

August 20, 2021 434432

Mr. Charlie King KING ASSET MANAGEMENT 265 Lytton Avenue, Suite 304 Palo Alto, California 94301 RE: GEOTECHNICAL INVESTIGATION

OFFICE BUILDING AND PARKING STRUCTURE

455 HICKEY BOULEVARD DALY CITY, CALIFORNIA

Dear Mr. King:

We are pleased to present the results of our geotechnical investigation for the above referenced project. Our report includes a description of the geotechnical and seismic aspects of the site along with our conclusions and geotechnical recommendations for design of the proposed Office Building and Parking Structure in Daly City, California.

We refer you to the text of this report for detailed recommendations. If you have any questions concerning our findings, please call us and we will be glad to discuss them with you.

Very truly yours,

TRC

Mustafa Dogan, P.E., G.E. Senior Geotechnical Engineer

SML:MBD:AC

Copies: Addressee (2 and email)



Geotechnical Investigation

Office Building and Parking Structure
455 Hickey Boulevard
Daly City, California

Report No. 434432 has been prepared for:

King Asset Management

265 Lytton Avenue, Suite 304

August 20, 2021

Mustafa B. Dogan, P.E., G.E. Senior Geotechnical Engineer

Alberto Cortez, E.I.T.

Senior Staff Engineer

for Scott M. Leck, P.E., G.E.
Principal Geotechnical Engineer
Quality Assurance Reviewer

TABLE OF CONTENTS

| 1.0 | INTRO | DDUCTION | 1 |
|-----|-------|--|----|
| | 1.1 | Project Description | 1 |
| | 1.2 | Scope of Services | 2 |
| 2.0 | SITE | CONDITIONS | 2 |
| | 2.1 | Site Reconnaissance | 2 |
| | 2.2 | Exploration Program | 2 |
| | 2.3 | Subsurface Conditions | 2 |
| | 2.4 | Ground Water | 3 |
| 3.0 | GEOL | OGIC HAZARDS | 3 |
| | 3.1 | Fault Rupture | 3 |
| | 3.2 | Maximum Estimated Ground Shaking | 3 |
| | 3.3 | Future Earthquake Probabilities | 4 |
| | 3.4 | Liquefaction | 4 |
| | 3.5 | Dry Seismic Settlement | 4 |
| | 3.6 | Lateral Spreading | 4 |
| | 3.7 | Flooding | 5 |
| 4.0 | CORR | OSION EVALUATION | 5 |
| | | Table 2. Results of Corrosivity Testing | |
| | | Table 3. Relationship Between Soil Resistivity and Soil Corrosivity | |
| | | Table 4. Relationship Between Sulfate Concentration and Sulfate Exposure | |
| 5.0 | CONC | LUSIONS AND RECOMMENDATIONS | 6 |
| | 5.1 | Primary Geotechnical Concerns | 6 |
| | | 5.1.1 Strong Seismic Shaking | - |
| | | 5.1.2 Demolition Debris | |
| | | 5.1.3 Difficult Excavation in Bedrock | 7 |
| | | 5.1.4 Differential Bearing/Settlement | |
| | | 5.1.5 Corrosion Potential of Near-Surface Soils | |
| | | 5.1.6 Excavation Support | |
| | 5.2 | Plans, Specifications, and Construction Review | |
| 6.0 | EARTI | HWORK | |
| | 6.1 | Clearing and Site Preparation | |
| | 6.2 | Removal of Existing Fill | |
| | 6.3 | Abandoned Utilities | |
| | 6.4 | Bedrock Rippability | |
| | 6.5 | Remedial Grading for Soil/Fill and Bedrock Transitions | |
| | 6.6 | Subgrade Preparation | |
| | 6.7 | Material for Fill | |
| | 6.8 | Reuse of On-site Recycled Materials | |
| | 6.9 | Compaction | |
| | 6.10 | Wet Soils and Wet Weather Conditions | |
| | 6.11 | Trench Backfill | |
| | 6.12 | Temporary Slopes and Trench Excavations | |
| | 6.13 | Temporary Shoring Support System | |
| | | Table 4. Temporary Shoring System Design Parameter | |
| | 6.14 | Surface Drainage | _ |
| | 6.15 | Landscaping Considerations | _ |
| | 6.16 | Construction Observation | 14 |



| 7.0 | FOUN | NDATIONS | 14 | | | |
|------|--------------|--|----|--|--|--|
| | 7.1 | ASCE 7-16 Site Class and Site Seismic Coefficients | 14 | | | |
| | | Table 5. ASCE 7-16/CBC 2019 Site Class and Site Seismic Coefficients | 15 | | | |
| | 7.2 | Drilled Pier Foundations | 15 | | | |
| | | 7.2.1 Axial Capacity | 15 | | | |
| | | 7.2.2 Estimated Settlement | 16 | | | |
| | | 7.2.3 Lateral Response | 16 | | | |
| | | Table 6. LPILE Geotechnical Parameters | 16 | | | |
| | | 7.2.4 Drilled Pier Construction Considerations | 16 | | | |
| | 7.3 | Garage Floor Slabs | 17 | | | |
| | 7.4 | Moisture Protection Considerations for Slabs-on-Grade | 18 | | | |
| 8.0 | RETA | INING WALLS | 18 | | | |
| | 8.1 | Lateral Earth Pressures | 18 | | | |
| | 8.2 | Seismic Lateral Earth Pressures | 19 | | | |
| | 8.3 | Drainage | 19 | | | |
| | 8.4 | Backfill | 19 | | | |
| | 8.5 | Foundation | 19 | | | |
| | 8.6 | Lateral Loads on Footings | 20 | | | |
| 9.0 | PAVE | MENTS | 20 | | | |
| | 9.1 | Asphalt Concrete | 20 | | | |
| | | Table 7. Recommended Asphalt Concrete Pavement Design Alternatives | 21 | | | |
| | | Pavement Components | 21 | | | |
| | | Design R–Value = 15 | 21 | | | |
| | 9.2 | Exterior Portland Cement Concrete (PCC) Pavements | 21 | | | |
| | | Table 8. Recommended Minimum PCC Pavement Thickness | 21 | | | |
| | 9.3 | Pavement Cutoff | 22 | | | |
| | 9.4 | Asphalt Concrete, Aggregate Base and Subgrade | | | | |
| | 9.5 | Flatwork and Sidewalks | | | | |
| 10.0 | LIMITATIONS | | | | | |
| 11.0 | REFERENCES23 | | | | | |

FIGURE 1 — VICINITY MAP

FIGURE 2A — SITE PLAN - SCENARIO 1

FIGURE 2B — SITE PLAN - SCENARIO 2

FIGURE 3 — REGIONAL FAULT MAP

FIGURE 4A — ULTIMATE AXIAL CAPACITY PLOT, 1-FOOT DRILLED PIER

FIGURE 4B — ULTIMATE AXIAL CAPACITY TABLE, 1-FOOT DRILLED PIER

APPENDIX A — FIELD INVESTIGATION

APPENDIX B — LABORATORY PROGRAM

APPENDIX C — PREVIOUS INVESTIGATION BY LOWNEY ASSOCIATES



GEOTECHNICAL INVESTIGATION OFFICE BUILDING AND PARKING STRUCTURE 455 HICKEY BOULEVARD DALY CITY, CALIFORNIA

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed Office Building and Parking Structure to be constructed at 455 Hickey Boulevard in Daly City, California. The site location is shown on the Vicinity Map, Figure 1. The purpose of our investigation was to evaluate the geologic and subsurface conditions and to provide geotechnical recommendations for design of the proposed project.

As you know our predecessor company, Lowney Associates, prepared a geotechnical report for the site titled, "Geotechnical Investigation, Serramonte Office Building Complex, Daly City, California" dated July 18, 1979. In addition to our current explorations, selected previous borings at the site were also used as the basis in preparation of this geotechnical report.

We received and/or reviewed the following:

- An application submittal titled "455 Hickey Boulevard, Preliminary Application Submittal" dated December 18, 2020 prepared by DES Architects + Engineers.
- A cross-section titled "455 Hickey Blvd, Medical Office Building, Cross Section A," undated.
- A cross-section titled "455 Hickey Blvd, Medical Office Building, Cross Section B," undated.
- A scope of work titled "Draft Geotechnical Consulting Work Scope for Proposed Office Building and Related Site Improvements at Hickey, Daly City, CA" undated document.

1.1 Project Description

We understand that the project consists of two development scenarios as shown on the documents provided. In Scenario 1, proposed improvements are to consist of the installation of an approximately 180,000 square foot 6-story medical office building over 3 levels of podium parking and a new 3-level parking structure. In Scenario 2, proposed improvements are to consist of the installation of two tech office buildings, approximately 180,000 square foot 6-story building over 3 levels of podium parking and approximately 100,000 square foot 4-story office building over 4 levels of podium parking. The layout of the proposed development scenarios are shown on the Site Plan, Figure 2A and 2B.

Based on the planned improvements and topographic plan provided, to accommodate for the proposed Scenario 1 and 2, cuts of up to approximately 47 feet will need to be made into the existing slope along the southwest, south, and northeast, and east portion of the site with the cut depth increasing towards the east portion of the site. Additionally, fills of up to approximately 5 feet will need to be made along the northwest portion of the site.

Structural loads have not been provided to us; therefore, we assumed that structural loads will be representative for this type of construction.



1.2 Scope of Services

Our scope of services was presented in our agreements with you dated March 11, 2021. To accomplish this work, we have provided the following services:

- Exploration of subsurface conditions by drilling four borings in the areas of the proposed development and retrieving soil samples for observation and laboratory testing.
- Evaluation of the physical and engineering properties of the subsurface soils by visually classifying the samples and performing various laboratory tests on selected samples.
- Engineering analysis to evaluate structure foundations, site earthwork, slabs-on-grade, retaining walls, pavements.
- Preparation of this report to summarize our findings and to present our conclusions and recommendations.

2.0 SITE CONDITIONS

2.1 Site Reconnaissance

Our Staff Engineer performed a reconnaissance of the site on July 23, 2021. The project site is bordered by Serra Lane and Serravista Avenue to the south, Hickey Boulevard to the north, Serravista Avenue to the west, and Highway 280 to the east. At the time of the reconnaissance the project site was occupied by an existing parking structure and a multi-story building on the northern half of the site with an at-grade parking lot and driveway along the southern half of the site. The west and east sides of the parking lot and driveway are occupied by trees with an approximately 2:1 (horizontal:vertical) slope that is sloping east to west and west to east from the parking lot and driveway. Based on the topographic map provided, in the proposed project area the elevations range from approximately 338 feet to 390 feet with the site generally sloping north to south.

Additionally, the field exploration locations were marked, and notification was provided to Underground Service Alert (USA) prior to beginning fieldwork to identify public and/or private underground utilities. We also contracted a private utility locator to reduce the risk of damaging unidentified underground utilities.

2.2 Exploration Program

Subsurface exploration was performed on July 30' 2021 and August 6, 2021 using conventional, truck-mounted hollow-stem auger drilling equipment to investigate, sample, and log subsurface soils. Four hollow-stem auger exploratory borings were drilled to depths ranging from approximately 30 to 50 feet.

Our borings were permitted and backfilled in accordance with San Mateo County Environmental Health Services Division guidelines. The approximate locations of the borings are shown on the Site Plan, Figures 2A and 2B. The logs of the borings and details regarding our field investigation are included in Appendix A; laboratory tests are discussed in Appendix B.

2.3 Subsurface Conditions

All of our borings encountered a pavement section consisting of 2 to 4 inches of asphalt concrete over 4 to 6 inches of aggregate base. Below the pavement section, our borings generally encountered medium dense to very dense silty sand to depths of approximately 2½ to 5 feet below the ground surface (bgs)



underlain by sandstone (Merced Formation) with blow counts ranging from 17 to 75 blows per foot consisting of completely weathered, soft to very soft, and friable to a depth of 50 feet, the maximum depth explored.

Based on the previous subsurface explorations (Lowney Associates, 1979), borings EB-1 through EB-8 generally encountered fill consisting of firm to very stiff sandy silt with some interbedded layers of medium dense to very dense silty sand to depths ranging from approximately $8\frac{1}{2}$ to $18\frac{1}{2}$ feet bgs. Below the depths of $8\frac{1}{2}$ to $18\frac{1}{2}$ feet, sandstone (Merced Formation) was encountered with blow counts ranging from 6 to more than 50 blows per foot to a maximum depth of approximately $51\frac{1}{2}$ feet bgs.

Two Plasticity Index (PI) tests were performed on samples collected during this study from Borings EB-3 and EB-4 at depths of approximately 2 feet resulting in PIs of 5 and 11, indicating low plasticity and expansion potential of the near surface soils. Results of these tests are presented on the boring logs and in Appendix B.

2.4 Ground Water

Free ground water was not encountered in any of our borings or previous borings (Lowney Associates, 1979) at the time of drilling to a depth of 51½ feet, the maximum depth explored.

The California Geological Survey (CGS) has no published data on historically high ground water levels (CGS, 2000) at the site. Based on the above information, we judged a ground water depth of 50 feet to be appropriate for design. Our borings were backfilled immediately after drilling. Fluctuations in the level of the ground water may occur due to variations in rainfall, underground drainage patterns, regional influences, and other factors not evident at the time the borings were drilled.

3.0 GEOLOGIC HAZARDS

A brief qualitative evaluation of geologic hazards was made during this investigation. Our comments concerning these hazards are presented below.

3.1 Fault Rupture

The San Francisco Bay Area is one of the most seismically active regions in the United States. The significant earthquakes that occur in the Bay Area are generally associated with crustal movement along well-defined active fault zones of the San Andreas Fault system, which regionally trend in a northwesterly direction.

The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (known formerly as a Special Studies Zone). The nearest known active faults are the Serra Fault and the San Andreas Fault, which are located approximately ¾-kilometer northeast and southwest of the project site. Fault rupture through the site, therefore, is not anticipated.

3.2 Maximum Estimated Ground Shaking

Based on Equation 11.8-1 of ASCE 7-16, we judge a maximum considered earthquake geometric mean peak ground acceleration of 1.12g to be appropriate for geotechnical analyses for the project site.



3.3 Future Earthquake Probabilities

Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when or where an earthquake will occur. The U.S. Geological Survey's Working Group on California Earthquake Probabilities (WGCEP, 2014) estimates there is a 72 percent chance of at least one magnitude 6.7 earthquake occurring in the San Francisco Bay region between 2014 and 2044. This result is an important outcome of WGCEP's work because any major earthquake can cause damage throughout the region. The 1989 Loma Prieta earthquake demonstrated this potential by causing severe damage in Oakland and San Francisco, more than 50 miles from the fault epicenter.

Although earthquakes can cause damage at a considerable distance, shaking will be very intense near the fault rupture. Therefore, earthquakes located in urbanized areas of the region have the potential to cause much more damage than the 1989 Loma Prieta earthquake.

3.4 Liquefaction

The site is not located within an area zoned by the State of California for seismically induced liquefaction hazard (CGS, 2000). During cyclic ground shaking, such as earthquakes, cyclically-induced stresses may cause increased pore water pressures within the soil matrix, which results in liquefaction. Liquefied soil may lose shear strength that may lead to large shear deformations and/or flow failure (Youd et al., 2001). Liquefied soil can also settle as pore pressures dissipate following an earthquake. Limited field data is available on this subject; however, settlement on the order of 2 to 3 percent of the thickness of the liquefied zone has been measured in some cases.

Soils most susceptible to liquefaction are loose to moderately dense, saturated, non-cohesive soils with poor drainage, such as sands and silts with interbedded or capping layers of relatively low permeability soil.

Groundwater was not encountered in our explorations to a depth of 50 feet. Therefore, we judge the risk of liquefaction at the project site to be low.

3.5 Dry Seismic Settlement

If near-surface soils vary in composition both vertically and laterally, strong earthquake shaking can cause non-uniform densification of loose to medium dense cohesionless soil strata. This results in movement of the near-surface soils. Our explorations encountered some medium dense silty sand layers and completely weathered sandstone layers with relatively low blow counts.

We performed dry sand settlement calculations following Tokimatsu and Seed method (1987) for the medium dense silty sand and sandstone layers. Our calculations indicated that the medium dense granular layers encountered during our investigation may densify and settle on the order of less than 1/4-inch.

3.6 Lateral Spreading

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvial material toward an open or "free" face such as an open body of water, channel, or excavation. In soils this movement is generally due to failure along a weak plane and may often be associated with liquefaction. As cracks develop within the weakened material, blocks of soil displace laterally towards the open face. Cracking and lateral movement may gradually propagate away from the face as blocks continue to break free. Generally, failure in this mode is analytically unpredictable since it is difficult to evaluate where the first tension crack will occur.



Colma Creek is located approximately 1 mile east of the project site. Additionally, because of the low probability for liquefaction, the probability of lateral spreading occurring at the site during a seismic event is low.

3.7 Flooding

The site is located in a Federal Emergency Management Agency Zone X (FEMA 2012) which is defined as "area of minimal flood hazard."

4.0 CORROSION EVALUATION

To evaluate the corrosion potential of the subsurface soils at the site, we submitted four samples collected during our subsurface investigations to an analytical laboratory for pH, resistivity, soluble sulfate and chloride content testing. The results of these tests are summarized in Table 2 below.

Table 2. Results of Corrosivity Testing

| Sample | Depth (feet) | Chloride (mg/kg) | Sulfate (mg/kg) | рН | Resistivity (ohm-cm) | Estimated Corrosivity Based on Resistivity | Estimated Corrosivity Based on Sulfates |
|--------|-----------------|---------------------|--------------------|-----|-------------------------|---|--|
| EB-1 | 3.5 | 21 | 143 | 6.7 | 3,236 | Moderately | Negligible |
| EB-1 | 5.5 | 13 | 120 | 6.7 | 5,476 | Mildly | Negligible |
| EB-4 | 2.0 | 17 | 1,520 | 4.3 | 1,507 | Severely | Moderately |
| EB-4 | 4.0 | 14 | 5,577 | 4.6 | 1,093 | Severely | Severely |

Notes: 1. mg/kg = milligrams per kilogram.

Many factors can affect the corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor. Based on classification developed by William J. Ellis (1978), the approximate relationship between soil corrosiveness was developed as shown in Table 3 below.

Table 3. Relationship Between Soil Resistivity and Soil Corrosivity

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

Chloride and sulfate ion concentrations and pH appear to play secondary roles in affecting corrosion potential. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried metallic improvements or reinforced concrete structures. Sulfate ions in the soil can lower the soil resistivity and can be highly aggressive to Portland cement concrete (PCC) by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. Soils containing high sulfate content could also cause corrosion of the reinforcing steel in concrete. Table



4.2.1 of the American Concrete Institute (ACI, 2008) provides requirements for concrete exposed to sulfate-containing solutions as summarized in Table 4.

Table 4. Relationship Between Sulfate Concentration and Sulfate Exposure (Table 4.2.1 of ACI)

| Water-Soluble Sulfate (SO ₄) in soil, ppm | Sulfate Exposure |
|---|-----------------------|
| o to 1,000 | Negligible |
| 1,000 to 2,000 | Moderate ¹ |
| 2,000 to 20,000 | Severe |
| over 20,000 | Very Severe |

¹⁼ seawater

Acidity is an important factor of soil corrosivity. The lower the pH (the more acidic the environment), the higher will the soil corrosivity be with respect to buried metallic structures. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures due to protective surface films which form on steel in high pH environments. A pH between 5 and 8.5 is generally considered relatively passive from a corrosion standpoint.

As shown in Table 2, the soil resistivity results ranged from 1,093 to 5,476 ohm-centimeters. Based on these results and the resistivity correlations presented in Table 3, the corrosion potential to buried metallic improvements may be characterized as mildly to severely corrosive. We recommend that a corrosion protection engineer be consulted about appropriate corrosion protection methods for buried metallic materials.

Based on our previous experience and Table 4.2.1 of the ACI, it is our opinion that sulfate exposure to PCC may be considered negligible to severely for the native subsurface materials sampled.

5.0 CONCLUSIONS AND RECOMMENDATIONS

From a geotechnical engineering viewpoint, the proposed improvements may be constructed as planned, in our opinion, provided the design and construction are performed in accordance with the recommendations presented in this report.

5.1 Primary Geotechnical Concerns

The primary geotechnical concerns at the site are as follows:

- Strong seismic shaking
- Demolition of the existing building prior to site development
- Difficult excavation in bedrock
- Differential settlement between foundations bearing on soil, fill, and bedrock transitions
- Corrosion potential of the near-surface soils
- Excavation support

We have prepared a brief description of the issues and present typical approaches to manage potential concerns associated with the long-term performance of the development.



5.1.1 Strong Seismic Shaking

We recommend that, at a minimum, the proposed improvements be designed in accordance with the seismic design criteria presented in Table 5.

5.1.2 Demolition Debris

Construction debris is anticipated because of the site demolition required prior to site grading. The debris should be either: 1) collected and off-hauled to an appropriate facility prior to beginning the earthwork for the project, or 2) the concrete crushed and re-used as fill at the site. If generated, recycled materials containing asphalt concrete (AC) should not be used below interior floor slabs, therefore if recycled materials are proposed to be re-used beneath interior floor slabs, AC pavements should be segregated from the debris. It has been our experience that some debris will remain in the soil on-site after the demolition contractor has completed their work. Therefore, it should be anticipated that some debris would be encountered in excavations for underground utilities and foundations. Some coordination between the demolition contractor, grading contractor and geotechnical engineer is needed to identify the scope of the excavation backfill and other similar work items. Recommendations for re-use of recycled materials are presented in the Earthwork section of this report.

5.1.3 Difficult Excavation in Bedrock

Based on the proposed building scenarios and existing site grades, grading and utility trenches will involve excavations into the sandstone bedrock. We recommend that local grading and/or underground utility contractors experienced in rock excavation methods be contacted to aid in determining the efficiency, time and costs associated with excavations in this type of bedrock. Please refer to the "Bedrock Rippability" section below.

5.1.4 Differential Bearing/Settlement

It is possible that differential settlement could occur due to the differential thickness of soil/engineered fill above the bedrock surface. As discussed, bedrock was encountered in all our borings at depths ranging from approximately 2½ to 5 feet below the existing grade surface. The proposed buildings are planned to have cuts as much as 47 feet and fills up to approximately 5 feet, which we anticipate will span between the soil/engineered fill and bedrock transitions.

Because of geotechnical characteristics of bedrock and soil/engineered fill are different, the long-term performance of these materials is also different. For instance, engineered fill materials, even if well compacted, are typically more compressible than bedrock materials and as a result can experience a greater amount of settlement. Foundations constructed over engineered fill will be subject to long-term settlement. Even well-compacted fills may experience minor long-term settlements due to secondary strains or hydrocompression. In addition, shallow foundations constructed over engineered fill and bedrock transitions may experience differential movements under static and seismic loading conditions. To minimize the effects of differential settlement across fill and bedrock transitions, we recommend that building pads be over-excavated to provide a uniform soil cushion foundation support.

Additional recommendations addressing fill and bedrock transition concerns are presented in the "Earthwork" and "Foundation" sections of this report.



5.1.5 Corrosion Potential of Near-Surface Soils

As discussed above, the corrosion potential to buried metallic improvements constructed within the native soils may be characterized as mildly to severely corrosive. Sulfate exposure to PCC may be considered negligible to severely within the native soils. A qualified corrosion engineer should be contacted to provide specific recommendations regarding corrosion protection for buried metal pipe or buried metal pipe fittings.

5.1.6 Excavation Support

The excavation for the proposed buildings may be supported by several methods including tiebacks, soldier beams and wood lagging or temporary slopes if space is adequate. The choice should be left to the contractor's judgment since economic considerations and/or the individual contractor's construction experience may determine which method is more economical and/or appropriate. Support of any adjacent existing structures without distress should also be the contractor's responsibility. We recommend that the contractor forward his plan for the support system to the structural engineer and geotechnical engineer for pre-construction review. In addition, it should be the contractor's responsibility to undertake a pre-construction survey with benchmarks and photographs of the adjacent properties.

5.2 Plans, Specifications, and Construction Review

We recommend that our firm perform a plan review of the geotechnical aspects of the project design for general conformance with our recommendations. In addition, subsurface materials encountered in the relatively small diameter, widely spaced borings may vary significantly from other subsurface materials on the site. Therefore, we also recommend that a representative of our firm observe and confirm the geotechnical specifications of the project construction. This will allow us to form an opinion about the general conformance of the project plans and construction with our recommendations. In addition, our observations during construction will enable us to note subsurface conditions that may vary from the conditions encountered during our investigation and, if needed, provide supplemental recommendations. For the above reasons, our geotechnical recommendations are contingent upon our firm providing geotechnical observation and testing services during construction.

6.0 EARTHWORK

6.1 Clearing and Site Preparation

The proposed project area should be cleared of all surface and subsurface improvements to be removed and deleterious materials including existing building foundations, slabs, irrigation lines, utilities, fills, pavements, debris, designated trees, shrubs, and associated roots. Abandonment of existing buried utilities is discussed below. Excavations extending below the planned finished site grades should be cleaned and backfilled with suitable material compacted as recommended in the "Compaction" section of this report. We recommend that backfilling of holes or pits resulting from demolition and removal of existing building foundations, buried structures or other improvements be carried out under our observation and that the backfill be observed and tested during placement.

After clearing, any vegetated areas within the proposed improvements should be stripped to sufficient depth to remove all surface vegetation and topsoil containing greater than 3 percent organic matter by weight. The actual stripping depth required depends on site usage prior to construction and should be established in the field by us at the time of construction. The stripped materials should be removed from the site or may be stockpiled for use in landscaped areas, if desired.



6.2 Removal of Existing Fill

If undocumented fill is encountered, it should be removed down to the native soil. If the fill material meets the requirements in the "Material for Fill" section below, it may be reused an engineered fill. Side slopes of fill removal excavations in building and pavement areas should be sloped at inclinations no steeper than 3:1 (horizontal:vertical) to minimize abrupt variations in fill thickness. All fill should be compacted in accordance with the recommendations for fill presented in the "Compaction" section of this report.

6.3 Abandoned Utilities

Abandoned utilities within the proposed project areas should be removed in their entirety. As an alternative, it may be feasible to abandon underground utilities in-place within the proposed project areas provided the utility does not conflict with new improvements, is completely grouted, and previous fills associated with the utility do not pose a risk to the structures. Existing underground utilities outside the proposed project areas may be removed or abandoned in-place by grouting or plugging the ends with concrete. The decision to abandon in-place versus removal should be based on the level of risk associated with the particular utility line.

Fills associated with underground utilities abandoned in-place may have an increased potential for settlement, and partially grouted or plugged pipelines will have a potential risk of collapse that may result in ground settlement, soil piping and leakage of pipeline constituents. The potential risks are relatively low for small diameter pipes (4 inches or less) above the ground water table and increasingly higher with increasing diameter.

6.4 Bedrock Rippability

The proposed project grading involves excavations up to approximately 47 feet, which will have excavation into bedrock. The hardness of the sandstone will be variable, but based on the results of our subsurface exploration, we believe earthwork can generally be performed with conventional equipment with occasional added effort in areas with hard rock. These areas could be very difficult to excavate with backhoe equipment and therefore, hydraulic jacking or hammering with a hoe ram may be necessary in these hard rock areas.

We recommend that local grading and/or underground utility contractors experienced in rock excavation methods be contacted to aid in determining the efficiency, time and costs associated with excavations in this type of bedrock.

6.5 Remedial Grading for Soil/Fill and Bedrock Transitions

To reduce the potential for differential movement beneath shallow foundations and/or slabs-on-grade, we recommend that the building pad areas be over-excavated to provide a uniform cushion of compacted material for subgrade support. The fill thickness should be at least 24 inches below the footing bearing level, and should extend at least 5 feet laterally beyond the building footprints.

6.6 Subgrade Preparation

After the site has been properly cleared, stripped and necessary excavations have been made, exposed surface soils in those areas to receive fill (non-building areas) or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the recommendations for fill presented in the "Compaction" section. The finished compacted subgrade should be firm and non-yielding under the weight of compaction equipment.



6.7 Material for Fill

All on-site soils below the stripped layer having an organic content of less than 3 percent by weight are suitable for use as fill at the site. In general, fill material should not contain rocks or lumps larger than 6 inches in greatest dimension, with 15 percent or less larger than 2½ inches in the greatest dimension.

Import fill material should be inorganic, have a PI of 20 or less and should have sufficient binder to reduce the potential for sidewall caving of foundation and utility trenches. Non-expansive fill (NEF) should have a PI of 15 or less. Samples of the proposed import fill should be submitted to us at least 10 working days prior to delivery to the site to allow for visual review and laboratory testing. This will allow us to evaluate the general conformance of the import fill with our recommendations.

Consideration should also be given to the environmental characteristics and corrosion potential of any imported fill. Suitable documentation should be provided for import material. In addition, it may be appropriate to perform laboratory testing of the environmental characteristics and corrosion potential of imported materials. Import soils should not be more corrosive than the on-site native materials, including pH, soluble sulfates, chlorides and resistivity.

6.8 Reuse of On-site Recycled Materials

Some asphalt concrete/aggregate base grindings may be generated during removal of any existing pavements. If it is desired to reuse the grindings for new site pavement structural support, we recommend the asphalt concrete be pulverized and mixed with the underlying aggregate base to meet Caltrans Class 2 Aggregate Base requirements. If laboratory testing of the recycled material indicates that it meets Caltrans Class 2 specifications, it may be used as Class 2 Aggregate Base beneath pavements and sidewalks. Recycled material containing asphalt concrete grindings should not be used below building areas. Laboratory testing may be performed on initial grindings generated to evaluate the material further and refine the pavement recommendations.

6.9 Compaction

All fill, as well as scarified surface soils in those areas to receive fill, should be uniformly compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D1557, latest edition, at a moisture content 1 to 2 percent over the laboratory optimum. Fill should be placed in lifts no greater than 8 inches in uncompacted thickness. Each successive lift should be firm and relatively non-yielding under the weight of construction equipment.

In pavement areas, the upper 6 inches of subgrade and full depth of aggregate base should be compacted to at least 95 percent relative compaction (ASTM D1557, latest edition). Aggregate base and all import soils should be compacted at a moisture content near the laboratory optimum moisture content.

6.10 Wet Soils and Wet Weather Conditions

Earthwork such as subgrade preparation, fill placement and trench backfill may be difficult for soil containing high moisture content or during wet weather. If the soil is significantly above its optimum moisture content, it will become soft, yielding, and difficult to compact. Based on the results of our laboratory tests, the in-situ moisture contents of the near surface soils are generally near to above optimum moisture contents. If saturated soils are encountered, aerating or blending with drier soils to achieve a workable moisture content may be required. We recommend that earthwork be performed during periods of suitable weather conditions, such as the "summer" construction season.



There are several alternatives to facilitate subgrade preparation, fill placement and trench backfill if the soil is wet or earthwork is performed during the wet winter season.

- Scarify and air dry until the fill materials have a suitable moisture content for compaction,
- Over-excavate the fill and replace with suitable on-site or import materials with an appropriate moisture content,
- Install a layer of geo-synthetic (geotextile or geogrid) to reduce surface yielding and bridge over soft fill,
- Chemically treat the higher moisture content soils with quicklime (CaO), kiln-dust, or cement to reduce the moisture content and increase the strength of the fill.

The implementation of these methods should be reviewed on a case-by-case basis so that a cost-effective approach may be used for the specific conditions at the time of construction.

6.11 Trench Backfill

Bedding and pipe embedment materials to be used around underground utility pipes should be well graded sand or gravel conforming to the pipe manufacturer's recommendations and should be placed and compacted in accordance with project specifications, local requirements of the governing jurisdiction. General fill to be used above pipe embedment materials should be placed and compacted in accordance with local requirements or the recommendations contained in this section, whichever is more stringent.

On-site soils may be used as general fill above pipe embedment materials provided, they meet the requirements of the "Material for Fill" section of this report. General fill should be placed in lifts not exceeding 8 inches in uncompacted thickness and should be compacted to at least 90 percent relative compaction (ASTM D1557, latest edition) by mechanical means only. Water jetting of trench backfill should not be allowed. The upper 6 inches of general fill in all pavement areas subject to wheel loads should be compacted to at least 95 percent relative compaction.

Utility trenches located adjacent to footings should not extend below an imaginary 1:1 (horizontal:vertical) plane projected downward from the footing bearing surface to the bottom edge of the trench. Where utility trenches will cross beneath footing bearing planes, the footing concrete should be deepened to encase the pipe or the utility trench should be backfilled with sand/cement slurry or lean concrete within the foundation-bearing plane.

Where relatively higher permeability sand or gravel backfill is used in trenches through lower permeability soils, we recommend that a cut-off plug of compacted clayey soil or a 2-sack cement/sand slurry be placed where such trenches enter the building and pavement areas. This would reduce the likelihood of water entering the trenches from the landscaped areas and seeping through the trench backfill into the building and pavement areas and coming into contact with the subgrade soils.

6.12 Temporary Slopes and Trench Excavations

The contractor should be responsible for all temporary slopes and trenches excavated at the site and design of any required temporary shoring. Shoring, bracing, and benching should be performed by the contractor in accordance with the strictest governing safety standards. On a preliminary basis, site soils can be classified as Type B based on soil classification by OSHA. Therefore, a maximum slope 1:1



(horizontal: vertical) should be anticipated. A TRC representative should be retained to verify soil conditions in the field at the time of the excavation.

6.13 Temporary Shoring Support System

As previously discussed, excavations on the order of up to approximately 47 feet are planned to construct the proposed buildings. The excavations could potentially be temporarily supported by several methods including tiebacks, soil nailing, braced shoring, temporary slopes if space is adequate, or potentially other methods. Where shoring is required, restrained shoring will most likely be necessary to limit deflections and disruption to nearby improvements. It has been our experience that cantilever shoring might be feasible for temporary shoring to a height of about 10 to 13 feet where allowable deflections are limited. The choice of shoring method should be left to the contractor's judgment since economic considerations and/or the individual contractor's construction experience may determine which method is more economical and/or appropriate. However, other factors such as the location of nearby utilities and encroachment on adjacent properties may influence the choice of support.

The temporary shoring should be designed for additional surcharges due to adjacent loads such as from construction vehicles and street traffic. To prevent excessive surcharging of the walls, we recommend that heavy loads such as construction equipment and stockpiles of materials be kept at least 30 feet from the top of the excavations. If this is not possible, the shoring must be designed to resist the additional anticipated lateral loads. Shoring systems should be designed with sufficient rigidity to prevent detrimental lateral displacements. Minimum geotechnical parameters for design of a temporary shoring system are given in Table 4.

| Design Parameter | Design Value (psf) |
|---|-----------------------------|
| Minimum Lateral Wall Surcharge ¹ | 120 psf |
| Earth Pressure – Cantilever Wall | 40 pcf |
| Earth Pressure – Restrained Wall ² | |
| From ground surface to H/4 (ft) | Increase from 0 to 25H psf |
| Earth Pressure – Restrained Wall Below H/4 (ft) | |
| | Uniform pressure of 25H psf |
| Passive Pressure ³ | 300 pcf up to 1,500 psf max |

Table 4. Temporary Shoring System Design Parameter

Note: 1 For the upper 5 feet (minimum for incidental loading)

2 Where H equals height of excavation

3 Can assume to act over 2 times the diameter of soldier piles, neglecting the upper foot

To limit potential movements of the shoring system, the shoring designer and contractor should consider several design and construction issues. For the movements of shoring to be reduced, the designer will have to provide for a uniform and timely mobilization of the soil pressures. Tiebacks or internal bracing should be loaded to the design loads prior to excavation of the adjacent soil so that load induced strains in the retaining system will not result in the system moving toward the excavation. In addition, a relatively stiff shoring system should be designed to limit deflections under loading. In general, we recommend designing a shoring system to deflect less than 1-inch.

In addition, ground subsidence and deflections can be caused by other factors such as voids created behind the shoring system by over-excavation, soil sloughing, erosion of sand or silt layers due to perched water, etc. All voids behind the shoring system should be filled as soon as feasible by grouting to minimize potential problems during installation of the shoring system.



Since we drilled our borings with hollow-stem auger drilling equipment, we are not able to evaluate the potential for caving of on-site soils, which may become a factor during soldier pile and/or tieback installation. The contractor is responsible for evaluating excavation difficulties prior to construction. Pilot holes using proposed production drilling equipment may be prudent, to evaluate possible excavation difficulties such as caving soils, cobbles, boulders and/or other excavation difficulties.

In conjunction with the shoring installation, a monitoring program should be set up and carried out by the contractor to determine the effects of the construction on the adjacent buildings, street and other improvements such as sidewalks and utilities. As a minimum, we recommend horizontal and vertical surveying of reference points on the shoring and on the adjacent street, buildings and other improvements in addition to an initial crack survey. We also recommend that all supported, and/or sensitive utilities be located and monitored by the contractor. Reference points should be set up and read prior to the start of construction activities. Points should also be set on the shoring as soon as initial installations are made. Alternatively, inclinometers could be installed by the contractor at critical locations for a more detailed monitoring of shoring deflections. Surveys should be made at least once a week and more frequently during critical construction activities, or if significant deflections are noted. TRC can provide inclinometer materials and we have the equipment and software to read and analyze the data quickly.

This report is intended for use by the design team. The contractor should perform additional subsurface exploration and/or geotechnical studies as they deem necessary for the chosen shoring system. The contractor is also responsible for site safety and the means and methods of construction, including temporary shoring. Temporary shoring must be designed by a licensed California Civil or Structural Engineer. Prior to construction, we recommend that the contractor forward his plan for the support system to the structural engineer and geotechnical engineer for preconstruction review.

6.14 Surface Drainage

Positive surface water drainage gradients, at least 2 percent in landscaping and 0.5 percent in pavement areas, should be provided to direct surface water away from foundations and slabs towards suitable discharge facilities. Ponding of surface water should not be allowed on or adjacent to structures, slabs-on-grade, or pavements. Roof runoff should be directed away from foundation and slabs-on-grade. Downspouts may discharge onto splash-blocks provided the area is covered with concrete slabs or asphalt concrete pavements.

6.15 Landscaping Considerations

We recommend restricting the amount of surface water infiltrating these soils near structures and slabs-on-grade. This may be accomplished by:

- Selecting landscaping that requires little or no watering, especially within 3 feet of structures, slabs-on-grade, or pavements,
- Using low flow rate sprinkler heads, or drip irrigation systems
- Regulating the amount of water distributed to lawn or planter areas by installing timers on the sprinkler system,
- Providing surface grades to drain rainfall or landscape watering to appropriate collection systems and away from structures, slabs-on-grade, or pavements,
- Preventing water from draining toward or ponding near building foundations, slabs-on-grade, or pavements, and



Avoiding open planting areas within 3 feet of the building perimeters.

We recommend that the landscape architect consider these items when developing the landscaping plans.

6.16 Construction Observation

A representative from our company should observe the geotechnical aspects of the grading and earthwork for general conformance with our recommendations including site preparation, selection of fill materials, and the placement and compaction of fill. To facilitate your construction schedule we request sufficient notification (48 hours) for site visits. The project plans and specifications should incorporate all recommendations contained in the text of this report.

7.0 FOUNDATIONS

Based on our investigation, the proposed structures may be supported on drilled pier foundations, which will be able to support the structures with only minor settlements. Foundation recommendations are discussed in the sections below.

7.1 ASCE 7-16 Site Class and Site Seismic Coefficients

The 2019 California Building Code (CBC) outlines the procedure for seismic design of a structure. Based on subsurface explorations, the site is generally underlain by medium dense to very dense sand and bedrock, which correspond to a soil profile type D. The 2019 CBC requires that a site-specific ground motion study be performed in accordance with Section 11.4.8 of ASCE 7-16 for Site Class D sites with a mapped S_2 value greater than or equal 0.2.

However, Section 11.4.8 of ASCE 7-16 provides an exception from a site-specific ground motions study for certain structures on Site Class D. It is our understanding that requirements in Exception Note No. 2 in Section 11.4.8 of ASCE 7-16 may apply to this project. If the exception does not apply, a site-specific ground motion hazard analysis is required.



Table 5. ASCE 7-16/CBC 2019 Site Class and Site Seismic Coefficients

| Latitude: 37.66328 N Longitude: -122.46729 W | CBC Table/ Figure | Factor/ Coefficient | Value |
|---|----------------------|------------------------|------------------------------|
| Soil Profile Type | Section 1613.2.2 | Site Class | О |
| Mapped Spectral Response Acceleration for MCE at 0.2 second Period | Figure 1613.2.1(1) | S _s | 2.37 |
| Mapped Spectral Response Acceleration for MCE at 1 Second Period | Figure 1613.2.1(2) | Sı | 0.99 |
| Site Coefficient | Table 1613.2.3(1) | Fa | 1.00 |
| Site Coefficient | Table 1613.2.3(2) | F _v | Null – See Section 11.4.8 |
| Adjusted MCE Spectral Response Parameter | Equation 16-36 | S _{MS} | 2.37 |
| Adjusted MCE Spectral Response Parameter | Equation 16-37 | S _{M1} | Null – See Section 11.4.8 |
| Design Spectral Response Acceleration Parameter | Equation 16-38 | S _{DS} | 1.58 |
| Design Spectral Response Acceleration Parameter | Equation 16-39 | S _{D1} | Null – See Section 11.4.8 |

7.2 Drilled Pier Foundations

The proposed structures may be supported on reinforced concrete drilled cast-in-place, straight-shaft friction pier foundations. Drilled pier diameters should range in size in increments of 6 inches and 1 foot, for diameters up to 6 feet and larger than 6 feet, respectively. Recommendations for design and construction of drilled pier foundations are presented in the following sections of this report.

7.2.1 Axial Capacity

Axial loads on drilled piers should be supported by the frictional capacity of the pier without casing. End bearing was not considered in the axial capacity due to considerably more displacement that is needed to fully mobilize end bearing capacity. Displacement needed to fully mobilize end bearing is generally greater than the structural tolerance and beyond the movement required to mobilize side friction. In addition, the potential for loose materials to exist at the bottoms of the pier excavations during construction that cannot be effectively cleaned out is another factor for not including end bearing.

Figure 4A provides the ultimate axial downward capacity of a 1-foot diameter straight-sided drilled pier installed from the current grades under static conditions. Figure 4B tabulates the data from the capacity curve shown on Figure 4A. These values can be used for piers that are spaced at least 3 diameters apart. Federal Highway Administration (FHWA) procedures for design of drilled pier foundations (Brown et al., 2010) was used to compute the skin friction capacity. For evaluation of allowable downward capacity under static conditions, we recommend a safety factor of 3 be applied to the ultimate capacity. A one-third increase in the allowable capacity may be used for consideration of transient loads such as wind or seismic.

Ultimate uplift capacity of a straight sided drilled pier may be obtained by multiplying the downward capacity by a factor of o.8 and adding the weight of the foundation. A factor of safety of 1.5 may be used to obtain the allowable uplift capacity of a straight sided drilled pier.



Downward and uplift capacities for drilled piers with foundation diameters other than 1-foot may be obtained by multiplying the capacity for the 1-foot diameter pier as shown on Figure 4A by the actual pier diameter (in feet). The weight of the foundation is not included in the ultimate resistance shown on Figure 4A. The curve may be used for drilled pier foundations up to 7 feet in diameter that are spaced at least 3 diameters apart. For closer spacing, group effects may govern and the group capacity should be evaluated.

7.2.2 Estimated Settlement

Total static settlement of each drilled pier should be on the order of 0.1 percent of the pier diameter for a drilled pier designed and constructed in accordance with the recommendations presented in this report. We suggest allowing for about ½ inch of settlement to accommodate potential long-term settlement, construction issues, and some soil variability across the site. The majority of the settlement should occur during and shortly after application of the structure loads.

7.2.3 Lateral Response

Lateral load resistance for pier-supported structures may be developed through pier bending/soil interaction. The magnitude of the lateral load resistance is dependent upon many factors, including pier stiffness and embedment length, conditions of fixity at the pile cap, the physical properties of the surrounding soils, the tolerable top deflection and the yield moment capacity of the pier. The foundation engineer should account for any construction installation tolerances that could impact recommended embedment lengths.

Table 6 contains recommended input soil parameters for lateral response analysis of deep foundations using the LPILE computer program (by Ensoft, Inc., Version 2012). Program default values may be used for strain factor (E_{50}) and horizontal subgrade reaction (K).

| Depth (ft) | Modal P-Y Curve | Effective Unit Weight (pcf) | Cohesion, c (psf) | Internal Angle of Friction (degrees) |
|---------------|----------------------|--------------------------------------|----------------------|---|
| 0 to 20 | Sand (Reese, et al.) | 120 | - | 33 |
| 20 to 50 | Sand (Reese, et al.) | 120 | - | 36 |

Table 6. LPILE Geotechnical Parameters

7.2.4 Drilled Pier Construction Considerations

Based on the on-site soils, sandy soils may pose caving problems during drilled pier construction. The pier drilling contractor may need to use temporary straight-sided steel casing to maintain hole stability.

In addition, the contractor should expect difficult drilling and excavation, and should be prepared to use specialized heavy drilling, ripping, and excavating equipment including heavy torque locking Kelly drilling bars, core barrels, diamond-studded rock bits, hydraulic hammers and/or hoe-ram equipment in the sandstone.

Procedures provided in the FHWA manual on drilled shaft construction (Brown et al., 2010) are recommended to be followed by the contractor. We recommend that each pier excavations should be inspected and approved by a geotechnical engineer prior to installation of reinforcement. We recommend



a representative of TRC be present during drilled pier construction to observe and verify soil and excavation conditions prior to placing steel reinforcement or concrete.

Drilled pier excavations should be constructed in a continuous way to reduce the time from excavation of hole to placement of reinforcing steel and concrete placement to a minimum. Steel reinforcement and concrete should be placed on the same day of completion of each pier excavation. Drilling of adjacent holes that are closer than 4 diameters center to center should not be allowed until the concrete in the previous hole is set.

Concrete used for drilled pier construction should be discharged vertically into the drilled holes to reduce aggregate segregation. Under no circumstances during pier construction should concrete be allowed to free-fall against either the steel reinforcement or the sides of the excavation. Sufficient space should be provided in the pier reinforcement cage during fabrication to allow the insertion of a pump hose or tremie tube for concrete placement. The pier reinforcement cage should be installed and the concrete pumped immediately after drilling is completed.

In order to develop the design skin friction values provided above, concrete used for drilled pier construction should have a slump ranging from 4 to 6 inches if placed in a dry shaft without temporary casing, and from 6 to 8 inches if temporary casing or slurry drilling methods are used. If temporary steel casing is used, we recommend its removal from the hole as concrete is being placed. The bottom of the casing should be maintained below the top of the concrete during casing withdrawal and concrete placement operations. The concrete mix should be designed with appropriate admixtures and/or water/cement ratios to achieve these recommended slumps. Adding water to a conventional mix to achieve the recommended slump should not be allowed. For concrete mixes with slumps over 6 inches, vibration of the concrete during placement is generally not recommended as aggregate settlement may result in the lack of aggregate within the upper portion of the pile. Careful vibration of the concrete around anchor bolt assemblies is recommended.

If slurry drilling methods are used for drilled pier construction, concrete should be placed into the hole using tremie methods. Tremie concrete placement should be performed in accordance with American Concrete Institute (ACI) 304R. The tremie pipe should be rigid and remain several feet below the surface of the in-place concrete at all times to maintain a seal between the water or slurry and the fresh concrete. The upper concrete seal layer will likely become contaminated with excess water and/or soil as the concrete is placed and should be removed to expose uncontaminated concrete during or immediately following completion of concrete placement. It has been our experience that the concrete seal layer may be on the order of 3 to 5 feet thick but will depend on the pile diameter, amount of water seepage, and construction workmanship.

7.3 Garage Floor Slabs

The parking garage slabs should be at least 5 inches thick, have a compressive strength of at least 3,000 pounds per square inch (psi), and supported on at least 6 inches of Class 2 aggregate base compacted to at least 95 percent relative compaction. Adequate slab reinforcement should be provided to satisfy the anticipated use and loading requirements.

If desired to limit moisture rise through garage slabs, the guidelines presented in the "Moisture Protection Considerations" Section 7.4 below should be considered.



7.4 Moisture Protection Considerations for Slabs-on-Grade

Since the long-term performance of concrete slabs-on-grade depends to a large degree on good design, workmanship, and materials, the following general guidelines are presented for consideration by the developer, design team, and contractor. The purpose of these guidelines is to aid in producing a concrete slab of sufficient quality to allow successful installation of floor coverings and reduce the potential for floor covering failures due to moisture-related problems associated with the slab-on-grade construction. These quidelines may be supplemented, as necessary, based on the specific project requirements.

- A minimum 15-mil thick vapor barrier meeting minimum ASTM E 1745, Class A requirements should be placed directly below the slab. The vapor barrier should extend to the edge of the slab. At least 4 inches of free-draining gravel, such as ½-inch or ¾-inch crushed rock with no more than 5 percent passing the ASTM No. 200 sieve, should be placed below the vapor barrier to serve as a capillary break (no sand). The crushed rock should be consolidated in place with vibratory equipment. The vapor barrier should be sealed at all seams and penetrations.
- The concrete water/cement ratio should not exceed 0.45. Midrange plasticizers could be used to facilitate concrete placement and workability.
- Water should not be added after initial batching, unless the slump of the concrete is less than specified, and the resulting water/cement ratio will not exceed 0.45.
- Polishing the concrete surface with metal trowels should not be permitted.
- All concrete surfaces to receive any type of floor covering should be moist-cured for a minimum of 7 days. Moist curing methods may include frequent sprinkling, or using coverings such as burlap, cotton mats, or carpet. The covering should be placed as soon as the concrete surface is firm enough to resist surface damage. The covering should be kept continuously wet and not allowed to dry out during the required curing period.
- Water vapor emission levels and pH should be determined before floor installation as required by the manufacturer of the floor covering materials. Measurements and calculations should be made according to ASTM F1869-98 and F710-98 protocol.

The guidelines presented above are based on information obtained from various technical sources, including the American Concrete Institute (ACI), and are intended to present information that can be used to reduce potential long-term impacts from slab moisture infiltration. It should be noted that the application of these guidelines does not affect the geotechnical aspects of the foundation performance.

8.0 RETAINING WALLS

8.1 Lateral Earth Pressures

Any proposed retaining walls should be designed to resist lateral earth pressures from adjoining natural materials, backfill, and surcharge loads. Provided that adequate drainage is provided as recommended below, we recommend that walls restrained from movement at the top be designed to resist an equivalent fluid pressure of 45 pcf plus a uniform pressure of 8H pounds per square foot, where H is the distance in feet between the bottom of the footing and the top of the retained soil. Restrained walls should also be designed to resist an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface. Any unrestrained retaining walls with adequate drainage should be designed to resist an equivalent fluid pressure of 45 pcf plus one-third of any surcharge loads.



The above lateral earth pressures assume level backfill conditions and sufficient drainage behind the walls to prevent build-up of hydrostatic pressure from surface water infiltration and/or a rise in the ground water level. If adequate drainage is not provided, we recommend an equivalent fluid pressure of 40 pcf be added to the values recommended above for both restrained and unrestrained walls. Damp proofing of the walls should be included in areas where wall moisture and efflorescence would be undesirable.

8.2 Seismic Lateral Earth Pressures

Walls greater than 6 feet in height need to be designed for seismic lateral loading. For our analysis, we have assumed that the walls will have flat, non-sloping backfill. We used the Mononobe-Okabe approach to approximate the increased earth pressures induced by earthquakes. As discussed in Section 3.2 of our report, a peak ground acceleration of 1.12g is expected at the site. We performed calculations using this ground acceleration and estimated an additional seismic increment of 36.5 pcf to be applied to in addition to the static lateral earth pressures given in Section 8.1 for flexible walls. For restrained walls, under seismic conditions the total pressure to be used in analysis (seismic plus static) should be the greater of atrest pressure or the sum of the active pressure and the seismic increment acting in a triangular distribution. For unrestrained walls under seismic loading, the total pressure should be sum of the active pressure and the seismic increment acting in a triangular distribution.

8.3 Drainage

Adequate drainage may be provided by a subdrain system behind the walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 12 inches out from the wall and to within 2 feet of outside finished grade. Alternatively, ½- to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or equivalent. The upper 2 feet of wall backfill should consist of relatively low permeable compacted on-site clayey soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or Enkadrain drainage matting may be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. The drainage panel should be connected to the perforated pipe at the base of the wall, or to some other closed or through-wall system. Miradrain panels should terminate 24 inches from final exterior grade. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

8.4 Backfill

Where surface improvements will be located over the retaining wall backfill, backfill placed behind the walls should be compacted to at least 95 percent relative compaction using light compaction equipment. Where no surface improvements are planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced.

8.5 Foundation

Reinforced concrete retaining walls may be supported on reinforced concrete drilled cast-in-place, straight shaft friction pier foundations in accordance with the recommendations presented in the "Drilled Pier Foundations" section of this report. Lateral load resistance for the walls may be developed in accordance with the recommendations presented in the "Lateral Response."



Alternatively, reinforced concrete retaining walls may be supported on conventional continuous footings bearing on natural, undisturbed soil or compacted fill. All footings should have a minimum width of 18 inches and footing bottoms should extend at least 24 inches below lowest adjacent finished grade.

Footings constructed in accordance with the above recommendations would be capable of supporting maximum allowable bearing pressures of 2,000 pounds per square foot (psf) for dead loads, 3,000 psf for combined dead and live loads, and 4,000 psf for all loads including wind or seismic. These allowable bearing pressures are based upon factors of safety of 3.0, 2.0, and 1.5 for dead, dead plus live, and seismic loads, respectively.

These maximum allowable bearing pressures are net values; the weight of the footing may be neglected for design purposes. All footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

All continuous footings should be reinforced with top and bottom steel to provide structural continuity and to help span local irregularities. Footing excavations should be kept moist by regular sprinkling with water to prevent desiccation. It is essential that we observe the all footing excavations before reinforcing steel is placed.

We estimate that total foundation movement under static loads will be less than 1-inch, with post-construction differential movement of less than ½-inch between adjacent footings.

8.6 Lateral Loads on Footings

Lateral loads may be resisted by friction between the bottom of footings and the supporting subgrade. A maximum allowable frictional resistance of 0.30 may be used for design. In addition, lateral resistance may be provided by passive pressures acting against footings poured neat against competent soil. We recommend that an allowable passive pressure based on an equivalent fluid pressure of 300 pounds per cubic foot (pcf) be used in design. The upper 12 inches of soil should be neglected when determining lateral passive resistance unless confined by pavements or flatwork.

9.0 PAVEMENTS

9.1 Asphalt Concrete

Based on the near-surface soils encountered during our explorations, which generally consisted of silty sand, we judged an R-value of 15 to be applicable for design based on a subgrade consisting of untreated on-site soils. Using estimated traffic indices for various pavement-loading requirements and untreated on-site soils, we developed the following recommended pavement sections based on Procedure 608 of the Caltrans Highway Design Manual, presented in Table 7.



Table 7. Recommended Asphalt Concrete Pavement Design Alternatives

Pavement Components

Design R-Value = 15

| General Traffic Condition | Design Traffic Index | Asphalt Concrete (Inches) | Aggregate Baserock* (Inches) | Total Thickness (Inches) |
|---------------------------------|----------------------------|---------------------------------|------------------------------------|--------------------------------|
| Automobile | 5.0 | 3.0 | 8.0 | 11.0 |
| Parking Channel | 5.5 | 3.0 | 10.0 | 13.0 |
| Truck Access & | 6.0 | 3.5 | 11.0 | 14.5 |
| Parking Areas | 6.5 | 4.0 | 12.0 | 16.0 |

^{*}Caltrans Class 2 aggregate base; minimum R-value equal to 78.

The traffic indices used in our pavement design are considered reasonable values for the proposed development and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. The traffic parameters used for design were selected based on engineering judgment and not on information furnished to us such as an equivalent wheel load analysis or a traffic study.

Because the full thickness of asphalt concrete is frequently not placed prior to construction traffic being allowed to use the streets (or parking lots), rutting and pavement failures can occur prior to project completion. To reduce this occurrence, we recommend that either the full design pavement section be placed prior to use by construction traffic, or a higher Traffic Index (TI) be specified where construction traffic will use the pavement.

9.2 Exterior Portland Cement Concrete (PCC) Pavements

Recommendations for exterior PCC pavements are presented below in Table 8. Since the expected Average Daily Truck Traffic (ADTT) is not known at this time, we have provided alternatives for minimum pavement thickness. An allowable ADTT should be chosen that is greater than expected for the development.

Table 8. Recommended Minimum PCC Pavement Thickness

| Allowable ADTT | Minimum PCC Pavement Thickness (inches) |
|-------------------|--|
| 5 | 5 |
| 57 | 5 ¹ / ₂ |
| 480 | 6 |

Our design is based on an R-value of 15 and a 28-day unconfined compressive strength for concrete of at least 3,500 pounds per square inch (psi), and a modulus of rupture of at least 550 psi .In addition, our design assumes that pavements are restrained laterally by a concrete shoulder or curb and that all PCC pavements are underlain by at least 6 inches of Class 2 aggregate base. We recommend that adequate construction and control joints be used in design of the PCC pavements to control the cracking inherent in this construction.



9.3 Pavement Cutoff

Surface water infiltration beneath pavements could significantly reduce the pavement design life. While the amount of reduction in pavement life is difficult to quantify, in our opinion, the normal design life of 20 years may be reduced to less than 10 years. Therefore, long-term maintenance greater than normal may be required.

To limit the need for additional long-term maintenance, it would be beneficial to protect at-grade pavements from landscape water infiltration by means of a concrete cut-off wall, deepened curbs, redwood header, "Deep-Root Moisture Barrier," or equivalent. However, if reduced pavement life and greater than normal pavement maintenance are acceptable, the cutoff barrier may be eliminated. If desired to install pavement cutoff barriers, they should be considered where pavement areas lay downslope of any landscape areas that are to be sprinkled or irrigated, and should extend to a depth of at least 4 inches below the base rock layer.

9.4 Asphalt Concrete, Aggregate Base and Subgrade

Asphalt concrete and aggregate base should conform to and be placed in accordance with the requirements of Caltrans Standard Specifications, latest edition, except that ASTM Test Designation D1557 should be used to determine the relative compaction of the aggregate base. Pavement subgrade should be prepared and compacted as described in the "Earthwork" section of this report.

9.5 Flatwork and Sidewalks

We recommend that exterior slabs-on-grade, such as flatwork and sidewalks be at least 4 inches thick and be underlain by at least 6 inches of NEF or Class 2 aggregate base compacted to a minimum of 90 percent relative compaction in accordance with ASTM Test Method D1557, latest edition. If sidewalks are subject to wheel loads, they should be designed in accordance with the "Exterior Portland Cement Concrete Pavements" section of this report.

We recommend that exterior slabs be isolated from adjacent foundations and that adequate construction and control joints be used in design of the concrete slabs to control cracking inherent in concrete construction.

10.0 LIMITATIONS

This report has been prepared for the sole use of King Asset Management, specifically for design of the proposed Office Building and Parking Structures in Daly City, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was written. No other warranty, expressed or implied, is made or should be inferred.

The opinions, conclusions and recommendations contained in this report are based upon the information obtained from our investigation, which includes data from widely separated discrete locations, and visual observations from our site reconnaissance along with our local experience and engineering judgment. The recommendations presented in this report are based on the assumption that soil and geologic conditions at or between explorations do not deviate substantially from those encountered or extrapolated from the information collected during our investigation. We are not responsible for the data presented by others.



We should be retained to review the geotechnical aspects of the final plans and specifications for conformance with our recommendations. The recommendations provided in this report are based on the assumption that we will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, TRC cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of TRC's report by others. Furthermore, TRC will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services and/or at the time another consultant is retained for follow up service to this report.

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of the property will likely occur with the passage of time due to natural processes and/or the works of man. In addition, changes in applicable standards of practice can occur as a result of legislation and/or the broadening of knowledge. Furthermore, geotechnical issues may arise that were not apparent at the time of our investigation. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any other properties.

11.0 REFERENCES

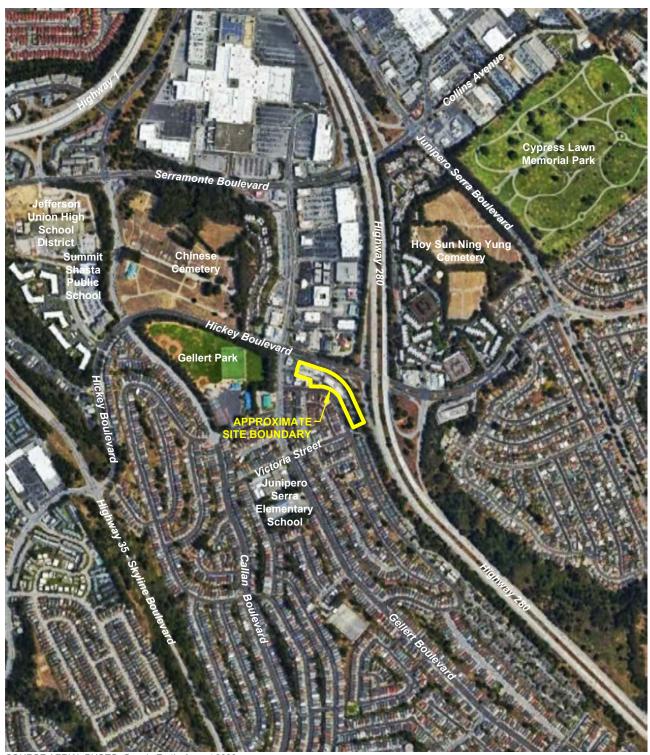
- American Concrete Institute, 2000, *Guide for Measuring, Mixing, Transporting, and Placing Concrete (ACI 304R)*, first printing, January.
- American Concrete Institute, 2008, *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary*, An ACI Standard, first printing, January.
- ASCE (American Society of Civil Engineers), 2016, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, ASCE/SEI Standard 7-16.
- Brown, Dan A., Turner, John P., and Castelli, Raymond J. (2010), Drilled Shafts: Construction Procedures and LRFD Design Methods, NHI Course No. 132014, Geotechnical Engineering Circular No. 10, National Highway Institute, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., Report No. FHWA NHI-10-016, May 2010.
- California Building Code, 2019, Structural Engineering Design Provisions, Vol. 2.
- California Department of Conservation Division of Mines and Geology, 1998, Maps of Known Active Fault Near-Source Zones in California and adjacent Portions of Nevada, International Conference of Building Officials February, 1998.
- California Geological Survey, 2000, State of California Seismic Hazard Report, City and County of San Francisco Quadrangle, Seismic Hazard Zones Report 043.
- Ellis, William J., 1978, Corrosion of Concrete Pipelines, Western States of Corrosion Seminar.
- Ensoft, Inc., 2012, SHAFT v.6.0 for Windows, Computer Software.
- Federal Emergency Management Agency, 2012, Flood Insurance Rate Map, San Mateo County, California, Panel 37 of 510, Map Number 06081C0037E, October 16.



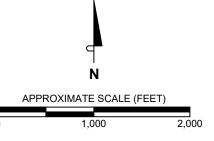
- Seed, H.B. and I.M. Idriss, 1971, A Simplified Procedure for Evaluation soil Liquefaction Potential: JSMFC, ASCE, Vol. 97, No. SM 9, pp. 1249 1274.
- State of California Department of Transportation, 2006, Highway Design Manual, May.
- Tokimatsu, K., and H.B. Seed. (1987). "Evaluation of settlements in sands due to earthquake shaking." J. Geotech. Eng. Div., ASCE, 113(8), 861-78.
- United States Geological Survey, 2014, Geologic Hazards Science Center 2014 USGS NSHM 2014

 Dynamic, https://earthquake.usgs.gov/hazards/interactive/
- U.S. Geological Survey, 2016, *US Seismic Design Maps, Earthquake Hazards Program*, http://earthquake.usgs.gov/designmaps/us/application.php.
- WGCEP [Working Group on California Earthquake Probabilities], 2014, The Uniform California Earthquake Rupture Forecast, Version 2: U.S Geological Survey, Open File Report 2014-2044.
- Youd, T.L., Idriss, I.M., et al (2001), "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol 127, No. 10, October, 2001.





SOURCE AERIAL PHOTO: Google Earth, August 2020.



VICINITY MAP

455 Hickey Boulevard Daly City, California

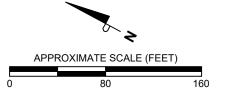


434432

FIGURE 1

POTENTIAL LOCATION FOR CELL TOWER **⊕** EB-2 RAMP TO PARKING BELOW **○** *EB*-2 NEW **⊕** EB-1 -RETAINING WALL/ BERMS ALONG PROPERTY LINE AVAILABLE FOR STREET PARKING -0 0 3.2 Acres 180,000 sf (1.29 FAR) 6-Story over 3 Levels of Podium Parking Required parking (@5/1,000)*: 900 cars 200 cars 705 cars 905 cars 46 cars *Specified parking ratio is a target ratio. The city's parking ratio stands at 1/300 for the first 21,000 sf of the gross building area and then 1/200 for the rest of the gross building area. Therefore, 865 cars are required per zoning ordinance.

Exploratory boring by TRC, 2021



SITE PLAN - SCENARIO 1

455 Hickey Boulevard Daly City, California



434432 **FIGURE 2A**

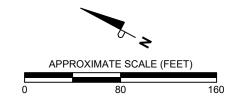
SOURCE: DES Architects Engineers, Preliminary Application Submittal, December 2020.

LEGEND

Approximate locations of:

Historical boring by John V. Lowney & Associates, July 1979

Exploratory boring by TRC, 2021

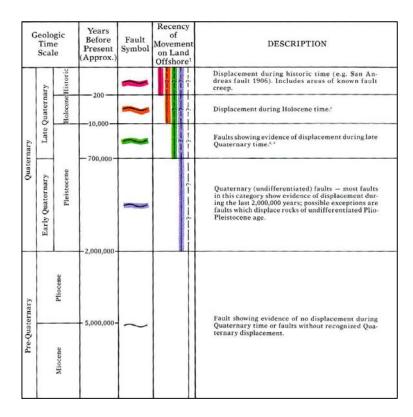


SITE PLAN - SCENARIO 2

455 Hickey Boulevard Daly City, California



434432 **FIGURE 2B**



NOTES:

Base map is a composite of part the San Francisco 1:250,000 scale map (reference code 37 122-A1-TF-250-00, 1980) and the San Jose 1:250,000 scale map (reference code 37 120-A1-TF-250-00, 1969). For cartographic details, refer to these maps. Bathymetric information is not intended for navigational purposes.

Transverse Mercator Projection 10,000-meter Universal Transverse Mercator grid, zone 10.

Minor corrections and additions to culture by California Division of Mines and Geology 1987.

From: Bortugno & others (1991)

Some faults highlighted in purple are not considered active (Holocene Movement) by the State of California.

REGIONAL FAULT MAP

455 Hickey Boulevard Daly City, California



Ultimate Axial Capacity

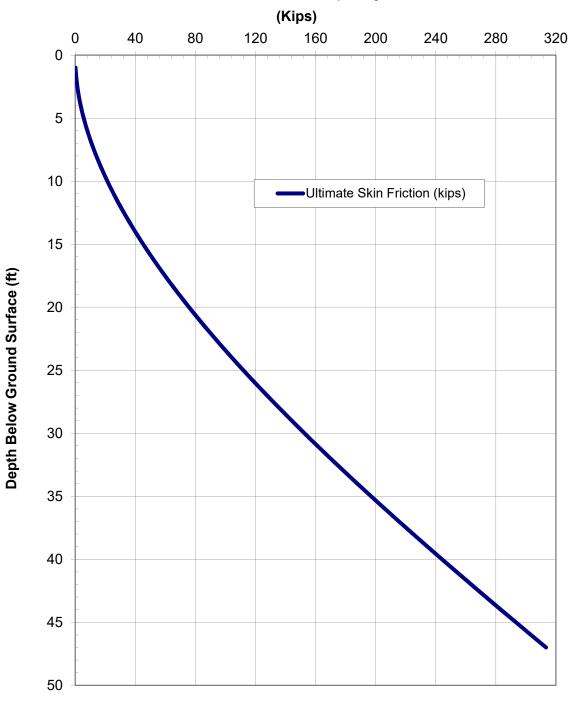




FIGURE 4A Ultimate Axial Capacity Plot, 1-Foot Drilled Pier 434432

| Depth | Utimate Axial |
|-------|-----------------|
| (ft) | Capacity (kips) |
| 1 | 0 |
| 2 | 1 |
| 3 | 2 |
| 4 | 4 |
| 5 | 6 |
| 6 | 8 |
| 7 | 11 |
| 8 | 14 |
| 9 | 18 |
| 10 | 21 |
| 11 | 26 |
| 12 | 30 |
| 13 | 35 |
| 14 | 40 |
| 15 | 45 |
| 16 | 51 |
| 17 | 57 |
| 18 | 63 |
| 19 | 69 |
| 20 | 76 |
| 21 | 83 |
| 22 | 90 |
| 23 | 97 |
| 24 | 104 |
| 25 | 112 |
| 26 | 120 |
| 27 | 128 |
| 28 | 136 |
| 29 | 144 |
| 30 | 153 |
| 31 | 161 |
| 32 | 170 |
| 33 | 179 |
| 34 | 188 |
| 35 | 197 |
| 36 | 206 |
| 37 | 216 |
| 38 | 225 |
| 39 | 234 |
| 40 | 244 |
| 41 | 254 |
| 42 | 264 |
| 43 | 273 |
| 44 | 283 |
| 45 | 293 |
| 46 | 303 |
| 47 | 313 |



APPENDIX A

FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using conventional, truck-mounted, hollow-stem auger drilling equipment. Four 8-inch diameter exploratory borings were drilled on July 30, 2021 and August 6, 2021 to a maximum depth of 50 feet. The approximate locations of the exploratory borings are shown on Figure 2. The soils encountered in the borings were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). The logs of the borings, as well as a key to the classification of the soil, are included as part of this appendix.

The locations of borings were approximately determined by pacing from existing site boundaries. Elevations of the boring were approximately determined based on the topographic site plan provided. The locations of the borings should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. Penetration resistance blow counts were obtained by dropping a 140-pound hammer 30 inches. Modified California 3.0-inch outside diameter (O.D.) samples and Standard Penetration Test (SPT) 2-inch O.D. samples were obtained by driving the samplers 18 inches and recording the number of hammer blows for each 6 inches of penetration. Unless otherwise indicated, the blows per foot recorded on the boring logs represent the accumulated number of blows required to drive the samplers the last two 6-inch increments. When using the SPT sampler, the sum of the last two 6-inch increments is the uncorrected SPT measured blow count. The various samplers are denoted at the appropriate depth on the boring logs and symbolized as shown on Figures A-1 and A-2.

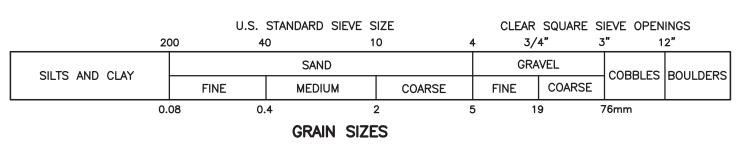
The attached boring logs and related information depict subsurface conditions at the locations indicated and, on the date, designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

* * * * * * * * * * * *



| PRIMARY DIVISIONS | | | SOIL TYPE | | SECONDARY DIVISIONS |
|--|--|-------------------------|--------------|------|--|
| | 0047/210 | CLEAN GRAVELS | GW | | Well graded gravels, gravel—sand mixtures, little or no fines |
| SOILS | GRAVELS MORE THAN HALF | (Less than 5% Fines) | GP | 300 | Poorly graded gravels or gravel—sand mixtures, little or no fines |
| ≤ | OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE | GRAVEL WITH | GM | | Silty gravels, gravel—sand—silt mixtures, plastic fines |
| GRAINED HALF OF M R THAN NO. | | FINES | GC | | Clayey gravels, gravel-sand-clay mixtures, plastic fines |
| I GR | 044100 | THAN HALF 5% Fines) | SW | | Well graded sands, gravelly sands, little or no fines |
| COARSE THA | SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE | | SP | | Poorly graded sands or gravelly sands, little or no fines |
| O S | | PANDS NAHT SANDS | SM | | Silty sands, sand-silt-mixtures, non-plastic fines |
| | | | SC | | Clayey sands, sand-clay mixtures, plastic fines |
| S. Islando | | | ML | | Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity |
| SOILS MATERIAL 10. 200 | SILTS AND CL LIQUID LIMIT IS LESS TH | | CL | | Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays |
| VED FAN N | | | OL | | Organic silts and organic silty clays of low plasticity |
| GRAINED HAN HALF OF ALLER THAN SIEVE SIZE | | | | | Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts |
| FINE GRAINED SOILS MORE THAN HALF OF MATERAL IS SMALLER THAN NO. 200 SIEVE SIZE | SILTS AND CLAY LIQUID LIMIT IS GREATER THAI | | СН | | Inorganic clays of high plasticity, fat clays |
| E Ø | | | ОН | | Organic clays of medium to high plasticity, organic silts |
| HIGHLY ORGANIC SOILS | | | PT | 7 77 | Peat and other highly organic soils |

DEFINITION OF TERMS











SAMPLERS

| SAND AND GRAVEL | BLOWS/FOOT* | |
|-----------------|-------------|--|
| VERY LOOSE | 0-4 | |
| LOOSE | 4-10 | |
| MEDIUM DENSE | 10-30 | |
| DENSE | 30-50 | |
| VERY DENSE | OVER 50 | |

| SILTS AND CLAYS | STRENGTH+ | BLOWS/FOOT* |
|---|---|---|
| VERY SOFT SOFT MEDIUM STIFF STIFF VERY STIFF HARD | 0-1/4 1/4-1/2 1/2-1 1-2 2-4 OVER 4 | 0-2 2-4 4-8 8-16 16-32 OVER 32 |

RELATIVE DENSITY

CONSISTENCY

*Number of blows of 140 pound hammer falling 30 inches to drive a 2-inch 0.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586). +Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

KEY TO EXPLORATORY BORING LOGS Unified Soil Classification System (ASTM D-2487)



WEATHERING

| FRESH | Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer if cystalline. | MODERATELY SEVERE | All rock except quartz, discolored or stained. In granitoid rocks, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick. Rock goes "clunk" when struck. |
|-------------|--|----------------------|---|
| VERY SLIGHT | Rock generally fresh, joints stained, some joints may show thin clay coatings, crystals in broken face show bright. Rock rings under hammer if crystalline. | SEVERE | All rock except quartz discolored or stained. Rock "fabric" clear and evident, but reduced in strength to strong soil. In granitoid rocks, all feldspars kaolinized to some extent. Some fragments of strong rock usually left. |
| SLIGHT | Rock generally fresh, joints stained, sand discoloration extends into rock up to 1 inch. Joints may contain clay. In granitoid rocks some occasional feldspar crystals are dull and discolored. Crystalline rocks ring under hammer. | VERY SEVERE | All rock except quartz discolored and stained. Rock "fabric" discernible, but mass effectively reduced to "soil" with only fragments of strong rock remaining. |
| MODERATE | Significant portions of rock show discoloration and weathering effects. In granitoid rocks, most feldspars are dull and discolored; some are clayey. Rock has dull sound under hammmer and shows significant loss of strength as compared with fresh rock. | COMPLETE | Rock reduced to "soil". Rock "fabric" not discernible or discernible only in small scattered locations. Quartz may be present as dikes or stringers. |

HARDNESS

| VERY HARD | Cannot be scratched with knife or sharp pick. Breaking of hand specimens requires several hard blows of geologist's pick. | MEDIUM | Can be grooved or gouged 1/16 inch deep by firm pressure on knife or pick point. Can be excavated in small chips to pieces abount 1 inch maximum size by hard blows of the point of a geologist's pick. |
|--------------------|---|-----------|--|
| HARD | Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach hand specimen. | SOFT | Can be gouged or grooved readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small thin pieces can be broken by finger pressure. |
| MODERATELY HARD | Can be scratched with knife or pick. Gouges or grooves to 1/4 inch deep can be excavated by hard blow or point of a geologist's pick. Hard specimen can be detached by moderate blow. | VERY SOFT | Can be carved with knife. Can be excavated readily with point of pick. Pieces 1 inch or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail. |

JOINT BEDDING AND FOLIATION SPACING IN ROCK*

ROCK QUALITY DESIGNATOR (RQD)**

| Spacing | Joints | Bedding and Foliation | RQD, as a percentage | Diagnostic description |
|--|--|--|---|--|
| Less than 2 in. 2 in. to 1 ft. 1 ft. to 3 ft. 3 ft. to 10 ft. More than 10 ft. | Very close Close Moderately close Wide Very Wide | Very thin Thin Medium Thick Very thick | Exceeding 90 90-75 75-50 50-25 Less than 25 | Excellent Good Fair Poor Very poor |

*Joint spacing refers to the distance normal to the plane of the joints of a single system or "set" of joints that are parallel to each other or nearly so. The spacing of each "set" should be described, if possible to establish.

***RQD should always be given as a percentage. Diagnostic description is intended primarily for evaluating problems with tunnels or excavation in rock.

RQD = 100 (lengths of core in pieces 4 in. and longer/length of run)(1 in. = 25.4 mm; 1 ft. = 0.305 m)

KEY TO BEDROCK DESCRIPTIONS



EXPLORATORY BORING: EB-1 Sheet 1 of 2 DRILL RIG: TRUCK MOBILE B-56 PROJECT NO: 434432 BORING TYPE: 8-INCH HOLLOW-STEM AUGER PROJECT: 455 HICKEY BOULEVARD LOGGED BY: JA LOCATION: DALY CITY, CA START DATE: 7-30-21 FINISH DATE: 7-30-21 COMPLETION DEPTH: 50.0 FT. This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual. Undrained Shear Strength (ksf) PERCENT PASSING NO. 200 SIEVE PENETRATION RESISTANCE (BLOWS/FT.) DRY DENSITY (PCF) MOISTURE CONTENT (%) O Pocket Penetrometer SOIL LEGEND SOIL TYPE DEPTH (FT) △ Torvane Unconfined Compression MATERIAL DESCRIPTION AND REMARKS ▲ U-U Triaxial Compression SURFACE ELEVATION: 4" of AC over 4" of AB AC/AB SILTY SAND (SM) medium dense, moist, yellowish brown, low plasticity, fine 42 10 107 SM 45 10 109 SANDSTONE [MERCED FORMATION] completely weathered, friable, very soft, yellowish brown 70 13 103 17 26 99 dark olive brown 10-58 13 olive brown 15 27 22 grayish brown 20 32 gray to dark gray 25 22 24 30-Continued Next Page **GROUND WATER OBSERVATIONS:** NO FREE GROUND WATER ENCOUNTERED



| DRING TYPE: 8-INCH HOLLOW-STEM AUGER DGGED BY. JA TART DATE: 7-30-21 FINISH DATE: 7-30-21 FOR the plant of the plant of the board of t | | | RUCK MOBILE B-56 | PROJECT | | | | יוסם | " ⊏\ / | /^ DC | ` | | | | |
|--|-----------------------|-------------|---|---|------------------|--|--------------|-------------------------|-------------------|----------------------------------|----|------------------|--|------------------|-----|
| TART DATE: 7-30-21 FINISH DATE: 7-30-21 COMPLETION DEPTH: 50.0 FT. The tag as a part of a report by TRC, and souther not busined as a control of the contro | | | | | | | | | LE v | АКЬ | | | | | |
| The last is sent of the report by TRC and shaded set have for the expansion of the standard set have for the expansion of the standard set have for the section of the expansion of the standard set have for the expansion of the standard set have for the section of the standard set have for the section of the standard set have for the expansion of the standard set have for the section of the standard set have for the section of the standard set have for the expansion of the standard set have for the section of the standard set have for the section of the standard set have for the section of the sectio | | | | | | | | | | | | | | | |
| SANDSTONE [MERCED FORMATION] SANDSTONE [MERCED FORMATION] SUbstitute of the state of the s | ART DA | <u>.TE:</u> | | - | יט TION! | EPTH: | <u>. 5</u> (| <u>J.0 ⊢</u> | T. | T | T | | | | _ |
| SANDSTONE [MERCED FORMATION] completely weathered, friable, very soft, yellowish brown brownish gray yellowish brown, soft to very soft yellowish brown, soft to very soft light brown Bottom of boring at 50 feet 29 21 | (FT) DEPTH (FT) | SOIL LEGEND | stand-alone document. This description applies only to the location o at the time of drilling, Subsurface conditions may differ at other loc change at this location with time. The description presented is a si actual conditions encountered. Transitions between soil types ma | of the exploration ations and may simplification of ay be gradual. | SOIL TYPE | PENETRATION RESISTANCE (BLOWS/FT.) | SAMPLER | MOISTURE CONTENT (%) | DRY DENSITY (PCF) | PERCENT PASSING NO. 200 SIEVE | | ocket Periorvane | (ksf) Penetron ned Com | meter mpressi | sio |
| brownish gray 32 yellowish brown, soft to very soft 40 light brown 50 Bottom of boring at 50 feet | 30- | 1 | CANDOTONE IMEDOED EODMATIONI | | | | <u> </u> - | <u></u> | | <u> </u> | 1. | .0 2. | .0 3 | .0 4 | 4.0 |
| yellowish brown, soft to very soft 32 | - | | completely weathered, friable, very soft, yello | wish brown | _ - - - | 32 | | 7 | | | | | | | |
| 40 yellowish brown, soft to very soft 45 | - | | | | | 32 | | 7 23 | | | | | | | |
| light brown 50 Bottom of boring at 50 feet 55- | 40- | | yellowish brown, soπ το very son | - | | | | 7 | | | | | | | - |
| 50 Bottom of boring at 50 feet | 45- | | | - | | 34 | | | | | | | | | _ |
| | 50- | | | | | 29 | | 21 | | | | | | | _ |
| | 55- | - - - | | | - - - | | | | | | | | | | _ |
| en_ | - | - - - | | | - - - | | | | | | | | | | |
| | 60- |] | | - | _ | | ' | ' | ' | | | | <u> </u> | 1 | _ |



EXPLORATORY BORING: EB-2 Sheet 1 of 1 DRILL RIG: TRUCK MOBILE B-56 PROJECT NO: 434432 BORING TYPE: 8-INCH HOLLOW-STEM AUGER PROJECT: 455 HICKEY BOULEVARD LOGGED BY: JA LOCATION: DALY CITY, CA START DATE: 8-6-21 FINISH DATE: 8-6-21 COMPLETION DEPTH: 30.0 FT. This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual. Undrained Shear Strength (ksf) PERCENT PASSING NO. 200 SIEVE PENETRATION RESISTANCE (BLOWS/FT.) DRY DENSITY (PCF) MOISTURE CONTENT (%) O Pocket Penetrometer SOIL LEGEND SOIL TYPE DEPTH (FT) △ Torvane Unconfined Compression MATERIAL DESCRIPTION AND REMARKS U-U Triaxial Compression SURFACE ELEVATION: 3" of AC over 5" of AB AC/AB SILTY SAND (SM) 26 SM very dense, moist, yellowish brown, low plasticity, fine 50/6 7 104 sand, trace bedrock fragments SANDSTONE [MERCED FORMATION] 28 completely weathered, friable, very soft, yellowish brown 50/6 103 73 8 96 24 10 10-25 15 25 42 light olive brown 20-22 25 46 11 Bottom of boring at 30 feet 30-**GROUND WATER OBSERVATIONS:** NO FREE GROUND WATER ENCOUNTERED



EXPLORATORY BORING: EB-3 Sheet 1 of 2 DRILL RIG: TRUCK MOBILE B-56 PROJECT NO: 434432 BORING TYPE: 8-INCH HOLLOW-STEM AUGER PROJECT: 455 HICKEY BOULEVARD LOGGED BY: JA LOCATION: DALY CITY, CA START DATE: 8-6-21 FINISH DATE: 8-6-21 COMPLETION DEPTH: 50.0 FT. This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual. Undrained Shear Strength (ksf) PERCENT PASSING NO. 200 SIEVE PENETRATION RESISTANCE (BLOWS/FT.) DRY DENSITY (PCF) MOISTURE CONTENT (%) O Pocket Penetrometer SOIL LEGEND SOIL TYPE DEPTH (FT) △ Torvane Unconfined Compression MATERIAL DESCRIPTION AND REMARKS ▲ U-U Triaxial Compression SURFACE ELEVATION: 2" of AC over 5" of AB AC/AB SILTY SAND (SM) medium dense, moist, olive brown, low plasticity, fine 35 111 14 sand SM Liquid Limit = 27, Plasticity Index = 5 57 18 dense, brown SANDSTONE [MERCED FORMATION] completely weathered, friable, very soft, reddish brown 59 21 95 37 100 19 10-19 yellowish brown 15 17 19 light olive brown 20 48 25 29 11 30-Continued Next Page **GROUND WATER OBSERVATIONS:** NO FREE GROUND WATER ENCOUNTERED



| | | | EXPLORATORY BO | RING | : E | B-3 | | Co | nt' | d | 5 | Sheet | 2 of | 2 | |
|-------------------|---------------|-------------|--|--|-----------|--|---------|-------------------------|----------------------|----------------------------------|------|--|-----------------------|-----------------|---|
| DRILL | RIG: | TRI | UCK MOBILE B-56 | PROJECT | NO: | 434432 | 2 | | | | | | | | |
| BORIN | IG TY | Έ: | 8-INCH HOLLOW-STEM AUGER | PROJECT | : 455 | HICKE | ΥE | 30U | LEV | ARD | | | | | |
| LOGG | ED B | Y: J | A | LOCATION | N: DA | LY CIT | Υ, (| CA | | | | | | | |
| STAR | T DA | ΓE: { | 8-6-21 FINISH DATE: 8-6-21 | COMPLET | TION D | EPTH: | 50 |).0 F | T | | | | | | |
| ELEVATION (FT) | DEPTH (FT) | SOIL LEGEND | This log is a part of a report by TRC, and should not be used as stand-alone document. This description applies only to the location of the at the time of drilling. Subsurface conditions may differ at other location change at this location with time. The description presented is a simplificactual conditions encountered. Transitions between soil types may be a MATERIAL DESCRIPTION AND REMARMATERIAL DESCRIPTION AND REMARMATE | exploration s and may ication of gradual. | SOIL TYPE | PENETRATION RESISTANCE (BLOWS/FT.) | SAMPLER | MOISTURE CONTENT (%) | DRY DENSITY (PCF) | PERCENT PASSING NO. 200 SIEVE | ○ Po | ocket Per orvane nconfined -U Triaxia | (ksf) netrome d Compl | eter ression | n |
| | 30— | | SANDSTONE [MERCED FORMATION] completely weathered, friable, very soft, reddish light brown Bottom of boring at 50 feet | brown - | | 45 38 75 | | 20 | | | 1. | 0 2.0 | 3.0 | 4.0 | |
| | - - - | | | - - - - | | | | | | | | | | | |
| | 60- | | | | | | | | | | | | | | : |
| GR | | | TER OBSERVATIONS: GROUND WATER ENCOUNTERED | | | | | | | | | | | | |



EXPLORATORY BORING: EB-4 Sheet 1 of 1 DRILL RIG: TRUCK MOBILE B-56 PROJECT NO: 434432 BORING TYPE: 8-INCH HOLLOW-STEM AUGER PROJECT: 455 HICKEY BOULEVARD LOGGED BY: JA LOCATION: DALY CITY, CA START DATE: 7-30-21 FINISH DATE: 7-30-21 COMPLETION DEPTH: 30.0 FT. This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual. Undrained Shear Strength (ksf) PERCENT PASSING NO. 200 SIEVE PENETRATION RESISTANCE (BLOWS/FT.) DRY DENSITY (PCF) MOISTURE CONTENT (%) O Pocket Penetrometer SOIL LEGEND SOIL TYPE DEPTH (FT) △ Torvane Unconfined Compression MATERIAL DESCRIPTION AND REMARKS ▲ U-U Triaxial Compression SURFACE ELEVATION: 3.0 2" of AC over 6" of AB AC/AB SILTY SAND (SM) dense, moist, dark olive gray, low plasticity, fine to 58 104 19 SM medium sand Liquid Limit = 35, Plasticity Index = 11 46 16 98 SANDSTONE [MERCED FORMATION] completely weathered, friable, soft to very soft, light brown 72 14 107 17 106 44 10-18 15 51 10 20-22 25 29 20 Bottom of boring at 30 feet 30-**GROUND WATER OBSERVATIONS:** NO FREE GROUND WATER ENCOUNTERED



APPENDIX B

LABORATORY PROGRAM

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site and to aid in verifying soil classification.

Moisture Content: The natural water content was measured (ASTM D2216) on samples of the materials recovered from the boring. These water contents are recorded on the boring log at the appropriate sample depths.

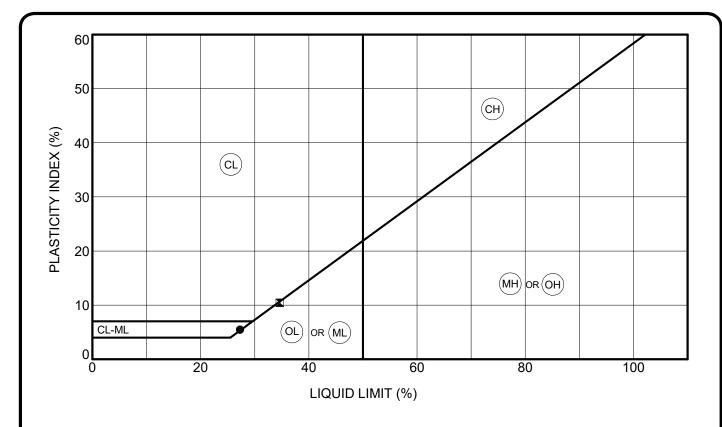
Dry Densities: In place dry density tests (ASTM D2937) were performed on samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring log at the appropriate sample depths.

Plasticity Index: Plasticity Index (PI) test determinations (ASTM D₄₃18) were performed on samples of the subsurface soil to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of these tests are presented on the Plasticity Chart of this appendix and on the logs of the boring at the appropriate sample depths.

Soil Corrosion: Soil pH, electrical resistivity, sulfate and chloride concentrations were performed on samples of the subsurface soils to aid in their classification. Results of these tests are included in this appendix.

* * * * * * * * * * *





| Symbol | Boring No. | Depth (ft.) | Natural Water Content (%) | Liquid Limit (%) | Plastic Limit (%) | Plasticity Index (%) | Passing No. 200 Sieve | Unified Soil Classification Description |
|--------|------------|----------------|------------------------------------|------------------------|-------------------------|----------------------------|-----------------------------|---|
| • | EB-3 | 2.0 | | 27 | 22 | 5 | | |
| | EB-4 | 2.0 | | 35 | 24 | 11 | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |



PLASTICITY CHART AND DATA

Project: 455 HICKEY BOULEVARD

Location: DALY CITY, CA

Project No.: 434432 FIGURE B-1



Corrosivity Tests Summary

| CTL# | 028-2988 | Date: | 8/10/2021 | Tested By: | PJ | Checked: | PJ |
|----------|----------|----------|-----------|-----------------|----|-----------|--------|
| Client: | TRC | Project: | | 455 Hickey Blvd | | Proj. No: | 434432 |
| Remarks: | | | | - | | _ | |

| | Location o | | | rity @ 15.5 °C (0 | | Chloride | | fate | pН | OR | | Sulfide | Moisture | |
|------------|------------|------------|----------|-------------------|----------|------------|------------|------------|----------|---------------------|---------|---------------|------------|----------------------------|
| | | | As Rec. | Min | Sat. | mg/kg | mg/kg | % | • | (Rede | | Qualitative | At Test | 0 1111 15 14 |
| | | | | | | Dry Wt. | Dry Wt. | Dry Wt. | | E _H (mv) | At Test | by Lead | % | Soil Visual Description |
| Boring San | mple, No. | Depth, ft. | ASTM G57 | Cal 643 | ASTM G57 | ASTM D4327 | ASTM D4327 | ASTM D4327 | ASTM G51 | ASTM G200 | Temp °C | Acetate Paper | ASTM D2216 | |
| EB-1 | 2A | 3.5 | - | - | 3,236 | 21 | 143 | 0.0143 | 6.7 | - | | - | 17.8 | Yellowish Brown Silty SAND |
| EB-1 | 3A | 5.5 | - | - | 5,476 | 13 | 120 | 0.0120 | 6.7 | - | - | - | 17.7 | Yellowish Brown Silty SAND |
| EB-4 | 1B | 2.0 | - | - | 1,507 | 17 | 1,520 | 0.1520 | 4.3 | - | - | - | 18.7 | Yellowish Brown Silty SAND |
| EB-4 | 2B | 4.0 | - | - | 1,093 | 14 | 5,577 | 0.5577 | 4.6 | - | ı | 1 | 18.1 | Yellowish Brown Silty SAND |
| | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | |

APPENDIX C PREVIOUS INVESTIGATION BY LOWNEY ASSOCIATES



GEOTECHNICAL INVESTIGATION
SERRAMONTE OFFICE BUILDING COMPLEX
DALY CITY, CALIFORNIA

July 18, 1979 277-22, PA 8891

Citizens Savings and Loan Association % Hagman Associates 114 Santa Margarita Menlo Park, California 94025

Attention: Mr. Robert W. Hagman, A.I.A.

RE: GEOTECHNICAL INVESTIGATION SERRAMONTE OFFICE BUILDING COMPLEX DALY CITY, CALIFORNIA

Gentlemen:

In accordance with your request, we have performed a geotechnical investigation for the above project. The accompanying report presents the results of our field investigation work, laboratory tests, and engineering analyses. The soil and foundation conditions are discussed and recommendations for the soil and foundation engineering aspects of the project are presented.

We refer you to the text of the report for detailed recommendations. If you have any questions concerning our findings, please call.

Very truly yours,

JOHN V. LOWNEY & ASSOCIATES

John V. Lowney

JVL:MG:maf

Copies: Addressee (5)

| CECTECUNICAL | INVESTIGATION |
|--------------|---------------|

For

SERRAMONTE OFFICE BUILDING COMPLEX DALY CITY, CALIFORNIA

То

Citizens Savings and Loan Association c/o Hagman Associates 114 Santa Margarita Menlo Park, California

July 1979

TABLE OF CONTENTS

| | · | Page No. |
|-----------|--|--------------------------------------|
| Letter of | Transmittal | |
| TITLE PA | GE | |
| TABLE OF | F CONTENTS | |
| INTRODU | CTION | 1 |
| SCOPE | | 1 |
| SITE INVE | STIGATION | 2 |
| D. | Groundwater Seismicity | 2 2 3 3 |
| CONCLUS | SIONS AND RECOMMENDATIONS EARTHWORK | 4 |
| | Clearing and Site Preparation Subgrade Preparation Material for Fill Compaction Trench Backfill Temporary Slopes Permanent Slopes Surface Drainage Subsurface Drainage Construction Observation Excavation in Rock | 4 5 6 6 7 7 8 9 |
| В. | FOUNDATIONS 1. Friction Piers 2. Footings 3. Vault 4. Slabs-on-Grade 5. Retaining Walls | 9 11 12 12 |
| FIGURE 1 | - SITE PLAN | |
| APPENDIX | A - FIELD INVESTIGATION | |
| | re A-1, Key to Exploratory Boring Logs oratory Boring Logs | |
| APPENDIX | K B - LABORATORY INVESTIGATION | |
| Figu | re B-1, Plasticity Chart and Data | |

APPENDIX C - GUIDE SPECIFICATIONS - SITE EARTHWORK

Office Building Complex in accordance with generally accepted geotechnical engineering practices. No warranty is expressed or implied. In the event that any changes in the type or location of the buildings are made, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing.

The investigation was conducted under the direction and review of John V. Lowney, Civil Engineer. Supervision of the subsurface exploration, laboratory testing and the engineering analyses were performed by Mark Grotkopf, Staff Engineer.

SITE INVESTIGATION

Our subsurface exploration and site reconnaissance began on June 18, 1979, and was completed on June 20, 1979. A truck-mounted continuous flight auger drill rig was used to investigate and sample the subsurface soils. A total of 15 exploratory borings were drilled to depths ranging from 15.9 to 50.8 feet. The location of all field work is shown on the Site Plan, Figure 1. Logs of borings and additional details regarding the field investigation are included in Appendix A. The laboratory test results are presented in Appendix B; certain test data are shown on the individual boring logs at the appropriate depths.

A. Surface

Elevations at the site vary from +340 feet along Hickey Boulevard to +375 feet adjacent to Serravista Avenue, with the maximum slope being approximately 2:1 (horizontal to vertical). The site is presently vacant with the exception of the western one-third of the property, which is occupied by a paved, fenced-in yard presently used to store lumber for an adjacent construction project.

B. Subsurface

The site is blanketed by a layer of fill material consisting primarily of medium dense sand-silt mixture which varied in gradation across the site.

The thickness of this fill averages approximately 16 feet, with a low of 10 feet and a high of 20 feet. This fill overlays dense to very dense weathered sandstone of the Merced Formation, which appears to be dipping slightly to the east. Boring EB7 encountered unweathered sandstone at a depth of 50 feet.

C. Groundwater

Free groundwater was not encountered in the borings at the time of drilling. Please be cautioned however, that fluctuations in the level ofthe groundwater may occur due to variations in rainfall and other factors at the time measurements were made. It should be noted that localized springs are relatively common in the area and may be encountered during construction, necessitating the installation of subdrains if water is encountered.

D. Seismicity

The San Francisco Bay Area is recognized by geologists and seismologists as one of the most active regions in the United States. The significant earthquakes which occur in the Bay Area are generally associated with crustal movement along well-defined, active fault zones. These zones include the San Andreas, Hayward and Calaveras Faults, located approximately 1.2 miles southwest and 18 and 28 miles northeast of the site, respectively.

Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when and where an earthquake will occur. Nevertheless, on the basis of current technology, it is reasonable to assume that the proposed development will be subjected to at least one moderate to severe earthquake during the 50-year period following construction. During such an earthquake, the danger from fault offset on the site is slight, but strong shaking of the site is likely to occur.

CONCLUSIONS AND RECOMMENDATIONS

From a soil and foundation engineering standpoint, it is our opinion that the site is suitable for the proposed development provided the recommendations presented in this report are incorporated in the design and construction of the project. The most significant geotechnical condition encountered on the site consists of the medium dense silty sand fill placed on the site. Since the fill was not placed to support substantial loads as would be imposed by multi-story structures, it is our opinion that shallow foundations should not be used in the fill under the multi-story buildings. To spread the load over a depth interval in the fill to minimize the risk of a possible loose pocket that may easily occur in mass earthwork operations placed for routine development, we recommend the use of drilled friction piers in the fill under the multi-story buildings and shallow spread footings elsewhere. These recommendations should be reviewed by us when the final building locations and loads are known.

Since subsurface conditions may vary considerably from those expected on the basis of relatively small diameter borings, the recommendations presented in this report are based upon John V. Lowney and Associates being retained to 1) review the final construction plans and specifications, and 2) observe the earthwork and foundation installation.

A. EARTHWORK

1. Clearing and Site Preparation

The site should be cleared of all surface and subsurface deleterious materials including any buried utility lines, pavements, debris, designated trees and shrubs and associated roots. The resulting excavations that extend below the planned finish site grades should be cleaned and backfilled with suitable material compacted to the requirements given below under Item A. 4, "Compaction".

An area to receive special attention is where the previously existing tanks were removed and replaced with compacted soil. Field density tests taken by others shortly after the completion of the backfill operations indicate that the relative compaction of the upper one foot of backfill ranged from 78% to 89%. To our knowledge no tests were taken below one foot deep. It is our opinion that the poor compaction of this material will bring about settlements that would cause substantial damage to the overlying pavements. We therefore recommend that this material be excavated to a depth of at least 5 feet below subgrade elevation, and be recompacted in accordance with the recommendations for structural fill provided below under Item A. 4, "Compaction."

After clearing, the site should be stripped to sufficient depth to remove all surface vegetation and organic laden topsoil. At the time of our field investigation, we estimated that a stripping depth of approximately 4 inches would be required. The actual stripping depth required will depend on site usage prior to construction and should, therefore, be determined in the field by us at the time of construction. The stripped materials should be removed from the site or may be stockpiled for use in landscaped areas, if desired.

2. Subgrade Preparation

After the site has been properly cleared, stripped, and the necessary excavations made, the exposed surface soils in those areas to receive structural fill, slabs-on-grade or pavements should be scarified to a depth of 6 inches, moisture conditioned to slightly above optimum moisture content, and compacted in accordance with the requirements for structural fill given below under Item A. 4, "Compaction."

3. Material for Fill

All on-site soils having an organic content of less than 3 percent by volume are suitable for use as fill at the site. In general, fill material should not contain rocks or lumps larger than 6 inches in greatest dimension with no more than 15 percent larger than 2.5 inches. Imported fill material should be predominantly granular with a sand equivalent of 15 or more.

4. Compaction

All structural fill placed at the site and scarified surface soils in those areas to receive structural fill or slabs-on-grade should be compacted by mechanical means to at least 90 percent relative compaction as determined by ASTM Test Designation D 1557, latest edition. Fill should be placed in lifts not exceeding 8 inches in uncompacted thickness. The upper 6 inches of subgrade in pavement areas should be compacted to at least 95 percent relative compaction (ASTM D 1557, latest edition).

5. Trench Backfill

Pipeline trenches should be backfilled with compacted structural fill. If on-site soil is used, the material should be placed in lifts not exceeding 8 inches in uncompacted thickness and compacted to at least 85 percent relative compaction (ASTM D 1557, latest edition) by mechanical means only. Imported sand may also be used for backfilling trenches provided the sand is compacted to at least 90 percent relative compaction. In all pavement and building pad areas, the upper 3 feet of trench backfill should be compacted to at least 90 percent

relative compaction for on-site soils, and to at least 95 percent where imported sand backfill is used. In addition, the upper 6 inches of all trench backfill in pavement areas should be compacted to at least 95 percent relative compaction.

6. Temporary Slopes

Temporary slopes will be necessary during the excavation for the office buildings and the parking structure. We recommend no steeper than 1:1 (horizontal to vertical). Additionally, we recommend that the tops of these slopes be located at least 15 feet away from adjacent buildings. Because of the variable nature of the underlying soils, field modifications of temporary cut slopes will be required during construction if adverse conditions are exposed. Construction equipment and material stockpiles should be located more than 15 feet behind temporary construction slopes to avoid overstressing the temporary slope.

7. Permanent Slopes

We recommend that any fill slopes have a maximum inclination of 2:1 (horizontal to vertical), and that cut slopes have a maximum inclination of 2:1. Exposed slopes may be subject to minor sloughing and erosion which may require periodic maintenance. We recommend that the slopes be planted to minimize erosion.

8. Surface Drainage

Positive surface gradients should be provided adjacent to the buildings to direct surface water away from the foundations and slabs toward suitable discharge facilities. Ponding of surface water should not be allowed adjacent to the structures or on the pavements.

9. Subsurface Drainage

A subdrain system may be required if springs or other subsurface water is encountered during construction. Neighboring projects have encountered such water and the possibility, therefore, becomes more real.

The subdrain system should consist of an 8-inch thick blanket of drain rock and a system of transverse subdrains extending under all building slabs. The transverse drains should be spaced more or less equally across the building. The spacing between the drains should not exceed 25 feet. The subdrains should consist of perforated pipe with a minimum diameter of 4 inches; the pipe invert should be located at a minimum depth of 18 inches below the finished basement floor. Water collected in the drainage system should be directed toward a free outlet or into a sump and pumped out.

10. Construction Observation

All grading and earthwork should be performed under the observation of our representative to see that proper site preparation, selection of satisfactory fill materials, as well as placement and compaction of the fills has been performed. Sufficient notification to us prior to earthwork operations is essential. All earthwork should be performed in accordance with the Guide Specifications - Site Earthwork presented in Appendix C. However, the guide specifications are only general in nature and the actual project specifications should incorporate all requirements contained in the text of this report.

Variations in soil conditions are possible and may be encountered during construction. In order to permit correlation

between the soil data obtained during our field and laboratory investigations and the actual subsurface conditions encountered during construction and to observe conformance with the plans and specifications as originally contemplated, it is essential that we be retained to perform the required continuous or intermittent review during construction of the earthwork, excavation, and foundation phases.

11. Excavation in Rock

Based on the information provided by the exploratory borings and our detailed on-site reconnaissance, it is our opinion that the planned excavation can probably be made with heavy ripping equipment. Even though we do not anticipate that blasting will be required, we recommend that blasting on a unit price basis be included as a bid item in the earthwork contract. It should be noted that the dividing line between ripping and blasting generally depends upon the effort the contractor puts forth with his ripping equipment and the amount of equipment wear which he is willing to accept.

B. FOUNDATIONS

1. Friction Piers

We recommend that the proposed structures, with the exception of the branch office building, be supported on drilled, cast-in-place, straight shaft friction piers. The piers should have a minimum diameter of at least 30 inches and extend to a depth of at least 5 feet into bedrock. For design purposes, the top of bedrock may be assumed to be at Elevation +345 for the two-story office building, at Elevation +325 for the parking structures. Piers may be designed for an

allowable skin friction value of 600 pounds per square foot for combined dead plus live loads above the top-of-bedrock elevation, and for 1200 pounds per square foot below the top-of-bedrock elevation, with a one-third increase allowed for either transient wind or seismic loading. For design purposes, the frictional resistance of the upper 18 inches of soil should be neglected. Piers should have a minimum center-to-center spacing of at least three pier diameters.

The bottoms of pier excavations should be dry, reasonably clean, and free of loose soil prior to installing reinforcing steel and placing concrete. We recommend that the excavation of all piers be performed under our direct observation to determine that the piers are founded in suitable materials and constructed in accordance with the recommendations presented in this report.

Resistance to uplift loads will be developed in friction along the pier shafts. We recommend that an allowable uplift frictional resistance of 350 pounds per square foot be used on the portion of the pier located above the top-of-bedrock elevation, and 800 pounds per square foot be used below the top-of-bedrock elevation.

Due to the variable nature of the upper sandy silts, casing of each shaft may be necessary.

Lateral loads may be resisted by a passive resistance acting against the projected area of the individual pier shaft. Above the top-of-bedrock elevation this resistance should be determined by an equivalent fluid pressure of 500 pounds per cubic foot, with a maximum value of 2000 pounds per square foot. The passive resistance of the upper 18 inches of soil should be

neglected. A uniform passive pressure of 3000 pounds per square foot should be used below the top-of-bedrock elevation.

2. Footings

We recommend that the proposed one-story branch office building be supported on conventional continuous and/or isolated spread footings bearing on either undisturbed natural soils or compacted fills. All footings should be founded at least 14 inches below rough building pad grade or 18 inches below outside finished grade, whichever is deeper. Any building foundations adjacent to utility trenches should have their bottom depths located below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the edge of the trench. Located at these depths, the footings may be designed for an allowable bearing pressure of 2000 pounds per square foot due to dead loads, and 2500 pounds per square foot due to dead plus live loads with a one-third increase for all loads including wind or seismic. All footings should have a minimum width of 12 inches.

All continuous perimeter footings should be reinforced with top and bottom reinforcement to provide structural continuity and to permit spanning of local irregularities. All visible cracks in the bottoms of footing excavations should be closed by soaking prior to placement of concrete. It is essential that we inspect the footing excavations prior to placing reinforcing steel.

Settlements under static building loads are expected to be within tolerable limits for the proposed structural frame. We estimate that post-construction differential settlements across the branch office building should be less than 3/8 inch over the 30-year period following construction.

3. Vault

We recommend that the vault be supported on a mat foundation bearing on compacted structural fill materials or natural soil. The bottom of the mat foundation should extend a minimum of 12 inches below the adjacent finished floor slab level. Founded at this depth, the mat may be designed for an allowable bearing pressure of 1500 pounds per square foot for dead plus live loads with a one-third increase for all loads including wind and/or seismic.

We anticipate the vault loading will be substantially greater than the adjoining building floor loads; hence some differential movement should be anticipated between the vault and the adjacent floor slabs. In order to minimize structural distress we recommend that the vault be separated from the floor slabs. However, if it is necessary that the vault be tied into the floor slabs we recommend that the floor, walls, and roof of the vault be constructed prior to construction of the adjacent floor slabs. This will tend to minimize the differential movement between the two floor systems.

4. Slabs-on-Grade

The proposed slab-on-grade floors may be supported directly on the 8-inch-thick blanket of drain rock described previously under Item A. 9, "Subsurface Drainage." Prior to final construction of the slab, the drain rock surface should be proof-rolled to provide a smooth, firm surface for slab support.

5. Retaining Walls

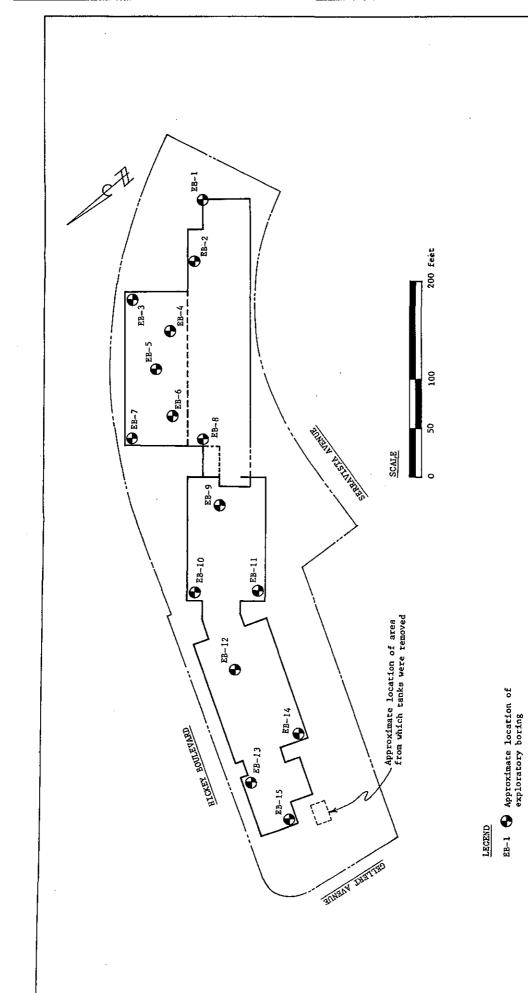
Any proposed retaining walls should be designed to resist lateral earth pressure from adjoining natural materials and/or

backfill as well as any surcharge loads. We recommend that walls that are restrained from movement at the top be designed to resist an equivalent fluid pressure of 35 pounds per cubic foot plus a uniform pressure of 5H pounds per square foot where H is equal to the height of the backfill behind the wall in feet. Such walls should also be designed to resist an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface. We recommend that walls that are free to deflect at the top be designed to resist an equivalent fluid pressure of 35 pounds per cubic foot and uniform pressure equal to one-third of any surcharge loads applied at the surface. The preceding pressures assume sufficient drainage behind the walls to prevent the build-up of hydrostatic pressures from surface water infiltration and/or a rise in the groundwater level.

Adequate drainage may be provided by a subdrain system positioned behind the walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall and below the adjacent slab elevation. The pipe should be embedded in 12 inches of Class 2 Permeable Material (California Division of Highways Standard Specifications, latest revision); the remaining backfill behind the wall should consist of granular material extending to within 2 feet of the level of the outside finish grade. The upper 2 feet should consist of compacted on-site soil. The subdrain outlet should be connected to a free-draining outlet or sump. Walls should be damp-proofed.

Retaining walls should be supported on drilled, cast-in-place friction pier foundations designed in accordance with the recommendations presented previously under Item B. 1, "Friction Piers." Lateral load resistance for the walls can also be developed in accordance with the recommendations presented under Item B. 1, "Friction Piers."

* * *



Base by Hagman Associates, not dated

| John V. Lowney & Associates | Foundation / Soil / Geological Engineers | |
|--|--|-----------|
| CITIZENS | PROJECT NO. | 277-22 |
| CITIZENS SAVINGS SERRAMONTE OFFICE Daly City, California | DATE | July 1979 |
| IE OFFICE | 1 | T ainfile |

SITE PLAN

APPENDIX A - FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and subsurface exploration program using a truck-mounted, continuous flight auger. Fifteen 6-inch diameter exploratory borings were drilled on June 18, 19 and 20, 1979, to a maximum depth of 50.8 feet. The locations of the exploratory borings are shown on the Site Plan, Figure 1. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D-2487). The logs of the borings as well as a key to the classification of the soil are included as part of this appendix.

The locations of borings were approximately determined by tape measurement based on the preliminary site plan. Elevations of borings are determined by interpolation from plan contours. The location and elevation of the borings should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths appropriate to the subsurface exploration. All samples were returned to our laboratory for evaluation and appropriate testing. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O. D. split spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration. The blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches.

The attached boring logs and related information depict subsurface conditions only at the locations indicated and at the particular date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring locations. The passage of time may result in a change in the subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

| PR | IMARÝ DIVISION | Š 🦠 🔭 | GROUP SYMBOL | SECONDARY DIVISIONS |
|---|-----------------------------|--------------------------|-----------------|---|
| | GRAVELS | CLEAN: | GW | Well graded gravels gravel≟sandsrowtures in the or, no fines: |
| JILS TERIZ 200 | MORE THAN HAUF OF COARSE | (LESS THAN 5 % FINES) | ĞР | Poorly graded gravels-or gravel-sand mixtures hiftle or ho fines |
| ED SOJES OF WATER! I.NO 200 | FRACTION IS | GRAVEL WITH * | GM | Silty gravels; gravel-sand-silt mixtures non-plastic fines |
| | ENO 4 SIEVE | FINES. | GC | Clayey gravels, gravel-sand-clay mixtures, plastic tines 1 |
| | SANDS | CLEAN SANDS | SW | Well graded sands gravelly sands (little of no lines |
| COARSE RE MAN IS LARCE | MORE THAN HALF | CLESS, THAN 5% FINES) | SP | Poorly graded sands or gravelly sands little of no tines to |
| CO ORE US | FRACTION IS SMALLER THAN | SANDS | SM | Silty sands, sand-silt mixtures, non=plastic-lines |
| 2 | NO: 4. SIEVE | FINES | ၭင | Clayey sands, sand-clay mixtures, plastic tings |
| SZ RZ | SILTS AND | CLAYS * | ML | Inorganic slits and very fine sands, rock flour slits or clayer fine sands or clayer slits with slight adasticity. |
| D SOILS HALF OF SMALLER SIEVE/SIZ | LIQUID LIM | it is | ÇL | [horganic clays of low to medium plasticity gravelly]? clays sandy clays silty clays lean clays at the same clay at the same clays at the |
| NED N HA IS SN O'SII | LESS THAN | 1 50% | OL | Organic silts and organic silty clays of low plasticity. |
| 3 £ 2 0 0 0 | SILTS AND | CLAYS- | MH | norganic silts micaceous or diatomaceous fine sandy ora- |
| (1) 10 mm (1) 1 | Mú giuouz | ĴŢ, ĴŜΊ | Сн | Inorganic clays of high plasticity fat clays |
| FINE MO MAT THAN | GREATER TH | N 50% | OH - | Organic clays of medium to high plasticity organic sits: |
| · "会验会的 | GHLY ORGANIC SOIL | S | . Pt | Peat and other highly organic soils are the common of the highly organic soils. |

DEFINITION OF TERMS

| | | | | 00.1 | U.S. 9 | TANDAR | SERIES | SIEVE | | · C | LEAR | QUARE | SIEVE 0 | PENINGS | |
|------------|--------|-------|-----|---------------------|----------------|------------------------------|-----------------|----------------------|----------------------|------------------------|-------|-------------------------------|---------------------------------------|----------------------------|-------|
| | 193 15 | | 1 | 00: ** * T > * * | | io est | 124.73 | 10:27:4 | 4 | | 3/4 / | | | 12.4 | |
| ci | LTC | ND+CL | NO. | | | SA. | ND ₂ | 7 M.S. | | The Late of the second | RAVEL | 12 74.27 | COBBLE | C PAUL | DEDE. |
| | | ND OL | | F | NE . | ME | ΣÍUM* | , co | ARSE. | FINE | cò | ARSE | COBBLE | | |
| TOTAL SANS | | 4 | J | 44 Ox 5 50 | and the second | STATE OF THE PERSON NAMED IN | 2-600-01-02 | in the second second | Gentle America (Alah | enter value of the | | and the state of the state of | 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 | at the same of the same of | |

GRAIN SIZES

| | SANDS AND GRAVELS | · 实验的证明,但 。 |
|----|--|--|
| ١. | CANDOMANDOMANDO | SOLOWO POOT |
| 33 | DAINDS HIND GLANCED | IMBEDANO/LOGIS S |
| S | というない かんかん かんかん かんかん かんかん | Commercial |
| ٠, | THE THE RESERVE THE STREET STREET | を表現している。 は、1000年により、1000年によっている。 1000年により、1000年によりによりによりによりによりによりによりによりによりによりによりによりによりに |
| à. | THE PARTY OF THE P | 12.15 5 4 5 5 37 - 1 1 1 1 1 2 1 2 1 1 1 1 1 1 1 1 1 1 1 |
| 4 | | TYPE TO SEE THE SECOND |
| 1 | VERY LOOSE | Francis Oracle Assessment |
| - | (1) (1) (1) (1) (1) (1) (1) (1) (1) (1) | 1000 P. C. |
| | THE WAR STORE TO A TOWN | TOTAL CONTRACTOR OF THE PARTY O |
| | LOOSE | 4 - 10 |
| - | | 1.发生,但是一个数量的基础。 |
| × | · · · · · · · · · · · · · · · · · · · | いる。これでは、大学の大学、大学を表現で |
| 4 | MEDIÚM DENSE | 130 ± 30 ± 455 |
| 8 | I DENOL | |
| А. | Subject to the subjec | The ballion of the Charles at |
| 5 | DENSE | 30 = 50 |
| ġ, | STATE OF THE PROPERTY OF THE PARTY OF THE PA | |
| 2 | · 大学、大学、大学、大学、大学、大学、大学、大学、大学、大学、大学、大学、大学、大 | ■ 10 10 10 10 10 10 10 10 10 10 10 10 10 |
| -3 | VERY_DENSE | 2 C OVER 60 4 % |
| | TO THE POLITOR OF THE | ISSUE CONTRACTOR OF THE PROPERTY OF THE PROPER |
| -6 | 是他们的"大学"的"不是"的"大学"的"大学"的"大学"的"大学"的"大学"的"大学"的"大学"的"大学 | 体保護がまたなななくだっ |
| | | 医 类进行系统系统 3 【1972】 1898年 |

| | SILTS AND CLAYS | STRENGTH [†] | BLOWS/FOOT |
|-----|-----------------|---------------------------------------|-------------------|
| | VERY SOFT | 0 - 1/4 | 0.00 |
| 7 | SOFT | 1/4= - 1/2 | 2.342 |
| 8 | FIRM | 1/2 = 1 == | 4 2 8 |
| | VERY STIFF | 2 + 4 | 8 = 16 16 = 32 |
| | HARD | - OVER 4 | OVER 32 |
| · • | | · · · · · · · · · · · · · · · · · · · | |

RELATIVE - DENSITY -

CONSISTENCY

- Number of blows of 140 pound harrimer falling 30 inches to drive at 2 inch 0.D (1-3/8 inch 1:D) split spoon (ASTM D-1586).

 **Unconfined compressive strength in tons/sg (files determined by laboratory testing or approximated by the standard penetration test (ASTM D=1586); pocket penetrometer torvane, or visual observation.

John V. Lowney & Associates

Foundation / Soil / Geological Engineers

| KEY | TO EXPLOR | RATORY BOR | ING LOGS CASTIM-D=2487) |
|------------|-----------------|------------|----------------------------|
| Unified So | oil*Classificat | ion System | CASTIVE DE 2487) |

| PROJECT 277− | NO. | DATI | | | | |
|-----------------|-----|----------|------|-------|--------|--|
| 277- | 22 | July .=1 | 979: | rigur | ea A≑l | |

| 711- | | | |
|------|--|--|--|
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |

| DRIELRIG Continuous Flight Auger | SURFACE | LEVATION AMETER | +345 fee | ATT POSSESS | DBY MG | 79.52 |
|---|---------------------------------|----------------------|----------------------|--------------------|------------------------------------|--------------------------------------|
| DESCRIPTION AND CLASS | | | DEPTH | ツレニコ 造める | AFER TENENCE HEAR- ORDANE | ONEINED PRESSIVE ENGTH (ST) |
| DESCRIPTION AND REMARKS# | SYM-COLOR BOL COLOR Brown | CONSIST. | SOIL (FEET) | SAN SAN PENE | | Negotian Property |
| (Cont.) | Ť. | | 21 - - 22 | | 15. 3.5 \$ | |
| | | | - 23 | | | |
| | | | =-24 25 | | | 3.7 2.7 2.7 |
| *Passing No. ;200 Siever 22% | Brown With Some | | - 26 - 26 | 36. | 10 | |
| | Grey | | 27 - 28 | | | |
| | | | - 29° | | | |
| | | | - 30 - - 31 | 44 | 9 | |
| Bostom of Boring = 31.5 feets | | | - 32 - 32 | | | |
| | | | — 33 — 34 | | | |
| Note: The statification. Lines represent the approximate boundary between soil Stypes and the transition. | | | 35 | | | |
| may be gradual. | | | - 36 - 37 | | | |
| | | | 7 38 1 38 | | | |
| | | | = 39 = - = 40= | | | |
| | | EXP | ORATOR | Y BORIN | GLOG | |
| John V. Lowney & Associate Foundation / Spli / Geological Englisers | | Da | ly City; (| aliforni | onte, of rice a | |
| | P | ROJECT NO. (7=22 | July | | BORING No: EB= | |

| | stire m | |
|--|---------|--|
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |

| | ere Ere Legistica Recep | | | |
|----------------------|---------------------------------------|--------------------|------------|--|
| | ##################################### | | | |
| | | | | |
| | | - 20 da <u>- 2</u> | | |
| | | | | |
| | | | | |
| | | | | |
| | | | | |
| | | | | |
| 74:3 11:3 11:4 | | \$1 24 3 | | |
| | | | | |
| | | | Vig. aTG γ | |
| | | | | |

| | | 1.04 1.04 <u>1.55</u> 2.04 1.5 1.5 1.5 1.5 1.5 1.5 1.5 1.5 1.5 1.5 |
|--|--|--|
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |

| DRILL RIGECOME INVOIS AELIGHT Augen | erene er er erene er er | CÉ ELEVATION IG DIAMETER | ar Burgeraye. | 342. fed | 77 J. J. Y. | OGGED. | ey = NG. Gééb: 6≓I | |
|---|-------------------------|-----------------------------|---------------|--|-------------|--------------------------|--|--|
| DESCRIPTION AND CLASS DESCRIPTION AND REMARKS. | SYM COL | | | DEPTH (FEET) | SAMPLER | RESISTANCE (BLOWS/ET) | SHENTER OF THE SHEET OF THE SHE | UNCONFINED COMPRESSIVE TSTRENGTH |
| WEATHERED SANDSTONE (Cont.) | iligh Brow Grey | m T | | 21 | | 58 . | 18 | |
| | | | | 22 - - 23 - | | | | |
| | | | | 24 - - * - - 25 - * | | | | |
| | | | in a | 26 F 27 - | | 78 | 19 | *** |
| | | | | | | | | |
| | | | | 30 31 31 | | 50/- 6" | 187 187 | |
| | | | | _ 32 + _ 32 + | | | | |
| | | | | - 33 - - 34 - | | | | |
| | | | | 35 - - 36 - | | 75 | 17 | |
| | | | | - 37 - 7 - 38 - | | | | |
| | | | | - 39 - - 40 - | | | | |
| John V. Łowney E. Associate | | | PLØRA | | | A 147 (6) | L OG | |
| Foundation //Soll / Geological Engineers | | PRÖJECTINO | "Daly | INGS City, DATE | Cali | forni Bo | a | B : 3 3 B-3 |

| | les. | | |
|--|------|--|--|

| | | ****** | ***** | | ***** | | T## | | | ***** | | | | ***** | | ***** | | | |
|------|-------|--------|--------|--------|--------|---------------|--------|---------------|-------|-------|--------|---|---------|--------|--------|---------|-------|--------|-----|
| 200 | **** | | **** | | 4 | | - | 22.5 | ***** | **** | | _ | | | B3+B | · | - | - | *** |
| | | 1014 | | | 1000 | | | 10.00 | **** | - | 100 | | | | - 4 | | | | |
| 100 | | 40.0 | | 10.00 | | malifornia di | 45.74 | 1000 | 100 | | | | | | **** | 44.6 | 51 M | u 19 | - 1 |
| | 7,790 | 100 | | 4-1-7 | -11-4- | =:=: | T | | | 100 | 100 | | - | 647 E. | | and the | - | E 14. | - |
| 1100 | | MARK | | | | | **** | ***** | 487 | | | | | | | | | | |
| | 77. | | *** | *12::1 | 117 | | 4:17:4 | #### # | - 4 | 100 | 1400 | | ****** | | :::::: | 4.55 | | ¥: 33 | *** |
| | | | | ***** | ***** | | | **** | - 1 | === | | | | | ***** | | ***** | | ٠., |
| | ***** | | ****** | ****** | | 1.717 | ****** | | | E | | | | ***** | | 42422 | HE: | ****** | **: |
| | | 1000 | 42.44 | | **** | | | | 1 | 2.70 | 717.01 | | | ***** | **** | | | | 323 |
| | | | | | | | | | | | | | | | | | | | |

| | Bit -: [E-4] bi Bit 44: -1-, :1: The all this 10: -1 11: 11: -1 11: 11 |
|---|--|
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| Barraren Britaria de Caración | |
| | |
| | |
| | |

| | | i Mr. Mr. | | |
|--|--|-----------|--|--|
| | | | | |
| | | | | |
| | | | | |
| | | | | |
| | | | | |

| | | 「 |
|--------|--|---|
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |
| r yeşe | | |
| | | |
| | | |

| | liter Krown Voly Still : | | |
|-----------|--------------------------------|-------------|--|
| | | | |
| | | | |
| | | | |
| | Clark Dense | | |
| | | | |
| | | | |
| | icona Denne | 14 | |
| | | | |
| | | | |
| | | | |
| | | | |
| Allen and | | i in i lini | |

| | Macina Sy | |
|--|-------------|--|
| | GELLE ML | |
| | | |
| | Cristal (1) | |
| | | |
| | | |
| | | |
| | | |
| | | |

| | egn Cobelin Sy | |
|--|----------------|--|
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |
| | September | |

| | | - Jan 1 - 26 | | |
|--|--|---|--|--|
| | | | | |
| | | | | |
| | | | | |
| | | | | |
| | | - 1 - 1 - 1 - 1 - 1 - 1 - 1 - 1 - 1 - 1 | | |
| | | | | |
| | | ALUTATO: | | |
| | | | | |
| | | | | |

| EXF. | | | |
|------|---|--|--|
| GPA | | | |
| | 1 | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |

| The contract of the contract o | |
|--|--|
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |
| | |

| | | Samily in the Market State of the Art of the State of the | | | |
|--|--|--|-----------------------------------|--|--|
| | | | | | |
| | | | | | |
| | | | : | | |
| | A CAMPAN AND AND AND AND AND AND AND AND AND A | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| (::::::::::::::::::::::::::::::::::::: | | Hillian († 1865) 1946 - Francisco († 1865) | | | |
| | | | e day 12010da R Rijasara Galai | | |

| | | 4 4 |
|--|--|--------|
| | | |
| | | |
| | | |

| | | | uwi Den | |
|--|--|--|------------|--|
| | | | | |
| | | | | |
| | | | | |
| | | | | |
| | | | | |
| | | | | |
| | | | | |
| | | | | |
| | | | | |
| | | | | |

| | ar is verte 120 Transfer 1 | | | 144 148 | |
|-----|-------------------------------|--|---------------------------------|------------|--|
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | - | | |
| | | | | | |
| | | | - 37 | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | . 14 | | |
| | | | | | |
| | | | | | |
| | | | 7 | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | Sante org Barlingsk <u>o</u> | | |
| 777 | | | | | |

| | | | ng di Tanggan |
|--|--|--|------------------|
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |

| | | | (-17-10) |
|--|--|--|-----------------------|
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |

| | | 74 T | |
|--|--|------------------|--|
| | | | |
| | | | |
| | | | |
| | | 70 — 1 - 10 — | |
| | | | |
| | | | |
| | | | |
| | | 6 | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |

| | | | |
|---|------|--|------|
| ī | | :::::::::::::::::::::::::::::::::::::: | |

| TAGA: | | | | | |
|-------|--|--|---------|--|--|
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |

| URILLIA CONTINUOUS ELICHT AUGE DEETH TO GROUNDWATER | | SURFACE E BORING DI | The standard of the standard o | 142.76 V | 5 feet nches | ree ees | LOĞĞEI DATE DI | ar engantaria | MG -6-20 | |
|---|-------------|------------------------|--|-------------|-------------------------|---------|--------------------------------|-----------------|---|-------------------------------------|
| DESCRIPTION AND CLAS | SIFIC | ATION - | | | DEPTH | PLER | TRATION STANCE: WS/(FT.) | oter Enj (%) | E CAR SPANE | NPINED RESSIVE ENGTH SELFL |
| DESCRIPTION AND REMARKS | SYM- BOL | COLOR | CONSIST | SOL TYPE | | S: | PENE BESI (BLO | CONT | STAN | COMP COMP STR |
| EXISTING PAVEMENT | | Brown | Very | ML | | | 24 | 20 | | |
| * Passing No. 200 Sieve = 56% | ができた。 | Grey | Stiff. | | 2 - 1 - 3 - 1 | | | | , W | |
| SILTY SAND W/Passing No. 200 Sieve = 34% | | Brown | Medium Dense | SM | - 74 - 74 - | | 20 | 12 | | |
| +Passing No:200 Sieve = 47% | | | | | 5 5 | | | 17 | | |
| | | | | | 6 - 6 | を変数 | 18. | | | |
| | | | | | 8 2- | | | | | 74.00 |
| WEARTHERED SANDSTONE | | Brown Grey | | | 91 - | | | | | |
| | | | | | 10 | X 2 | 50/ 5.5º | * | | |
| | | | | | - 12 - 12 | はないない | | | | |
| | | | | | - 13, - - | | | | | |
| | | | | | 14 - -15 - | | Xe / | | | |
| | 6 | | | | - :16 | | 95/ 11!! s | 7 | | |
| Bortom of Boring # 16.4 reet Note: The stratification lines represent the approxi- | | | * | | 17 | を対する | | | | |
| smane boundary between solify strings and the transition may be gradual. | | | | | 19≟ | | | | | |
| | | | | | _20_ | | | | | |
| John V. Kowney & Associate | 5 . | | | 200 | ATORY | ₹ 45° | | | 180 S C C | |
| Zs. Foundation / Soliz Geological Engineers ≥ 28 5 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 | | PR | CITIZEN OJECTNO | | VINGS City; ⊋DATI | Ca | lifor | nia. IORING | OLE FUL | |
| 1000 1000 1000 1000 1000 1000 1000 100 | dis. | 2 | 77-22 | | July 👍 | 197 | 9 . | NO - | EB- | 154 |

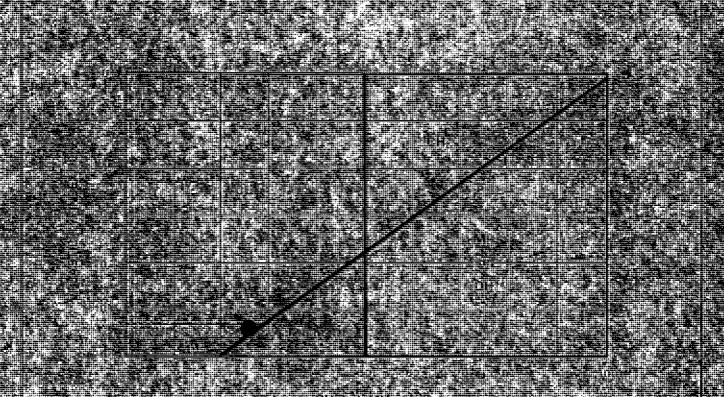
APPENDIX B - LABORATORY INVESTIGATION

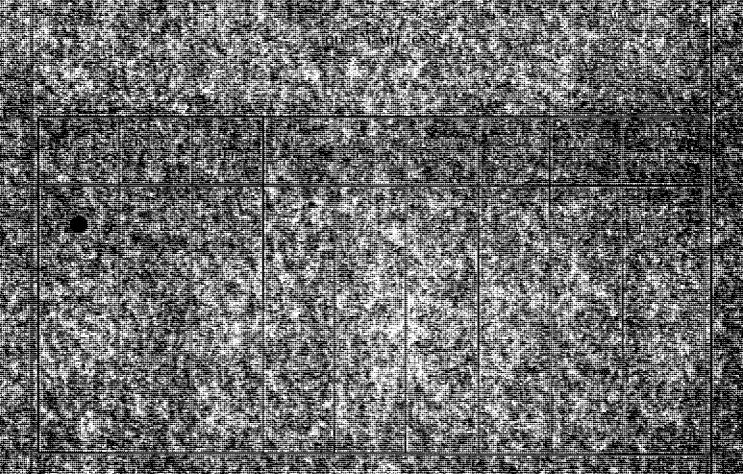
The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site.

The natural water content was determined on 120 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

One Atterberg Limit determination was performed on a sample of the subsurface soils to measure the range of water contents over which this material exhibits plasticity. The Atterberg Limit was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil's expansion potential. The results of this test are presented on Figure B-1 and on the log of the boring at the appropriate sample depths.

The percent soil fraction passing the #200 sieve was determined on 60 samples of the subsurface soils to aid in the classification of these soils. The results of these tests are shown on the boring logs at the appropriate sample depths.





APPENDIX C

GUIDE SPECIFICATIONS - SITE EARTHWORK

FOR

SERRAMONTE OFFICE BUILDING COMPLEX DALY CITY, CALIFORNIA

1. GENERAL

A. Scope of Work

These specifications and applicable plans pertain to all site earthwork including, but not limited to, the furnishing of all labor, tools, and equipment necessary for site clearing and stripping, disposal of excess materials, excavation, preparation of foundation materials for receiving fill, and placement and compaction of fill to the lines and grades shown on the project grading plans.

B. Performance

The Contractor warrants all work to be performed and all materials to be furnished under this contract against defects in materials or workmanship for a period of ______ year(s) from the date of written acceptance of the entire construction work by the Owner or his representative.

Upon written notice of any defect in materials or workmanship during said year period, the Contractor shall, at the option of the Owner, repair or replace said defect and any damage to other work caused by or resulting from such defect without cost to the Owner. This shall not limit any rights of the Owner under the "acceptance and inspection" clause of this contract.

The Contractor shall be responsible for the satisfactory completion of all site earthwork in accordance with the project plans and specifications. This work shall be observed and tested by a representative of John V. Lowney & Associates, hereinafter known as the Geotechnical Engineer. Both the Geotechnical Engineer and the Architect/Engineer are the Owner's representatives. If the Contractor should fail to meet the technical or design requirements embodied in this document and on the applicable plans, he shall make the necessary readjustments until all work is deemed satisfactory as determined by the Geotechnical Engineer and the Architect/Engineer. No deviation from the specifications shall be made except upon written approval of the Geotechnical Engineer or Architect/Engineer.

No site earthwork shall be performed without the presence or review of the Geotechnical Engineer. The Contractor shall notify the Geotechnical Engineer at least twenty-four hours prior to commencement of any aspect of the site earthwork.

The Geotechnical Engineer shall be the Owner's representative to observe the grading operations during the site preparation work and placement and compaction of fills. He shall make visits to the site to familiarize himself generally with the progress and quality of the work. He shall make tests and/or observations to enable him to form an opinion regarding the adequacy of the site preparation, the acceptability of the fill material, and the extent to which the compaction of the fill, as placed, meets the specification requirements. Any fill that does not meet the specification requirements shall be removed and/or recompacted until the requirements are satisfied.

In accordance with generally accepted construction practices, the Contractor shall be solely and completely responsible for working conditions at the job site, including safety of all persons and property during performance of the work. This requirement shall apply continuously and shall not be limited to normal working hours.