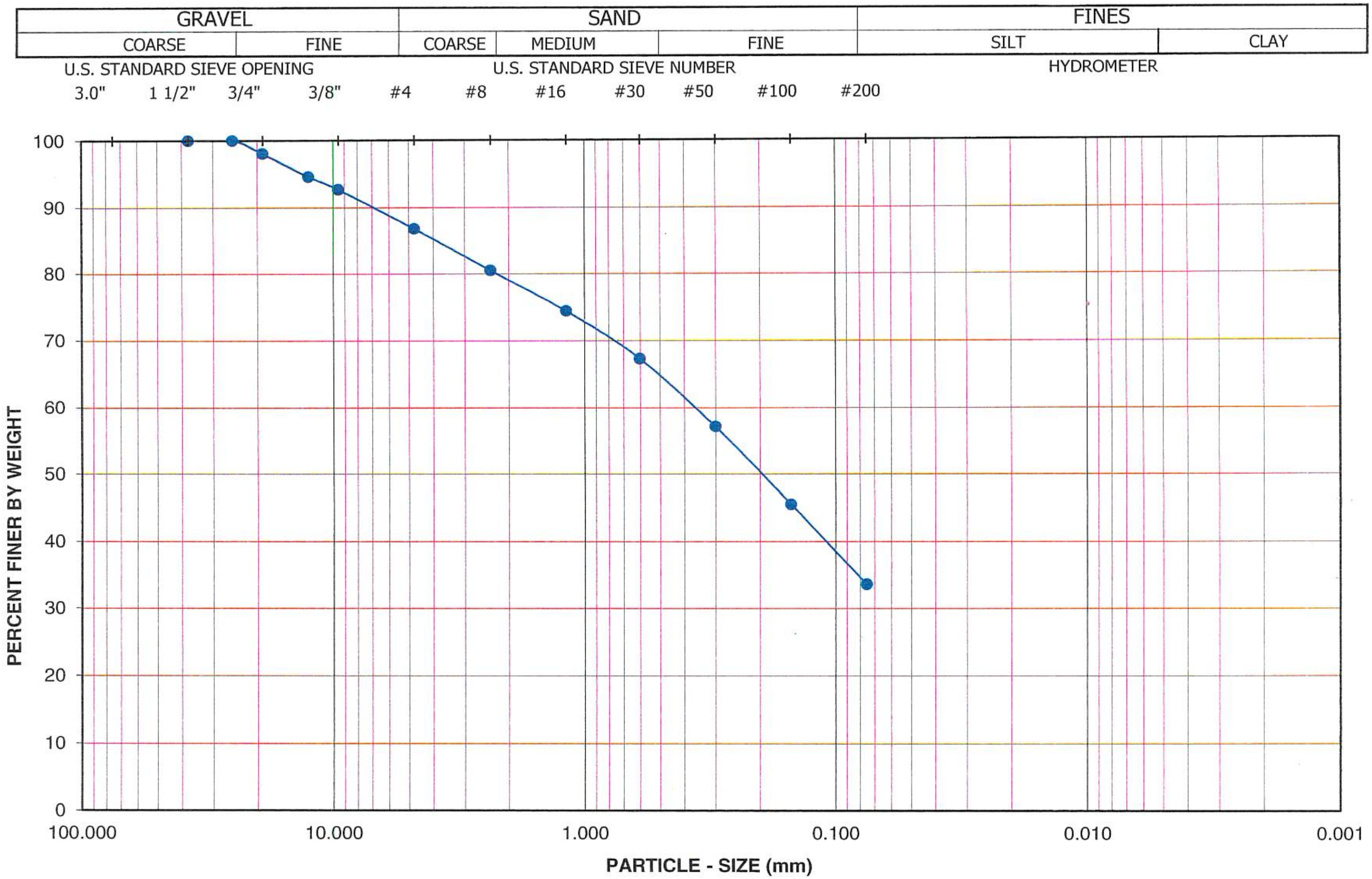
PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 11 : 51 : 38

Mar-18

Sieve; B-1, B-1 (3-1-18)



Project Name: Federal Blvd/MRD

Project No.: 11931.001

Leighton

PARTICLE - SIZE DISTRIBUTION
ASTM D 6913

Boring No.: B-2 Sample No.: B-1

Depth (feet): 0 - 5.0 Soil Type : SC

Soil Identification: Clayey Sand (SC), Dark Brown.

GR:SA:FI : (%) 13 : 53 : 34

Mar-18

Sieve; B-2, B-1 (3-1-18)

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, 2018). The calculated infiltration rates via the Porchet Method and applied safety factor of 2 ranges from 0.035 to 0.125 inches per hour. 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: No, based on the clayey nature of the onsite soils (EI=85) and the shallow depth of the well cemented and impermeable Stadium Conglomerate, storm water infiltration at the site may create adverse effects such as groundwater mounding and/or ponding of water near structures or pavement. 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.

Worksheet C.4-1 Page 3 of 4			
Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?			
Criteria	Screening Question	Yes	No
5	Do soil and geologic conditions allow for infiltration in any appreciable rate or volume? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.	X	
Provide basis: Based on our field percolation testing, the in-situ infiltration rates of the soils at the subject site are greater than 0.01 inches per hour (Leighton, 2018). The calculated infiltration rates via the Porchet Method and applied safety factor of 2 ranges from 0.035 to 0.125 inches per hour. 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.			
6	Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X
Provide basis: No, based on the clayey nature of the onsite soils (EI=85) and the shallow depth of the well cemented and impermeable Stadium Conglomerate, storm water infiltration at the site may create adverse effects such as groundwater mounding and/or ponding of water near structures or pavement. 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.			

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: For a partial infiltration condition (greater than 0.01 inches per hour), the risk of groundwater contamination will not be increased by partial infiltration provided there are no contaminated soil or groundwater sites within 250 feet of the proposed infiltration site. In addition, 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: For a partial infiltration condition (greater than 0.01 inches per hour), violation of downstream water rights is not anticipated based on the site location and that there are no unlined site drainages/creeks/streams within 250 feet of the proposed infiltration site. 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.

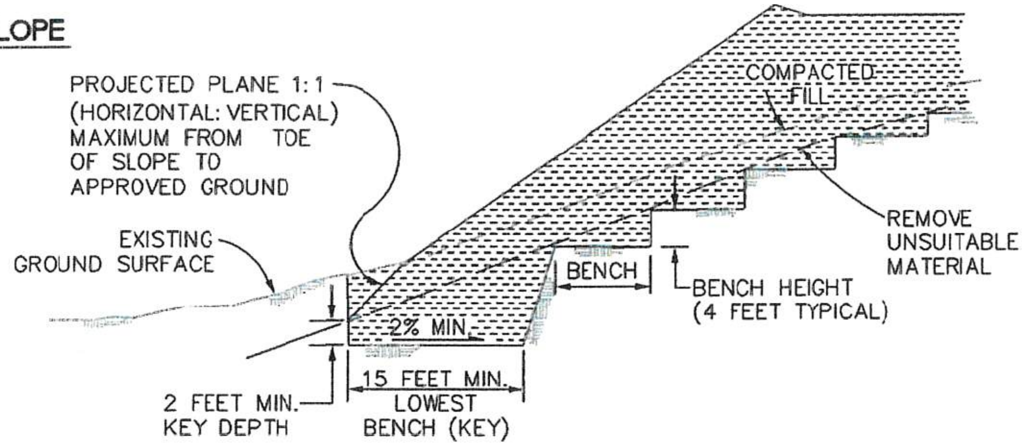
7.3 Lift Thickness

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.

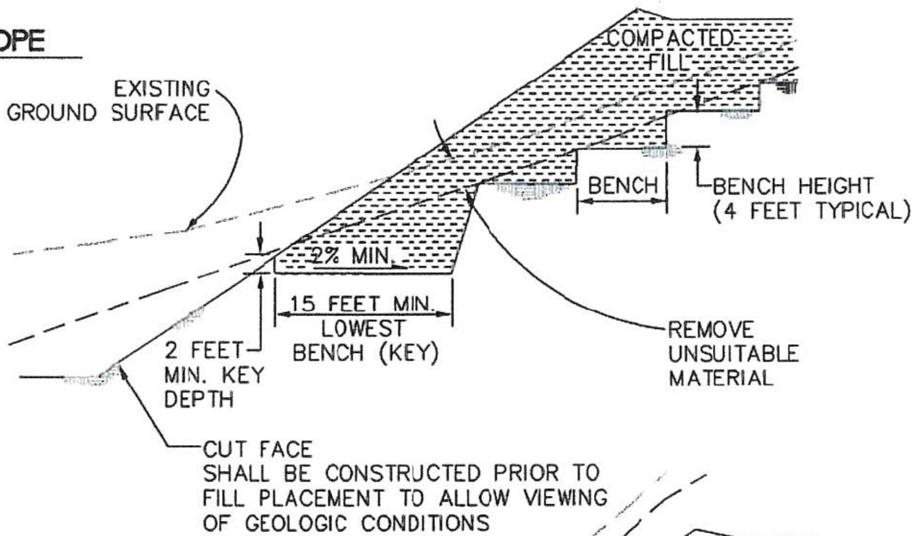
7.4 Observation and Testing

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

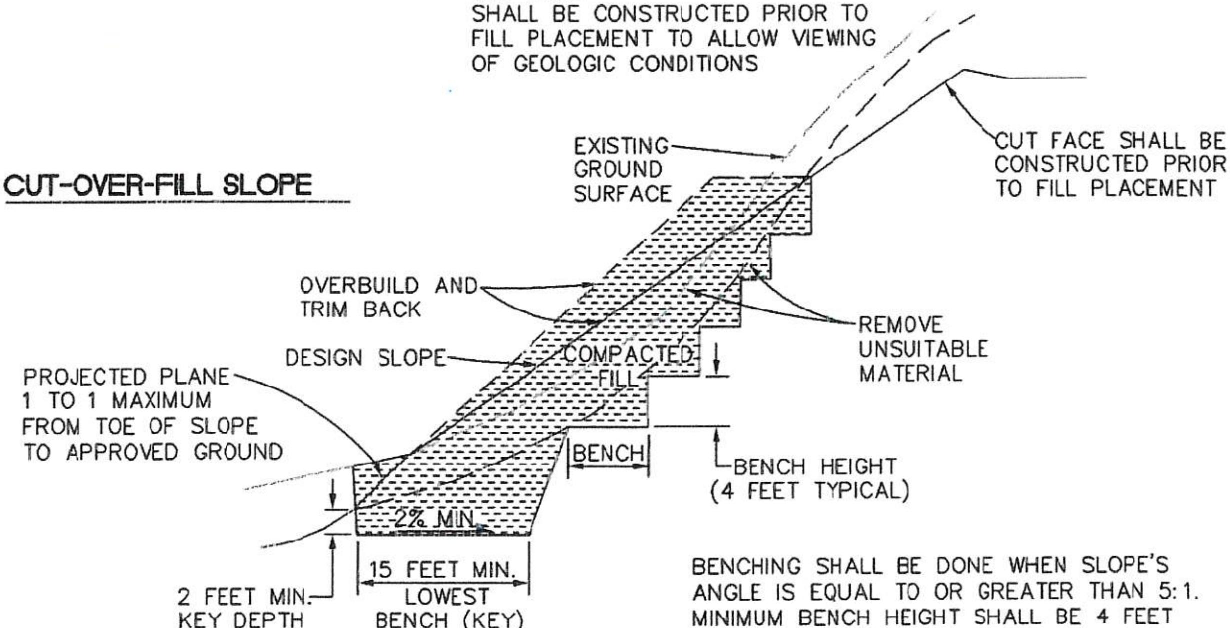
FILL SLOPE



FILL-OVER-CUT SLOPE



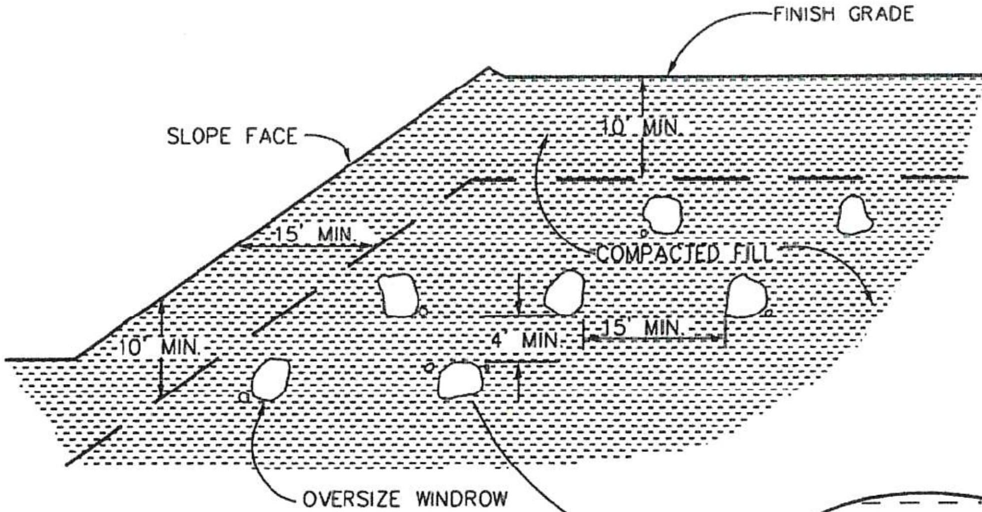
CUT-OVER-FILL SLOPE

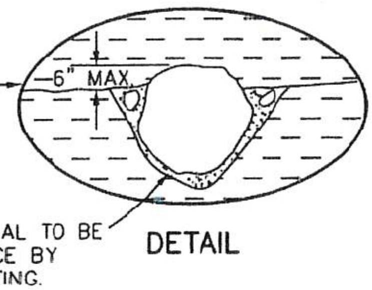


BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

KEYING AND BENCHING

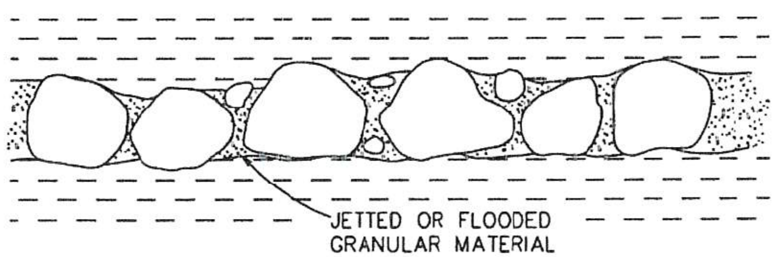
GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL A





DETAIL

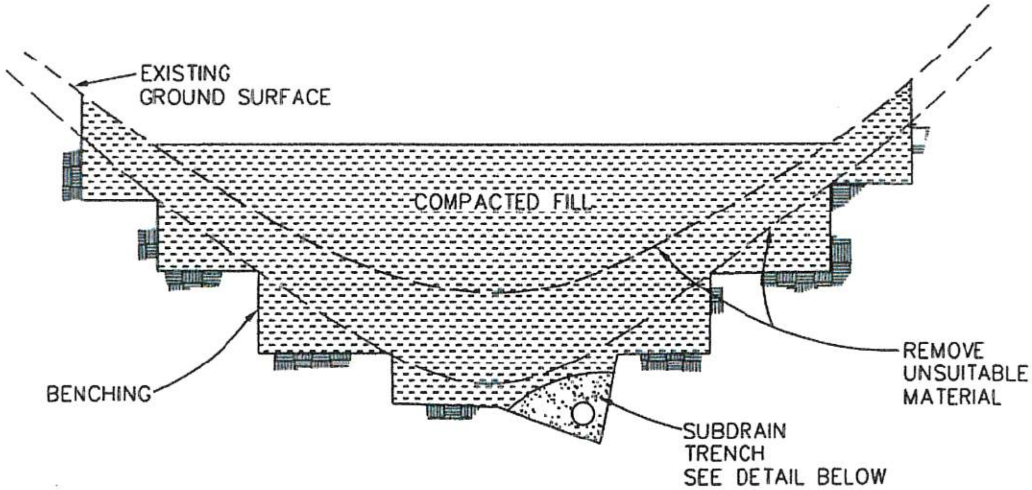
* OVERSIZE ROCK IS LARGER THAN 8 INCHES IN LARGEST DIMENSION.
* EXCAVATE A TRENCH IN THE COMPACTED FILL DEEP ENOUGH TO BURY ALL THE ROCK.
* BACKFILL WITH GRANULAR SOIL JETTED OR FLOODED IN PLACE TO FILL ALL THE VOIDS.
* DO NOT BURY ROCK WITHIN 10 FEET OF FINISH GRADE.
* WINDROW OF BURIED ROCK SHALL BE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

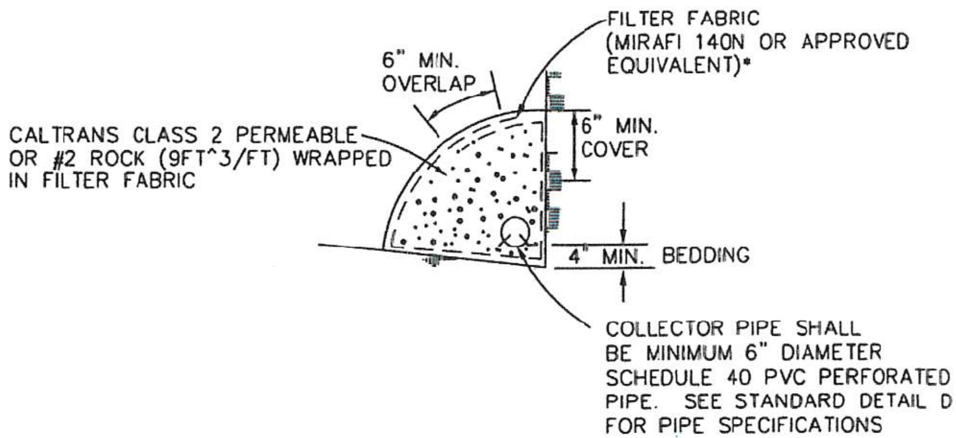


TYPICAL PROFILE ALONG WINDROW

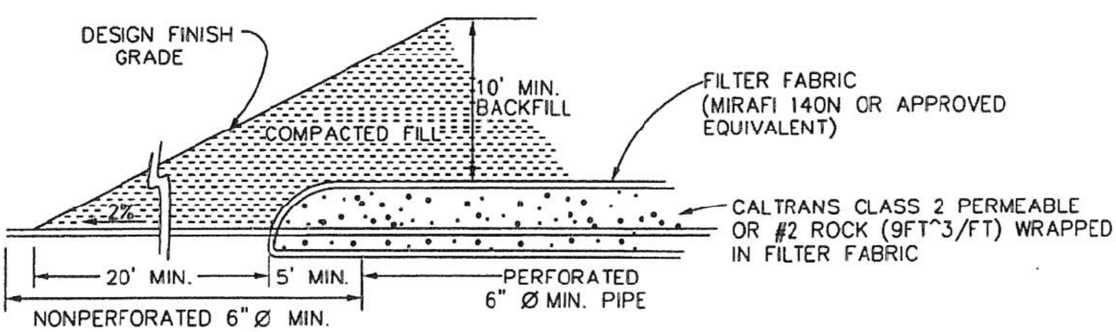
OVERSIZE ROCK DISPOSAL

GENERAL EARTHWORK AND GRADING SPECIFICATIONS
STANDARD DETAIL B





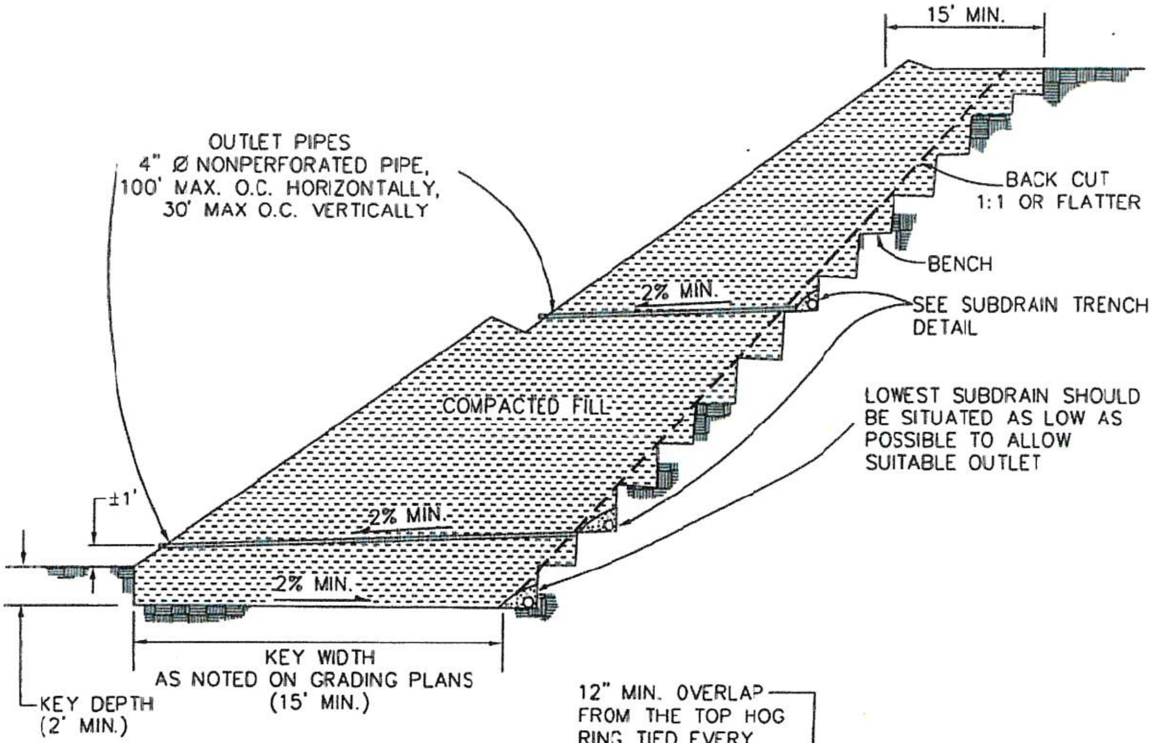
SUBDRAIN DETAIL

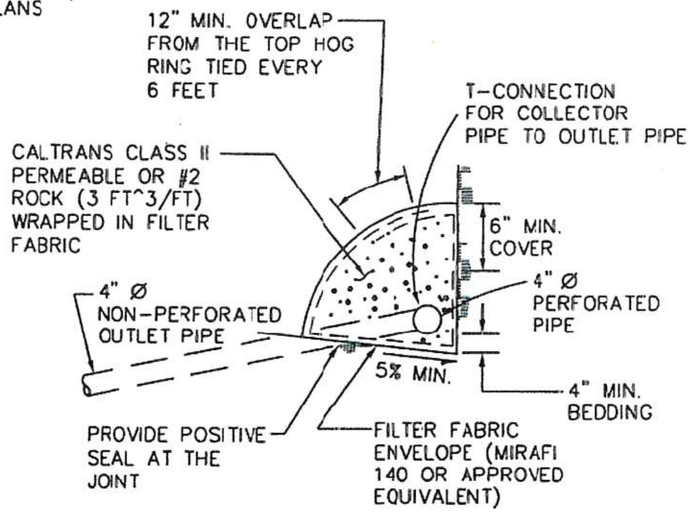


DETAIL OF CANYON SUBDRAIN OUTLET

CANYON SUBDRAINS

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL C





SUBDRAIN TRENCH DETAIL

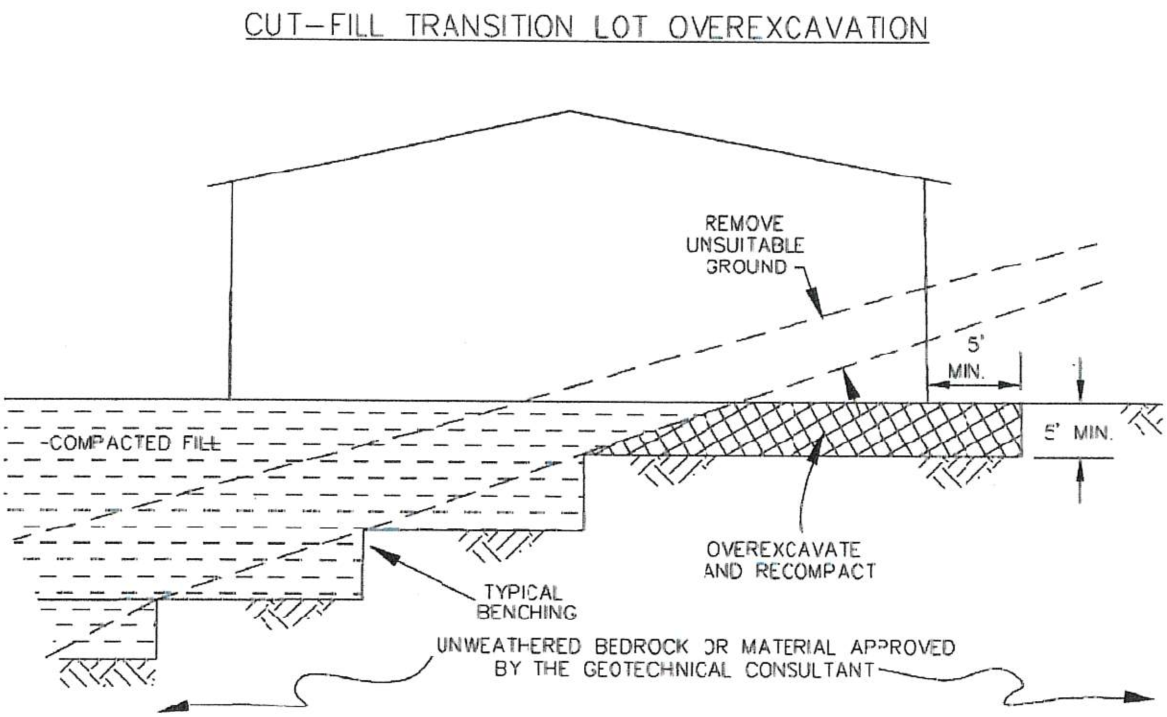
SUBDRAIN INSTALLATION – subdrain collector pipe shall be installed with perforation down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" 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.

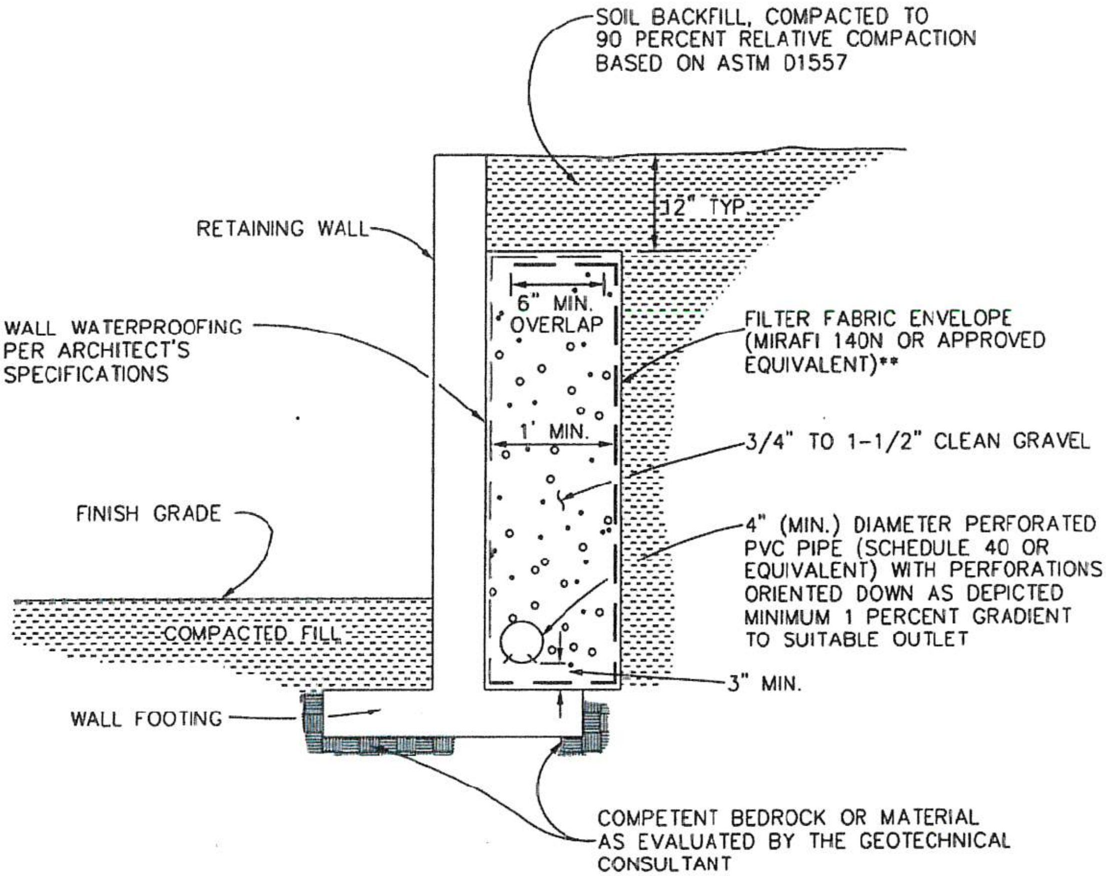
BUTTRESS OR REPLACEMENT FILL SUBDRAINS

GENERAL EARTHWORK AND GRADING SPECIFICATIONS
STANDARD DETAIL D



TRANSITION LOT FILLS

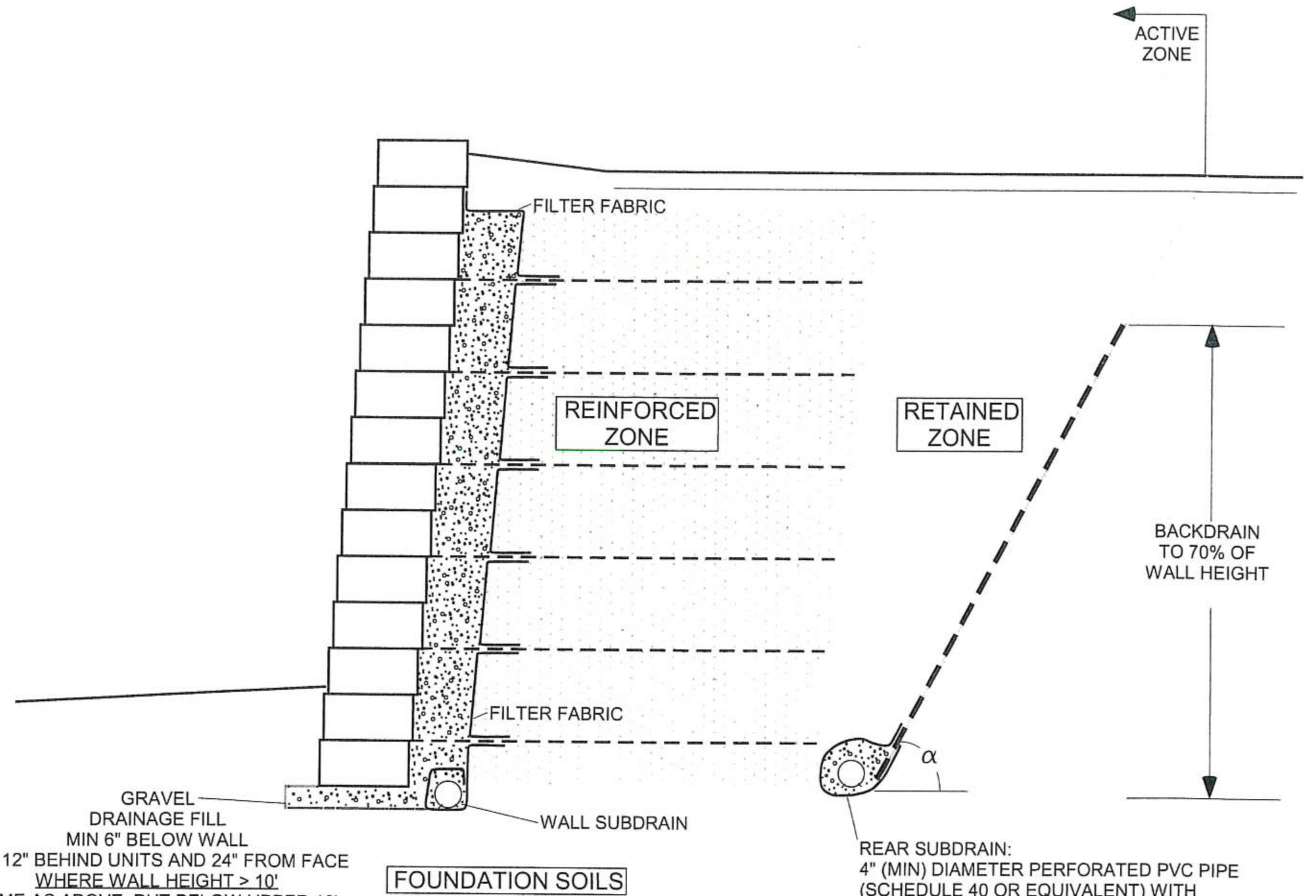
GENERAL EARTHWORK AND GRADING SPECIFICATIONS
STANDARD DETAIL E



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

RETAINING WALL DRAINAGE

GENERAL EARTHWORK AND GRADING SPECIFICATIONS
STANDARD DETAIL F



GRAVEL DRAINAGE FILL
MIN 6" BELOW WALL
MIN 12" BEHIND UNITS AND 24" FROM FACE
WHERE WALL HEIGHT > 10'
SAME AS ABOVE, BUT BELOW UPPER 10'
INCREASE GRAVEL TO
MIN 24" BEHIND UNIT AND 36" FROM FACE
WHERE WALL HEIGHT > 20'
SAME AS ABOVE, BUT BELOW UPPER 20'
INCREASE GRAVEL TO
MIN 36" BEHIND UNIT AND 48" FROM FACE

REAR SUBDRAIN:
4" (MIN) DIAMETER PERFORATED PVC PIPE (SCHEDULE 40 OR EQUIVALENT) WITH PERFORATIONS DOWN. SURROUNDED BY 1 CU. FT/FT OF 3/4" GRAVEL WRAPPED IN FILTER FABRIC (MIRAFI 140N OR EQUIVALENT)

OUTLET SUBDRAINS EVERY 100 FEET, OR CLOSER, BY TIGHTLINE TO SUITABLE PROTECTED OUTLET

NOTES:

1) MATERIAL GRADATION AND PLASTICITY

REINFORCED ZONE:

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

FOR WALL HEIGHT < 10 FEET, PLASTICITY INDEX < 20 AND LIQUID LIMIT < 40
FOR WALL HEIGHT 10 FEET OR TALLER, PLASTICITY INDEX < 6
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
1 INCH	100
3/4 INCH	75-100
NO. 4	0-60
NO. 40	0-50
NO. 200	0-5

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

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

3) GEOGRID TO BE PRETENSIONED DURING INSTALLATION.

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

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

SEGMENTAL RETAINING WALLS

GENERAL EARTHWORK AND GRADING SPECIFICATIONS
STANDARD DETAIL G

REVISED 11/16

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

August 28, 2019

Project No. 11931.002

Aaron Magagna
3639 Midway Drive, Suite B #132
San Diego, California 92110

Attention: Mr. Aaron Magagna

Subject: Geotechnical Response to City of San Diego
Cycle 7 BDR-Geology; L64A-003A
Federal Blvd M.O.
City of San Diego, PTS No. 644432

As requested, we have prepared a response to the City of San Diego review comments for the proposed Federal Blvd M.O. Project located northeast of the intersection of Federal Blvd and Winnett Street in the City of San Diego, California. The San Diego County Assessor's Office designates the subject property as Assessor Parcel Number (APN) 543-020-04-00. The comments include 7 BDR-Geology Comments, PTS No. 644432, dated August 20, 2019. For clarity, the City of San Diego comments are italicized and numbered in accordance with the order presented on the comment sheet. It should also be noted that comments addressed below are specific to the geologic aspects of the project and other comments for other disciplines are not addressed in this letter.

<u>Cycle 3 BDR-Geology Comments, PTS No. 607976</u>

2 *Submit an addendum geotechnical report or update letter that specifically addresses the current building plan set and the following:*

3934 Murphy Canyon Road, Suite B205 ▪ San Diego, CA 92123-4425
858.569.6914 ▪ Fax 858.292.0771 ▪ www.leightongroup.com

Response

Please consider this response to comments as an addendum to the referenced report. Based on our review of the most recent site plans (PDC, 2019), we understand that the project now includes the following changes since the submittal of our referenced geotechnical report:

- The proposed commercial building will consist of a slab-on-grade two-story building which will be built with typical wood frame construction.

3 *Submit digital copies (on CD/DVD/or USB data storage device) of the referenced and requested geotechnical reports for our records.*

Response

The referenced and requested geotechnical reports have been put on a USB data storage device and will be to the City of San Diego Geology Department for their records.

4 *Provide an updated/geotechnical map that shows the distribution of fill and geologic units, location of exploratory excavations, and current proposed construction on a topographic base map.*

Response

See attached Figure 2 (Geotechnical Map).

5 *The project's geotechnical consultant must review the building plans and indicate if the plans are in accordance with their recommendations. Please provide additional analysis and/or recommendations if necessary.*

Response

A review of the project building plan set, including the structural/foundation plans has been completed. The review letter is attached at the rear of this response.

6 *NOTE – Strom Water Requirements for the proposed project will be evaluated by LDR-Engineering Building review. Priority Development Projects (PDPs) may require an investigation of storm water infiltration feasibility in accordance with the Storm Water Standards (including Appendix C and D). Check with your LDR-Engineering Building reviewer on requirements. LDR-Engineering Building may determine that BDR-Geology review of a storm water infiltration evaluation is required.*

Response

We understand that Strom Water Requirements for the proposed project will be evaluated by LDR-Engineering Building review. In addition, we have performed an investigation of storm water infiltration feasibility in accordance with the Storm Water Standards (including Appendix C and D). The results of this investigation can be found in the referenced project geotechnical report.

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 CONSULTING, INC.

Roy N. Butz, PG 8942
Senior Project Geologist

PROFESSIONAL GEOLOGIST
ROY N. BUTZ
NO. 8942
3-31-21
STATE OF CALIFORNIA

Attachments: Appendix A – References
Appendix B – Plan Review Letter
Figure 2 – Geotechnical Map

Distribution: (1) Addressee via email

APPENDIX A
REFERENCES

City of San Diego, Cycle 7 BDR-Geology Comments, PTS No. 644432, dated August 20, 2019.

Leighton Consulting, Inc., 2018, Geotechnical Investigation, Federal Blvd Retail Building, Assessor Parcel Number 543-020-04-00, San Diego, California 92114, Project No. 11931.001, dated March 9, 2018.

Pacific Design Concepts, LLC, 2019, Building Plans, Federal Blvd Marijuana Outlet, dated July 21, 2019.

Projection Engineering, Inc., 2019, Grading Plans, Federal Blvd Marijuana Outlet, dated July 24, 2018.

TJ Engineering, 2019, Structural/Foundation Plans, Federal Blvd Marijuana Outlet, dated July 21, 2019.

APPENDIX B
Plan Review Letter

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

August 28, 2019

Project No. 11931.002

Aaron Magagna
3639 Midway Drive, Suite B #132
San Diego, California 92110

Attention: Mr. Aaron Magagna

Subject: Plan and Specification Review
Federal Blvd M.O.
(APN) 543-020-04-00
San Diego, California

In accordance with your request, we have performed a geotechnical review of the referenced Federal Blvd M.O. Project plans and specifications. It should be noted that our review was limited to the geotechnical aspects of the project and was performed to identify potential conflicts with the intent of the referenced project geotechnical document (Leighton, 2018). Based on our review, it is our professional opinion that the plans and specifications were prepared in substantial conformance with the geotechnical document.

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

Respectfully submitted,

LEIGHTON CONSULTING, INC.

PROFESSIONAL GEOLOGIST ROY N. BUTZ NO. 8942 3-31-21 STATE OF CALIFORNIA

Roy N. Butz, PG 8942
Senior Project Geologist

Distribution: Addressee via email

Attachments: Appendix A – References

3934 Murphy Canyon Road, Suite B205 ▪ San Diego, CA 92123-4425
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Appendix A

References

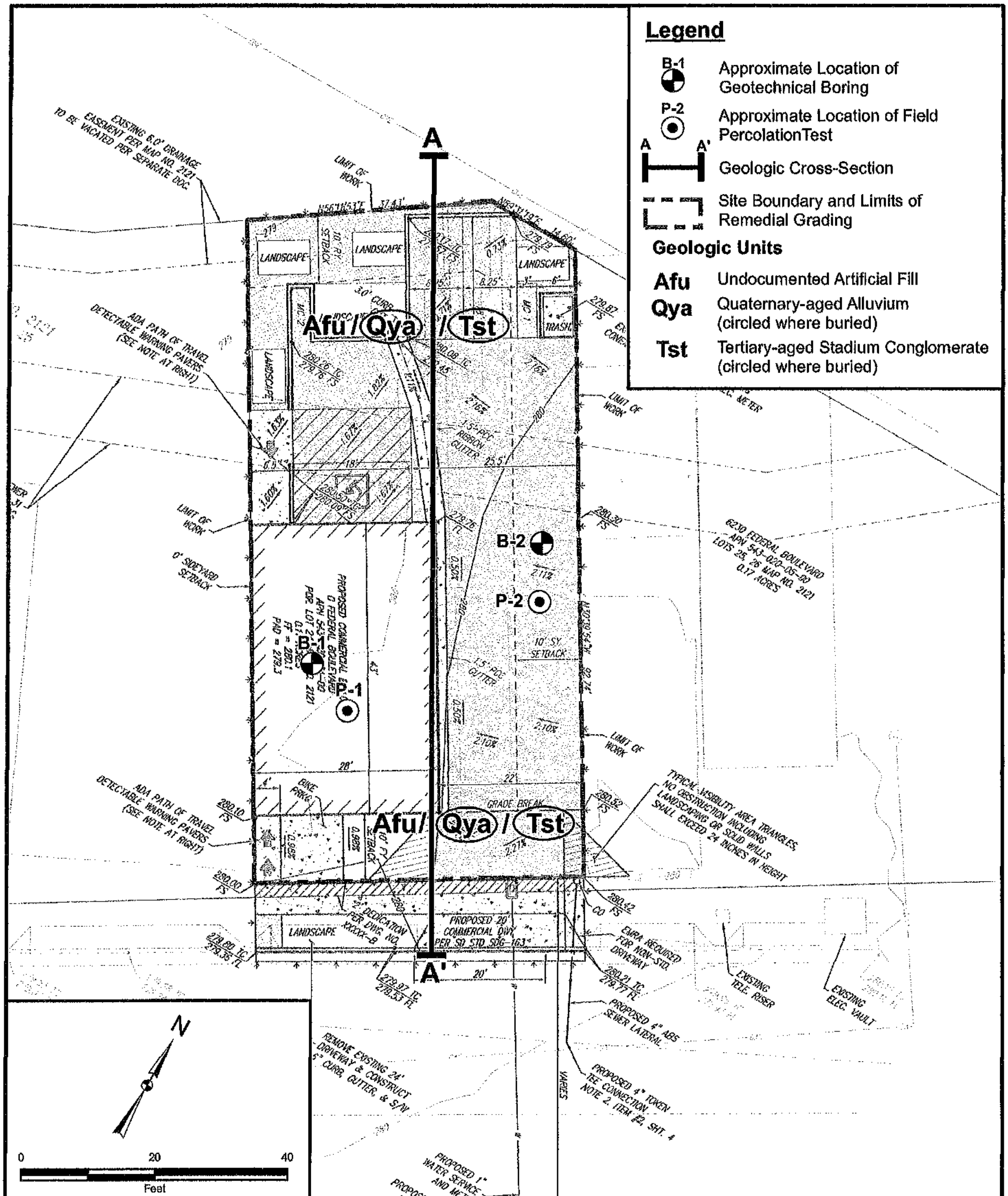
Leighton Consulting, Inc., 2018, Geotechnical Investigation, Federal Blvd Retail Building, Assessor Parcel Number 543-020-04-00, San Diego, California 92114, Project No. 11931.001, dated March 9, 2018.

Pacific Design Concepts, LLC, 2019, Building Plans, Federal Blvd Marijuana Outlet, dated July 21, 2019.

Projection Engineering, Inc., 2019, Grading Plans, Federal Blvd Marijuana Outlet, dated July 24, 2018.

TJ Engineering, 2019, Structural/Foundation Plans, Federal Blvd Marijuana Outlet, dated July 21, 2019.

FIGURE 2
Geotechnical Map



Project: 11931.002	Eng/Geol: SAC/RNB/MDJ
Scale: 1 " = 20 '	Date: August 2019

Base Map: Preliminary Grading Plan by Paul Fisher Engineering.

Author: Leighton Geomatics (mmurphy)

GEOTECHNICAL MAP

Federal Blvd M.O.
San Diego, California

Figure 2

Leighton

Map Saved as V:\Drafting\11931\002\Maps\11931-002_F02_GM_2019-08-28.mxd on 8/28/2019 9:20:43 AM