APPENDIX D: GEOTECHNICAL INVESTIGATION

ROMIG
ENGINEERS

November 8, 2019
3910-2

Mr. Ed Duffy
Renovattio Construction
625 California Drive
Burlingame, California 94010

RE: GEOTECHNICAL INVESTIGATION
RESIDENTIAL BUILDING
601 CALIFORNIA DRIVE
BURLINGAME, CALIFORNIA

Dear Mr. Duffy:

In accordance with your request, we have performed a geotechnical investigation for your proposed residential building to be constructed at 601 California Drive in Burlingame, California. The accompanying report summarizes the results of our field exploration, laboratory testing, and engineering analysis, and presents our geotechnical recommendations for the proposed project.

We refer you to the text of our report for specific geotechnical recommendations for the project.

Thank you for the opportunity to work with you on this project. Please call if you have any questions or comments concerning the findings, conclusions, or recommendations from our investigation.

Very truly yours,

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GEOTECHNICAL INVESTIGATION
RESIDENTIAL BUILDING
601 CALIFORNIA DRIVE
BURLINGAME, CALIFORNIA

PREPARED FOR:
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625 CALIFORNIA DRIVE
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NOVEMBER 2019

ROMIG ENGINEERS

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GEOTECHNICAL INVESTIGATION
FOR
RESIDENTIAL BUILDING
601 CALIFORNIA DRIVE
BURLINGAME, CALIFORNIA

INTRODUCTION

We are pleased to present this geotechnical investigation report for your proposed residential building to be constructed at 601 California Drive in Burlingame, California. The location of the site is shown on the Vicinity Map, Figure 1. The purpose of this investigation was to evaluate subsurface conditions at the site and to provide geotechnical recommendations for the proposed project.

Project Description

The project consists of constructing a five-story concrete residential building at referenced site in Burlingame. The ground level of the building is expected to include interior covered parking, a small lobby, electrical room, trash area, and stairwells. The 2^{nd} through 5^{th} floor will consist of 25 residential units and common terrace areas. Other improvements will likely include exterior flatwork, paved parking entrance driveway, and landscaping around the building. The proposed building includes a two bay car stacker pit along the southwest side of the building and an elevator pit at the entrance lobby. The car stacker pits will extend to a depth of about 6.5 feet. We understand that the car stacker and elevator pits will be supported on structural mat foundations and will be designed and constructed without underdrains or wall backdrains. The relatively flat site is currently occupied by a gasoline service station which is no longer in operation. The underground storage tanks were removed and the excavations backfilled between June and August of 2019. Structural loads are expected to be moderate as is typical for this type of construction.

Scope of Work

The scope of our work for this investigation was presented in our agreement with you dated May 2, 2019. In order to accomplish this investigation, we performed the following work.

- Review of geologic, geotechnical, and seismic conditions in the vicinity of the site.
- Subsurface exploration consisting of drilling, sampling, and logging two exploratory borings in the area of the proposed building.
- Laboratory testing of selected samples to aid in soil classification and to help evaluate the engineering properties of the soils encountered in our borings.
- Engineering analysis and evaluation of the subsurface data to develop geotechnical design criteria.
- Preparation of this report presenting our geotechnical findings and recommendations for the project.

Limitations

This report was prepared for the exclusive use of Mr. Ed Duffy for specific application to developing geotechnical design criteria for the currently proposed residential building to be constructed at 601 California Drive in Burlingame, California. We make no warranty, expressed or implied, except that our services were performed in accordance with geotechnical engineering principles generally accepted at this time and location. This report was prepared to provide engineering opinions and recommendations only. In the event there are any changes in the nature, design, or location of the project, or if any future improvements are planned, the conclusions and recommendations presented in this report should not be considered valid unless 1) the project changes are reviewed by us, and 2) the conclusions and recommendations presented in this report are modified or verified in writing.

The analysis, conclusions, and recommendations presented in this report are based on site conditions as they existed at the time of our investigation; our understanding of the currently proposed construction; review of readily available reports relevant to the site conditions; and laboratory test results. In addition, it should be recognized that certain limitations are inherent in the evaluation of subsurface conditions, and that certain conditions may not be detected during an investigation of this type. Changes in the information or data gained from any of these sources could result in changes in our conclusions or recommendations. If such changes occur, we should be advised so that we can review our report in light of those changes.

SITE EXPLORATION AND RECONNAISSANCE

Site reconnaissance and subsurface exploration were performed on August 16, 2019. Subsurface exploration was performed using a truck-mounted drill equipped with 8-inch diameter hollow-stem augers. Two exploratory borings were advanced to depths of 25 and 50 feet. The approximate locations of the borings are shown on the Site Plan, Figure 2. The boring logs and the results of our laboratory tests are attached in Appendices A and B, respectively.

Surface Conditions

The site is located in a commercial area at the southwest corner of the intersection of California Avenue and Floribunda Street. At the time of our investigation, the site was occupied by a single story retail gas station building which had a brick and wood siding exterior. The former service station canopy and fuel pump islands had been removed. Asphaltic concrete and concrete pavements extended along the northeast and southeast sides of the building. The pavement had been removed and an exposed soil surface remained where the former underground storage tanks had been removed and backfilled (further discussion regarding the tank removal and backfilling is presented below. Concrete walkways extended along the perimeter of the building. The relatively flat site was landscaped with a few small sized trees located within the landscaping areas along the perimeter of the site.

The depth and width of the existing building foundation is unknown. The perimeter stem walls were generally covered by the wood and brick exterior siding and not visible. The concrete pavement has numerous hairline to ½-inch wide cracks. Roof downspouts were not installed.

Underground Fuel Storage Tank Removal

As you know, we presented geotechnical recommendations for backfilling of the three 11,250 gallon underground fuel storage tanks (UST's) in our letter dated June 17, 2019. The fuel tanks were located to the northeast of the service station building and to the south of the former fuel pump islands as shown on Figure 2. The tanks were removed and backfilled between June and August of 2019. After the tanks were removed, the resulting excavation which was approximately 10 to 11 feet in depth, was backfilled with compacted Class 2 aggregate base. We performed construction observation and testing services during removal and backfilling of the fuel tank excavation. A summary of our observation and testing services was presented on our letter dated October 21, 2019.

We were not involved with backfilling of the underground oil tank excavation. Based on our discussion with you, we understand that the oil tank excavation was also backfilled with compacted Class 2 aggregate base following the same procedure as the fuel tank backfilling.

We briefly reviewed the underground tank removal report, prepared by Golden Gate Tank Removal, Inc., dated September 9, 2019. The report documented the removal of three underground storage tanks (UST), one oil storage tank, fuel dispenser piping, and hydraulic hoist removal activities performed at the site. San Mateo County issued a tank removal inspection report, dated July 1, 2019 (Permit #19-0758). The inspection report confirmed the tank removal, soil sampling and site closure activities related to the underground storage tanks removed from the site.

Subsurface Conditions

At the location of Boring EB-1, we encountered approximately 2.5 feet of very stiff sandy lean clay (possibly disturbed surface soil) of low plasticity underlain by dense to very dense clayey sand which extended to the maximum depth explored of 50 feet.

In Boring EB-2, we encountered approximately 2.5 feet of fill which consisted of very stiff sandy lean clay of low plasticity underlain by approximately 4.5 feet of very dense clayey sand. We then encountered approximately 6 feet of very stiff sandy lean clay of low plasticity underlain by medium dense to very dense clayey sand which extended to the maximum depth explored of 25 feet.

A Liquid Limit of 25 and a Plasticity Index of 9 were measured on a sample of near-surface soil obtained from Boring EB-1. These test results indicate that the surface and near-surface soils at the site generally have low plasticity and a low potential for expansion.

Ground Water

Ground water was measured at a depth of approximately 12 feet in both borings during drilling and sampling. The borings were backfilled with grout shortly after drilling, therefore the measured ground water may not represent a stabilized ground water level. Ground water was encountered at depths ranging between approximately 11 to 23 feet during our investigation at the adjacent site located at 619, 621, and 625 California Drive in 2016. Information in Seismic Hazard Zone Report 113 for the San Mateo Quadrangle (California Geological Survey, 2018) indicates the historic high ground water level in the area of the site is approximately 9 feet below the ground surface.

In addition, our work experience in the immediate area of the site indicates that the stabilized ground water table has been considerably shallower than encountered during our investigation. We measured stabilized ground water at a depth of approximately 6 feet at 755 California Drive (approximately 450 feet to the northwest) and at a depth of 7 feet at 808 Edgehill Drive (approximately 1,000 feet to the northwest). Ground water was also encountered at a depth of 9 feet in the excavation during removal and backfilling of the underground storage tanks in July/August of 2019.

Please be cautioned that fluctuations in the level of ground water can occur due to variations in rainfall, tidal fluctuations, local surface and subsurface drainage patterns, landscaping, and other factors. Based on the findings from our investigation and our local experience, on a design basis, it is our opinion that the highest projected ground water level for basement design at the site is approximately 3 feet below the existing ground surface.

GEOLOGIC SETTING

As part of our investigation, we reviewed our local experience and geologic literature in our files pertinent to the general area of the site. The information reviewed indicates the site is located in an area mapped as Holocene-age basin deposits, Qhb (Brabb, Graymer, Jones, 1998). These deposits are generally expected to consist of very fine silty clay to clay deposits occupying flat-floor basins at the distal edge of alluvial fans located adjacent to the BAY Mud. The deposits also contain unconsolidated, locally organic, and plastic silt and silty clay which was deposited in very flat valley floors. The geology of the site vicinity is shown on the Vicinity Geologic Map, Figure 3.

Based on information presented in a report titled "Geologic and Engineering Aspects of San Francisco Bay Fill" (CDMG, 1969), the site is mapped outside the area which is considered to be underlain by compressible younger Bay Mud (CDMG, 1969). The estimated extent and thickness of the young Bay Mud in the immediate site area is shown on the Contour Map of Bay Mud Thickness, Figure 4.

The lot and immediate site vicinity are located in an area that slopes very gently to the north towards the San Francisco Bay. The site is located at an elevation of approximately 25 feet above sea level.

Faulting and Seismicity

There are no mapped through-going faults across or immediately adjacent to the site and the site is not located within a State of California Earthquake Fault Zone (formerly known as a Special Studies Zone), an area where the potential for fault rupture is considered probable. The closest active fault is the San Andreas fault, located approximately 2.6 miles southwest of the property. Thus, the likelihood of surface rupture occurring from active faulting at the site is remote.

The San Francisco Bay Area is, however, an active seismic region. Earthquakes in the region result from strain energy constantly accumulating due to the northwestward movement of the Pacific Plate relative to the North American Plate. On average about 1.6-inches of movement occur per year. Historically, the Bay Area has experienced large, destructive earthquakes in 1838, 1868, 1906, and 1989. The faults considered most likely to produce large earthquakes in the area include the San Andreas, San Gregorio, Hayward, and Calaveras faults. The San Gregorio fault is located approximately 9.3 miles southwest of the site. The Hayward and Calaveras faults are located approximately 16 and 24 miles northeast of the site, respectively. These faults and significant earthquakes that have been documented in the Bay Area are listed in Table 1 on the following page and are shown on the Regional Fault and Seismicity Map, Figure 5.

Table 1. Earthquake Magnitudes and Historical Earthquakes
Residential Building
Burlingame, California

Fault	Maximum Magnitude (Mw)	Historical Earthquakes	Estimated Magnitude
San Andreas	7.9	1989 Loma Prieta	6.9
		1906 San Francisco	7.9
		1865 N. of 1989 Loma Prieta Earthquake	6.5
		1838 San Francisco-Peninsula Segment	6.8
		1836 East of Monterey	6.5
Hayward	7.1	1868 Hayward	6.8
		1858 Hayward	6.8
Calaveras	6.8	1984 Morgan Hill	6.2
		1911 Morgan Hill	6.2
		1897 Gilroy	6.3
San Gregorio	7.3	1926 Monterey Bay	6.1

In the future, the subject property will undoubtedly experience severe ground shaking during moderate and large magnitude earthquakes produced along the San Andreas fault or other active Bay Area fault zones. Using information from recent earthquakes, improved mapping of active faults, ground motion prediction modeling, and a new model for estimating earthquake probabilities, a panel of experts convened by the U.S.G.S. have concluded there is a 72 percent chance for at least one earthquake of Magnitude 6.7 or larger in the Bay Area before 2043. The Hayward fault has the highest likelihood of an earthquake greater than or equal to magnitude 6.7 in the Bay Area, estimated at 33 percent, while the likelihood on the San Andreas and Calaveras faults is estimated at approximately 22 and 26 percent, respectively (Aagaard et al., 2016).

Earthquake Design Parameters

The State of California currently requires that buildings and structures be designed in accordance with the seismic design provisions presented in the California Building Code. Based on site geologic conditions and on information from our subsurface exploration at the site, the site may be classified as Site Class D, stiff soil, in accordance with Chapter 20 of ASCE 7-16. Spectral Response Acceleration parameters and site coefficients may be taken directly from the U.S.G.S. website based on the longitude and latitude of the site. For site latitude (37.5817), longitude (-122.3506) and Site Class D, design parameters are presented on Table 2.

Table 2. 2016 and 2019 CBC Seismic Design Criteria
Residential Building
Burlingame, California

Spectral Response Acceleration Parameters		2016 Design Values	2019 Design Values
Mapped Value for Short Period -	S_S	2.075	2.004
Mapped Value for 1-sec Period -	S_1	0.981	0.827
Site Coefficient -	F_a	1.0	1.0
Site Coefficient -	F_v	1.5	-
Adjusted for Site Class -	S_{MS}	2.075	2.004
Adjusted for Site Class -	S_{M1}	1.472	-
Value for Design Earthquake -	S_{DS}	1.383	1.36
Value for Design Earthquake -	S_{D1}	0.981	-

Geologic Hazards

As part of our investigation, we reviewed the potential for geologic hazards to impact the site and the proposed building, considering the geologic setting and the soils encountered during our investigation. The results of our review are presented below and in the following sections of our report.

- Fault Rupture - The site is not located in a State of California Earthquake Fault Zone or area where fault rupture is considered likely. Therefore, active faults are not believed to exist beneath the site and the potential for fault rupture at the site is low.

- Ground Shaking - The site is located in an active seismic area. Moderate to large earthquakes are probable along several active faults in the greater Bay Area over a 30 to 50 year design life. Strong ground shaking should therefore be expected several times during the design life of the service center facility, as is typical for sites throughout the Bay Area. The building should be designed in accordance with current earthquake resistance standards.

- Differential Compaction - Differential compaction can occur during moderate and large earthquakes when soft or loose, natural or fill soils are densified and settle, often unevenly across a site. The native soils encountered in our borings above the projected high ground water level were generally stiff to hard clays and dense to very dense sands. In our opinion, the likelihood of significant differential compaction affecting the building is low provided the recommendations presented in our report are followed during design and construction.

  Several feet of surface clayey fill and/or disturbed surface soils were encountered in both borings at the site and up possibly up to approximately 7 feet of backfill for the underground oil tank is present. In our opinion, some static and seismic related differential settlement of slabs-on-grade and exterior flatwork/pavement areas is possible in areas where the existing fill is not excavated and properly compacted as discussed below.

Liquefaction Analysis

Severe ground shaking during an earthquake can cause loose to medium dense granular soils to densify. If the granular soils are below ground water, their densification can cause increases in pore water pressure, which can lead to soil softening, liquefaction, and ground deformation. Soils most prone to liquefaction are saturated, loose to medium dense, silty sands and sandy silts with limited drainage, and in some cases, sands and gravels that are interbedded with or that contain seams or layers of impermeable soil.

The clayey sand encountered at the site below the highest projected ground water depth, which is estimated to be about 3 feet below the ground surface, was considered in our liquefaction analysis. Soils with normalized standard penetration test, $(N_1)_{60}$, greater than 30 blows per feet were considered too dense to liquefy.

To evaluate the potential for earthquake-induced liquefaction of the sandy soils at the site within the depth of exploration, we performed a liquefaction analysis of the data from our borings generally following the methods described in the 2008 publication by Idriss and Boulanger titled "Soil Liquefaction During Earthquakes".

Based on our analysis, the clayey sand encountered in Boring EB-2 between depths of approximately 17 to 22 feet is potentially prone to liquefaction when subjected to a peak ground acceleration (PGA) of 0.81, the PGA_M for maximum considered earthquake based on ASCE 7-10. Based on the results of our analysis of these sand and gravel layers, we estimate that total settlement of about ¾-inch could occur within this sand strata due to severe ground shaking caused by a major earthquake. In our opinion, differential settlement of about ½- to ¾-inch over a horizontal distance of about 50 feet is possible at the ground surface from this amount of total settlement.

Several feet of soft to firm surface fill was encountered along the southeast area of the site. In our opinion, some static and seismic related differential settlement of slabs-on-grade and exterior flatwork/pavement areas is possible in areas where the existing fill is not excavated and properly compacted.

CONCLUSIONS

From a geotechnical viewpoint, the site is suitable for the proposed five-story residential building provided the recommendations presented in this report are followed during design and construction. Specific geotechnical recommendations for the project are presented in the following sections of this report.

The primary geotechnical concerns for the proposed improvements are the presence of up to 2.5 feet of surface fill and/or disturbed surface soil material encountered in our borings, the medium dense sand strata encountered in Boring EB-2 which is susceptible to liquefaction, a relatively high ground water table, and the potential for severe ground shaking during a major earthquake. In our opinion, the proposed building may be supported on a structural mat or conventional spread footing foundation bearing in stiff native/dense soils below any existing surface fill, or on properly compacted structural fill (including the properly compacted tank backfill). These preliminary foundation recommendations are based on the anticipated structural loading conditions. However, once the specific dead and live loads and the foundation configuration have been developed, we should update the range of expected foundation settlement and determine if revision to these preliminary recommendations are appropriate.

Since we assume that basement drainage will not be installed, the car stacker and elevator pit floors which extend below the design ground water level of 3 feet will need to be designed to resist potential hydrostatic uplift pressure and the pit walls will need to be designed to resist lateral loads from undrained soil backfill and full hydrostatic pressure. In addition, the structural engineer should confirm that the building will not become buoyant assuming that ground water is present at the design ground water elevation of 3 feet below the existing ground surface.

In our opinion, any existing fill/disturbed surface soil not removed during grading for the building pad should be excavated and recompacted below the building, exterior flatwork, and any other site improvements during site preparation and before foundation construction. The reworking of the fill/disturbed surface soil material and subgrade preparation should proceed as recommended in the section of this report titled "Earthwork." During pad grading, we should observe and test the condition of the backfill at former oil tank location and verify that the backfill was compacted per our recommendation. If poorly compacted backfill is encountered, we will recommend that the backfill be removed and recompacted.

Please note that some of the sandy soils encountered in our borings within the lower portion of the anticipated depths of the car stacker and elevator pit excavations were judged to have limited cohesion and may be prone to sloughing and/or caving if excavated near-vertical. This information should be considered by the contractor when establishing temporary shoring/cut slope criteria for the excavation and other temporary slopes and cuts. Depending on the ground water level at the time of construction, dewatering may be needed during construction of the car stacker and elevator pits and underground utility improvements.

Because subsurface conditions may vary from those encountered at the locations of our borings, and to observe that our recommendations are properly implemented, we recommend that we be retained to 1) review the grading and foundation plans for conformance with our recommendations; and 2) observe and test during the earthwork and foundation phases of construction.

FOUNDATIONS

Spread Footing Foundations

In our opinion, on a preliminary basis, the building may be supported on a conventional spread footing foundation bearing on stiff native soils or properly compacted structural fill. All continuous footings should have a width of at least 15 inches and should extend at least 30 inches below exterior grade and at least 24 inches below the bottom of concrete slabs-on-grade, whichever is deeper. Continuous footings with at least these minimum dimensions may be designed for an allowable bearing pressure of 3,000 pounds per square foot for dead loads, 3,500 pounds per square foot for live loads with a one-third increase allowed when considering additional short-term wind or seismic loading.

We recommend that any at-grade isolated footings and portions of continuous footings which are parallel to the car stacker or elevator pit walls be supported on undisturbed native soil below the basement wall backfill. Surcharge pressures from these footings should be applied to the basement walls in accordance with the criteria presented in the section of this report titled "Basement Retaining Walls." Footings that cross the basement wall backfill should be designed to span across the backfill zone.

All footings located adjacent to utility lines should bear below a 1:1 plane extending up from the bottom edge of the utility trench. We recommend that continuous foundations be designed with sufficient depth and reinforcing to tolerate the estimated differential settlement.

Our representative should observe all footing excavations prior to placement of reinforcing steel to confirm that they expose suitable material and have been properly cleaned. If soft or loose soils are encountered in the foundation excavations, our field representative may require overexcavation and/or compactive effort or a deeper footing depth before the reinforcing steel is placed.

Lateral Loads For Footings

Lateral loads may be resisted by friction between the bottom of the footings and the supporting subgrade. A coefficient of friction of 0.3 may be assumed. In addition to friction, lateral resistance may be provided by passive soil pressure acting against the sides of foundations cast neat in footing excavations within the stiff/medium dense native soils. We recommend assuming an equivalent fluid pressure of 300 pounds per cubic foot for passive soil resistance, where appropriate. The upper foot of passive soil resistance should be neglected where soil adjacent to the footing will be landscaped or subject to softening from rainfall and/or surface water runoff.

Structural Mat Foundation

As an alternative to the spread footing foundation described above, the at-grade areas of the building, as well as the car stacker and elevator pits, may be supported on a reinforced concrete mat foundation bearing on a properly prepared and compacted soil subgrade. On a preliminary basis, the mat may be designed for an average allowable bearing pressure of 2,000 pounds per square foot for combined dead plus live loads, with maximum localized bearing pressures of 3,000 pounds per square foot at column or wall loads. These pressures may be increased by one-third for total loads including wind or seismic forces. These pressures are net values; the weight of the mat may be neglected in design. It would be preferable for the mat foundation to have a thickened perimeter edge that extends at least 8 inches into the soil subgrade below the bottom of the mat or at least 4 inches below the base of the capillary break rock section. This should improve edge stiffness, reduce the potential for mat slab dampness, and increase resistance to lateral loads imposed on the mat.

The mat should be reinforced to provide structural continuity and to permit spanning of local irregularities. A modulus of subgrade reaction (Kv1) of 80 pounds per cubic inch may be assumed for the mat subgrade. This value is based on a 1-foot square bearing area and should be scaled to account for mat foundation size effects. Alternatively, based on the anticipated building load and differential static settlement, on a preliminary basis a modulus of subgrade reaction (Kv) of 25 pounds per cubic inch (pci) may be assumed for the mat subgrade.

The mat foundation should be reinforced to provide structural continuity and to permit spanning of local irregularities. We recommend the mat be designed with sufficient depth and reinforcing to be able to tolerate the estimated differential settlements.

Prior to mat construction, the mat subgrade should be proof-rolled to provide a smooth firm surface for mat support. Where dampness of the mat would be undesirable, a high-quality membrane vapor barrier should be installed below the mat as described in the section of this report titled "Slabs-on-Grade."

Lateral Loads for Mat Foundations

Lateral loads may be resisted by base friction between the vapor barrier or damp proofing membrane below the mat and the supporting subgrade and by passive soil pressure acting against the sides of foundations elements and basement walls. The structural engineer should consult with the membrane manufacturer for the coefficient of friction to be assumed for design.

In addition, lateral resistance may be provided by passive soil pressure acting against the sides of foundations cast neat in footing excavations or backfilled with compacted structural fill or shored excavation for the below grade pit areas. We recommend assuming an equivalent fluid pressure of 300 pounds per cubic foot for passive soil resistance, where appropriate. The upper foot of passive soil resistance should be neglected where soil adjacent to the mat foundation will be landscaped or subject to softening from rainfall and/or surface water runoff, rather than covered with a slab or pavement. The ultimate passive soil resistance acting on the car stacker and elevator pit mat foundations should be limited to 3,000 pounds per square foot. This passive pressure assumes lateral deflection at the top of the mat foundation on the order of ¼- to ½-inch.

Car Stacker and Elevator Pit Water Proofing

We have not provided recommendations regarding the method or details for car stacker and elevator pit damp-proofing since design of damp-proofing systems is outside of our scope of services and expertise. Installing adequate damp-proofing below and behind the edges of the pit floor and behind the pit walls is essential for the success of the below grade structure. Placing concrete with a low water cement ratio should be considered as one step of good damp-proofing as discussed in the Slab-On-Grade section below. The damp-proofing system below the pit mat slab may be placed directly on the compacted soil subgrade, a 4- to 6-inch section of crushed rock or baserock, or on a thin working slab, as determined by the water-proofing consultant.

Settlement

Based on the bearing capacity values presented above, on a preliminary basis, in our experience, the 30-year post-construction differential settlement due to static loads is not expected to exceed 1-inch across the proposed building, provided the building foundations are designed and constructed as recommended. Less differential movement would be expected across a structural mat foundation. Once the range of dead and live loads and the foundation configuration have been developed, we should update the magnitude of total and differential foundation settlement to help establish if an adjustment should be made to the allowable bearing capacity values and differential movement.

Additional differential settlement may occur as a result of liquefaction caused by severe ground shaking during a major earthquake, as discussed earlier.

SLABS-ON-GRADE

General Slab Considerations

To reduce the potential for movement of the slab subgrade, at least the upper 6-inches of subgrade soil should be scarified and compacted at a moisture content at least 2 percent above the laboratory optimum. The soil subgrade should be kept moist up until the time the non-expansive fill, aggregate base, and/or vapor barrier is placed. Slab subgrades and non-expansive fill should be prepared and compacted as recommended in the section of this report titled "Earthwork." Overly soft or moist soils should be removed from slab-on-grade areas. Exterior flatwork and interior slabs-on-grade should be underlain by a layer of non-expansive fill as recommended below. The non-expansive fill should consist of Class 2 aggregate base or clayey soil with a Plasticity Index of 15 or less.

Considering the potential for some differential movement of the surface and near-surface soils, we expect that reinforced slabs will perform better than unreinforced slabs. Consideration should be given to using a control joint spacing on the order of 2 feet in each direction for each inch of slab thickness.

Exterior Flatwork

Concrete walkways and exterior flatwork should be at least 4 inches thick and should be constructed on at least 6 inches of Class 2 aggregate base. For improved performance, exterior slabs-on-grade, may constructed with a thickened edge to improve edge stiffness and to reduce the potential for water seepage under the edge of the slabs and into the underlying base and subgrade. In our opinion, the thickened edges should be at least 8 inches wide and ideally should extend at least 4 inches below the bottom of the underlying aggregate base layer.

Interior Slabs

Concrete slab-on-grade floors for the building (other than the mat slab) should be constructed on a layer of non-expansive fill at least 10-inches thick and constructed on a properly prepared and compacted soil subgrade. Since the ground level garage floor for the building will support vehicle loads, we recommend that the garage floor slabs should be designed more heavily reinforced and at least 5 to 6 inches in thickness, in our opinion. Recycled aggregate base should not be used for non-expansive fill below interior slabs-on-grade, since adverse vapor could occur from crushed asphalt components.

Moisture Considerations

In areas where dampness of concrete floor slabs or mat would be undesirable, such as within building interiors, concrete slabs and mat should be underlain by at least 4 inches of clean, free-draining gravel, such as ½-inch to ¾-inch clean crushed rock with no more than 5 percent passing the ASTM No. 200 sieve. Pea gravel should not be used. The crushed rock should be compacted with vibratory equipment. To reduce vapor transmission up through at-grade concrete floor slabs, the crushed rock section should be covered with a high-quality, UV-resistant membrane vapor retarder meeting the minimum ASTM E 1745, Class C requirements or better. If moisture-sensitive floor coverings are proposed and/or additional protection is desired by the owner, a higher quality vapor barrier conforming to the requirements of ASTM E 1745 Class A, with a water vapor transmission rate less than or equal to 0.01 perms (such as 15-mil thick "Stego Wrap Class A") may be used rather than a Class C vapor retarder. The vapor retarder or barrier should be placed directly below the concrete slab. Sand above the vapor retarder/barrier is not recommended. The vapor retarder/barrier should be installed in accordance with ASTM E 1643. All seams and penetrations of the vapor barrier should be sealed in accordance with manufacturer's recommendations.

As discussed above, installing adequate damp-proofing below and behind the edges of the car stacker and elevator pit floor and behind the walls is essential for the success of the below grade structures.

The permeability of concrete is affected significantly by the water:cement ratio of the mix, with lower water:cement ratios producing more damp-resistant slabs and higher strength. Where moisture protection is important and/or where the concrete will be placed directly on the vapor barrier, the water:cement ratio should be 0.45 or less. To increase the workability of the concrete, mid-range plasticizers may be added to the mix. Water should not be added to the mix unless the slump is less than specified and the water:cement ratio will not exceed 0.45. Other steps that may be taken to reduce moisture transmission through concrete slabs-on-grade include moist curing for 5 to 7 days and allowing the slab to dry for a period of two months or longer prior to placing floor coverings. Prior to installation of floor coverings, it may be appropriate to test the slab moisture content for adherence to the manufacturer's requirements to determine whether a longer drying time is necessary.

CAR STACKER/ELEVATOR PIT WALLS

We recommend that the car stacker and elevator pit walls with level backfill that are not free to deflect or rotate be assumed to be undrained and should be designed to resist an equivalent fluid pressure of 85 pounds per cubic foot plus an additional uniform lateral pressure of 8H pounds per square foot (where H is the height of the wall in feet). Where the basement walls will be subjected to surcharge loads, such as from foundations or construction loading, the walls should be designed for an additional uniform lateral pressure equal to one-half of the surcharge pressure.

Based on the site peak ground acceleration (PGA), on Seed and Whitman (1970); Al Atik and Sitar (2010); and Lew et al. (2010); seismic loads on retaining walls that can yield may be simulated by a line load of $7H^2$ (in pounds per foot, where H is the wall height in feet). Seismic loads on walls that cannot yield may be subjected to a seismic load as high as about $13H^2$. This seismic surcharge line load should be assumed to act at 1/3H above the base of the wall (in addition to an undrained active wall design pressure of 45 pounds per cubic foot).

A reliable water-proofing system should be installed below and around the edges of the foundation and slab floor as well as behind the car stacker and elevator pit walls.

Backfill (if any) behind the retaining walls should be compacted to at least 90 percent relative compaction using light compaction equipment. If heavy equipment is used for compaction of wall backfill, the walls may need to be temporarily braced.

The car stacker pit walls should be supported on a structural mat foundation designed in accordance with the recommendations presented previously.

VEHICLE PAVEMENTS

Asphalt Concrete Pavements

Based on the anticipated composition of the surface soils, and an estimated traffic index for the proposed pavement loading conditions, we developed the minimum pavement sections presented in Table 3 below based on Procedure 630 of the Caltrans Highway Design Manual.

The Traffic Indices used in our pavement thickness calculations are considered reasonable values for this development and are based on engineering judgment rather than on detailed traffic projections. Asphalt concrete and aggregate base should conform to and be placed in accordance with the requirements of the Caltrans Standard Specifications, latest edition, except that compaction should be based on ASTM Test D1557.

Table 3. Pavement Sections
Residential Building
Burlingame, California

Traffic Loading Condition	Design Traffic Index	Asphalt Concrete (inches)	Aggregate Base* (inches)	Total Thickness (inches)
Automobile Parking	4.0	3.0	7.0	10.0
Automobile Access	4.5	3.0	8.0	11.0
Light Truck Traffic	5.0	3.0	9.0	12.0
Moderate Truck Traffic	6.0	4.0	11.0	15.0

*Caltrans Class 2 Aggregate Base (minimum R-value = 78).

We recommend that measures be taken to limit the amount of surface water that seeps into the aggregate base and subgrade below vehicle pavements, particularly where the pavements are adjacent to landscape areas. Seepage of water into the pavement base material tends to soften the subgrade, increasing the amount of pavement maintenance that is required and shortening the pavement service life. Deepened curbs extending 4-inches below the bottom of the aggregate base layer are generally effective in limiting excessive water seepage. Other types of water cutoff devices or edge drains may also be considered to maintain pavement service life.

Portland Cement Concrete Pavements

If Portland Cement Concrete (PCC) pavements are to be used on portions of the site, the minimum required thickness of the PCC pavements should be based on the anticipated traffic loading, the modulus of rupture of the concrete that will be used for pavement construction, and the composition and supporting characteristics of the soil subgrade below the pavement section.

To provide a general guideline for the minimum required thickness of PCC pavements, we used information in the Portland Cement Association publication titled "Thickness Design for Concrete Highway and Street Pavements." We assumed "low" subgrade support from the on-site soils, considering typical residential street traffic (up to 25 daily trucks with maximum single axle loads of 22 kips and maximum tandem axle loads of 36 kips), aggregate-interlock joints (i.e. no dowels), no concrete shoulder or curb, a modulus of rupture of concrete of 550 psi (which correlates to a concrete compressive strength of approximately 3,700 psi), at least 10 inches of Class 2 aggregate base below the PCC pavement, and 20-year pavement service life. Sufficient control joints should be incorporated in the design and construction to limit and control cracking.

Based on the design assumptions described above, a PCC pavement with a thickness of at least 6 inches would be adequate for average daily truck traffic (ADTT) of one; a thickness of at least 6.5 inches would be adequate for ADTT of 13; and a thickness of at least 7 inches would be adequate for ADTT of 110.

EARTHWORK

Clearing and Subgrade Preparation

All deleterious materials, such as existing pavements, utilities to be abandoned, vegetation, root systems, surface fills, topsoil, etc. should be cleared from areas of the site to be built on or paved. The actual stripping depth should be determined by a member of our staff in the field at the time of construction. Excavations that extend below finished grade should be backfilled with structural fill that is water-conditioned, placed, and compacted as recommended in the section of this report titled “Compaction.”

After the site has been properly cleared, stripped, and excavated to the required grades, exposed soil surfaces in areas to receive structural fill or slabs-on-grade should be scarified to a depth of 6 inches, moisture conditioned, and compacted as recommended for structural fill in the section of this report titled "Compaction." On-site soils, foundation and utility trench excavations, and slab and pavement subgrades should be kept in a moist condition throughout the construction period.

A member of our staff should observe the car stacker and elevator pit excavations to evaluate whether scarification and compaction or proof rolling of the excavation bottom is needed.

If a temporary ramp is constructed to access the car stacker pit excavation, the ramp should be properly backfilled with compacted on-site soil as recommended in this report for structural fill. A member of our staff should observe and test during backfilling of the temporary entrance ramp and car stacker and elevator pit retaining walls.

Building Pad Recommendations

In our opinion, the existing fill, disturbed surface soils, and oil tank excavation should be excavated and recompacted below the building, exterior flatwork, pavements, and other site improvements, with a 5 foot overbuild, where possible. The fill should be excavated down to stiff native soil and compacted under our direction. Imported backfill materials should be approved by a member of our staff prior to delivery to the site. The backfill should be moisture conditioned, and compacted as recommended in the section of this report titled "Compaction." A member of our staff should observe and test during re-working of the building pad, as required.

Material For Fill

All on-site soil containing less than 3 percent organic material by weight (ASTM D2974) may be suitable for use as structural fill. Structural fill should not contain rocks or pieces larger than 6 inches in greatest dimension and no more than 15 percent larger than 2.5 inches. Imported, non-expansive fill should have a Plasticity Index no greater than 15, should be predominately granular, and should have sufficient binder so as not to slough or cave into foundation excavations or utility trenches. Recycled aggregate base should not be used for non-expansive fill at building interior. A member of our staff should approve proposed import materials prior to their delivery to the site.

Temporary Slopes, Excavations and Dewatering

The contractor should be responsible for the design and construction of all temporary slopes and any required shoring. Shoring and bracing should be provided in accordance with all applicable local, state, and federal safety regulations, including current OSHA excavation and trench safety standards.

Due to the potential for variation of the on-site soil, field modification of temporary cut slopes may be required. Unstable materials encountered on excavations and slopes during and after excavation should be trimmed off even if this requires cutting the slopes back to a flatter inclination.

Please note that granular soils may be present within the depth of the car stacker and elevator pit and utility trench excavations. These sandy soils may have limited cohesion and could be prone to sloughing and/or caving if excavated near-vertical. This information should be considered by the contractor when establishing temporary shoring/cut slope criteria for excavations and other temporary slopes and cuts.

As discussed above, ground water will could seasonally be as high as approximately 3 feet below grade. Therefore, construction dewatering may be required depending on the depth of temporary excavations for utility trenches, car stacker and elevator pits, and the ground water level at the time of excavation.

Temporary dewatering for construction should be the responsibility of the contractor. The selection of equipment and methods of dewatering should be left up to the contractor and, due to the variable nature of the subsurface conditions, they should be aware that modifications to the dewatering system may be required during construction depending on the conditions encountered.

Preferably, dewatering of deep utility trench excavations should be carried out in such a manner as to maintain the ground water a minimum of 2 feet below the bottom of the trench excavations. The contractor should design a system to achieve this. Depending upon the depth and dimensions of the excavations, we anticipate that dewatering may be able to be accomplished from pumping from sumps.

Special considerations may be required prior to discharge of ground water from dewatering activities depending on the quality of the ground water, and environmental impacts at the site or at nearby locations. These requirements may include storage, testing and/or treatment under permit prior to discharge.

Protection of structures near cuts should also be the responsibility of the contractor. In our experience, a preconstruction survey is generally performed to document existing conditions prior to construction, with intermittent monitoring of the structures during construction.

Compaction

Scarified soil surfaces and all structural fill should be compacted in uniform lifts no thicker than 8-inches in uncompacted thickness, conditioned to the appropriate moisture content, and compacted as recommended for structural fill in Table 4 on the following page. The relative compaction and moisture content recommended in Table 4 is relative to ASTM Test D1557, latest edition.

Table 4. Compaction Recommendations
Residential Building
Burlingame, California

	Relative Compaction*	Moisture Content*
General		
• Scarified subgrade in areas to receive structural fill.	90 percent	Above optimum
• Structural fill composed of native soil.	90 percent	Above optimum
• Structural fill composed of non-expansive fill.	90 percent	Above optimum
• Structural fill below a depth of 4 feet.	92 percent	Above optimum
Pavement Areas		
• Upper 6-inches of soil below baserock.	95 percent	Near optimum
• Aggregate baserock.	95 percent	Near optimum
Utility Trench Backfill		
• On-site soil.	90 percent	Near optimum
• Imported sand	95 percent	Near optimum

* Relative to ASTM Test D1557, latest edition.

Car Stacker and Elevator Pit Excavation Support

Based on the assumed finished floor elevation of the car stacker and elevator pits, temporary excavations up to approximately 8 feet deep (depending on the finished floor elevation and foundation depth) will be required in order to construct the pits. If laying back the excavation is not possible, the walls of the pit excavations may be supported by several methods including tiebacks, soldier beams and wood lagging, soil nails, braced shoring or potentially other methods. 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 and improvements 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 preconstruction review. In addition, it should be the contractor's responsibility to undertake a preconstruction survey with benchmarks and photographs of the adjacent properties.

Finished Slopes

We recommend that finished slopes be cut or filled to an inclination no steeper than 3:1 (horizontal:vertical). Exposed slopes may be subject to minor sloughing and erosion, which could require periodic maintenance. We recommend that all slopes and soil surfaces disturbed during construction be planted with erosion-resistant vegetation.

Surface Drainage

Finished grades should be designed to prevent ponding and to drain surface water away from foundations and edges slabs and pavements, and toward suitable collection and discharge facilities. Slopes of at least 2 percent are recommended for flatwork and pavement areas with 5 percent preferred in landscape areas within 8 feet of the structures, where possible. At a minimum, splash blocks should be provided at the ends of downspouts to carry surface water away from perimeter foundations. Preferably, downspout drainage should be collected in a closed pipe system that is routed to a storm drain system or other suitable discharge outlet.

Infiltration basins or bioswales, if any, preferably should not be placed within about 10 feet of shallow foundation supported structures or slab or flatwork areas. Drains should be provided for infiltration basins that direct water to an appropriate outlet as required by the civil engineer.

Drainage facilities should be observed to verify that they are adequate and that no adjustments need to be made, especially during first two years following construction. We recommend that an as-built plan be prepared to show the locations of all surface and subsurface drain lines and clean-outs. Drainage facilities should be periodically checked to verify that they are continuing to function properly. The drainage facilities will probably need to be periodically cleaned of silt and debris that may build up in the lines.

FUTURE SERVICES

Plan Review

Romig Engineers should review the completed grading and foundation plans for conformance with the recommendations presented in this report. We should be provided with these plans as soon as possible upon their completion in order to limit the potential for delays in the permitting process that might otherwise be attributed to our review process. In addition, it should be noted that many of the local building and planning departments now require "clean" geotechnical plan review letters prior to acceptance of plans for their final review. Since our plan reviews often do result in recommendations for additional changes to the plans, our generation of a "clean" review letter often requires two iterations. At a minimum, we recommend that the following note be added to the general note sections of the architectural, structural, and civil plans:

"Earthwork, utility trench backfilling, slab subgrade preparation, foundation and slab construction, pavement construction, elevator and car stacker pit backfilling, and site drainage should be performed in accordance with the geotechnical report prepared by Romig Engineers, Inc., dated November 8, 2019. Romig Engineers should be notified at least 48 hours in advance of any earthwork or foundation construction and should observe and test during earthwork and foundation construction as recommended in the geotechnical report."

Construction Observation and Testing

Earthwork and foundation construction should be observed and tested by us to 1) confirm that subsurface conditions are compatible with those used in the analysis and design; 2) observe compliance with the design concepts, specifications and recommendations; and 3) allow design changes in the event that subsurface conditions differ from those anticipated. The recommendations in this report are based on a limited number of borings. The nature and extent of variation across the site may not become evident until construction. If variations are exposed during construction, it will be necessary to reevaluate our recommendations.

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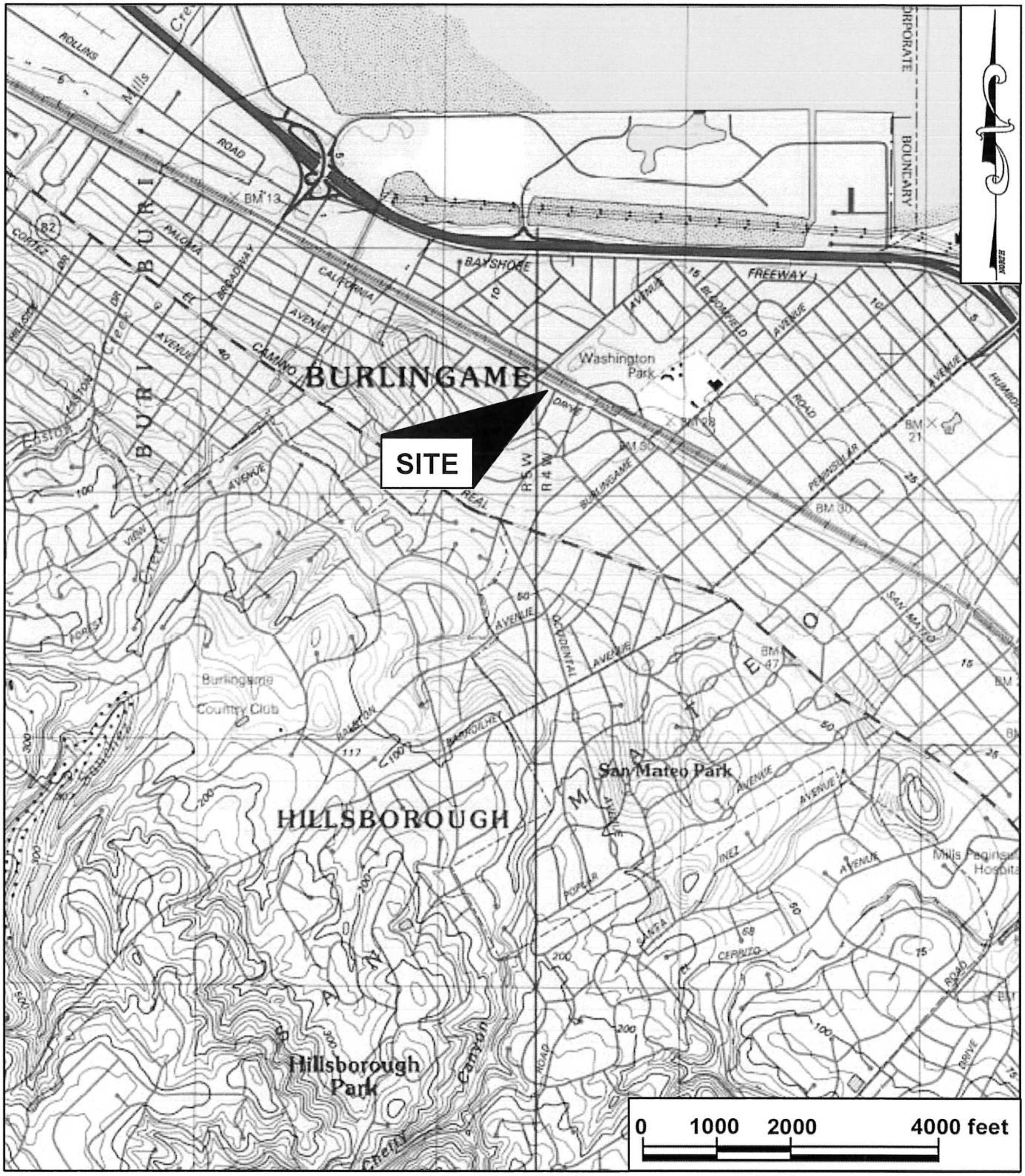
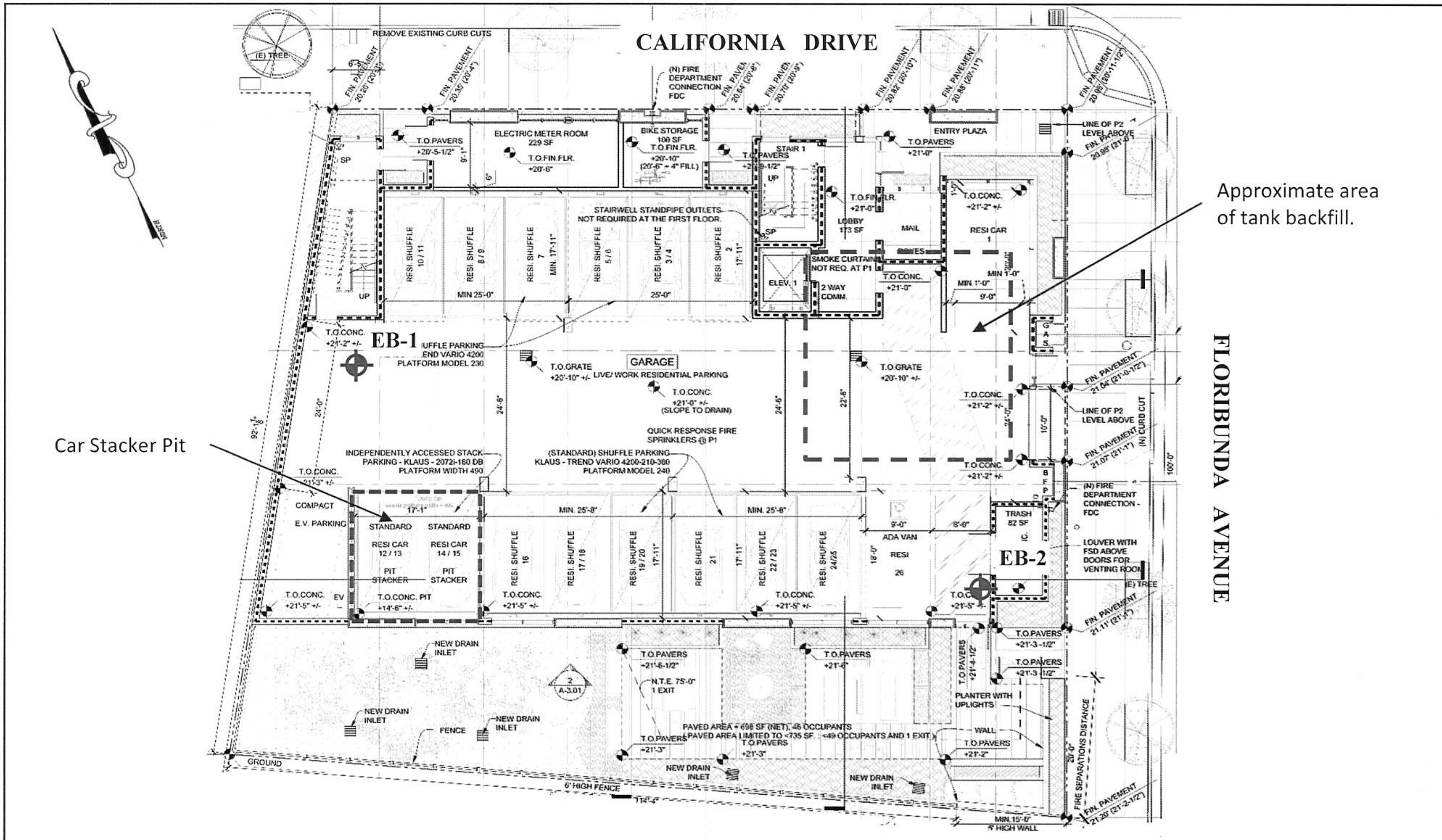


Scale: 1 inch = 2000 feet
Base is United States Geological Survey San Mateo 7.5 Minute Quadrangle, dated 1997.

VICINITY MAP
DUFFY RESIDENTIAL BUILDING
BURLINGAME, CALIFORNIA

FIGURE 1
NOVEMBER 2019
PROJECT NO. 3910-2

ROMIG ENGINEERS



LEGEND

EB-1 ⊕ Approximate Locations of Exploratory Borings.

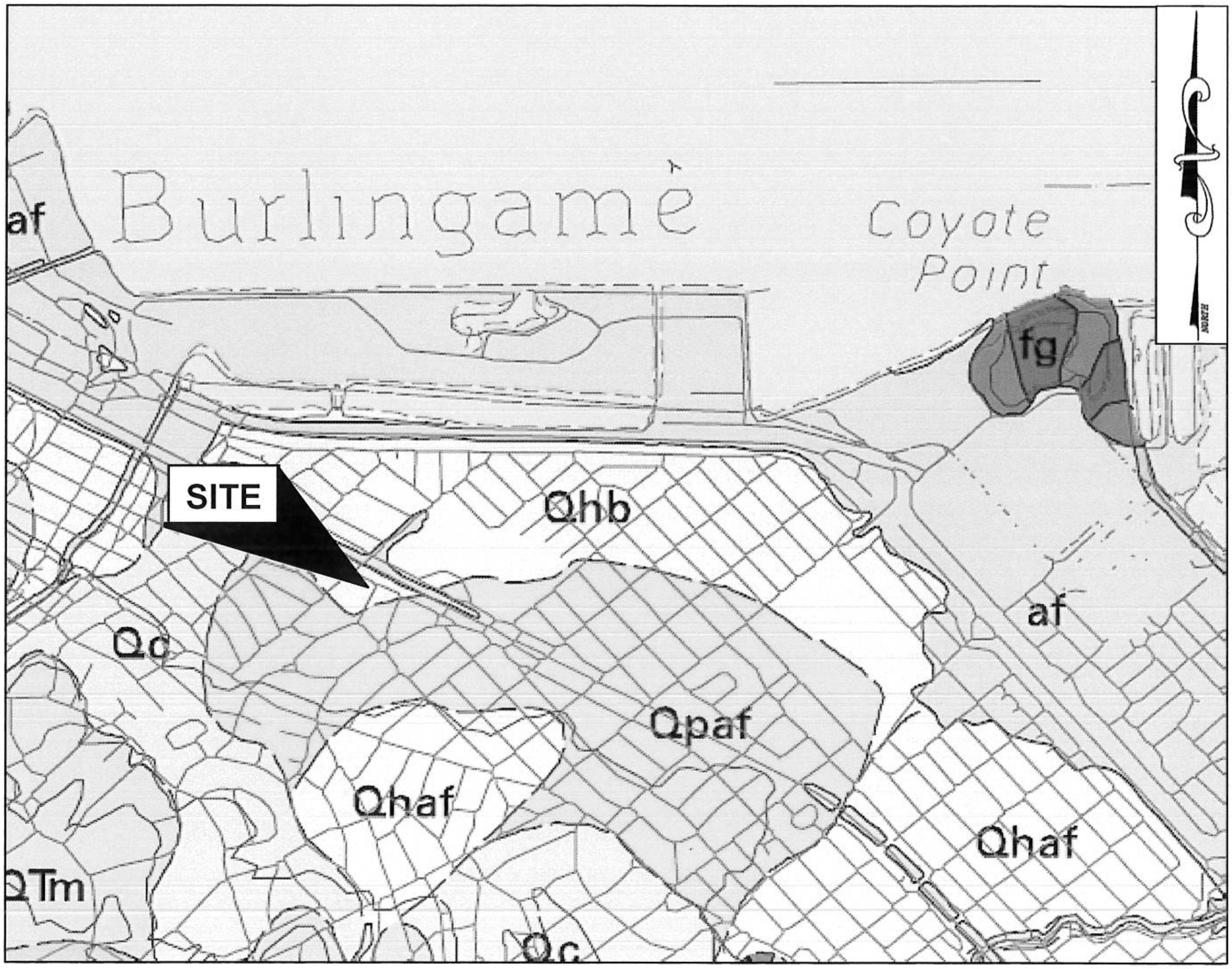
Approximate Scale: 1 inch = 20 feet.

Base is site plan prepared by IB+A Architecture, dated September 13, 2019.

SITE PLAN
DUFFY RESIDENTIAL BUILDING
BURLINGAME, CALIFORNIA

FIGURE 2
NOVEMBER 2019
PROJECT NO. 3910-2

ROMIG ENGINEERS



LEGEND

Qhaf	Alluvial fan and fluvial deposits
Qpaf	Alluvial fan and fluvial deposits
Qhb	Basin deposits
QTm	Merced Formation
af	Artificial fill
fg	Greenstone
fc	Chert

Geologic Contact - dashed where approximate, dotted where inferred.

Fault - dashed where approximate, dotted where inferred.

?

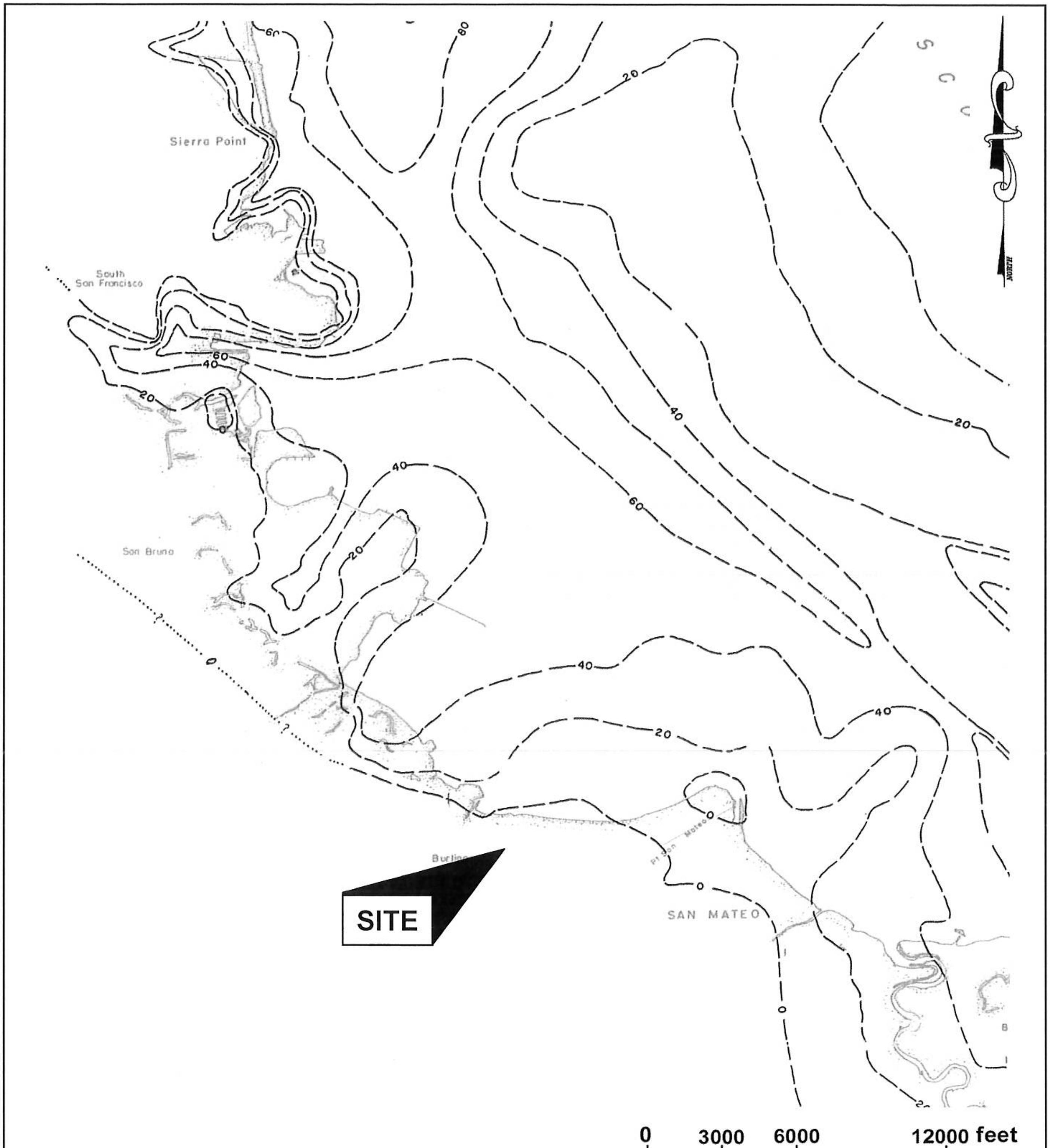
0 1000 2000 4000 feet

Base is USGS Map of San Mateo County, Brabb, Graymer, and Jones, dated 1998.

VICINITY GEOLOGIC MAP
DUFFY RESIDENTIAL BUILDING
BURLINGAME, CALIFORNIA

FIGURE 3
NOVEMBER 2019
PROJECT NO. 3910-2

ROMIG ENGINEERS

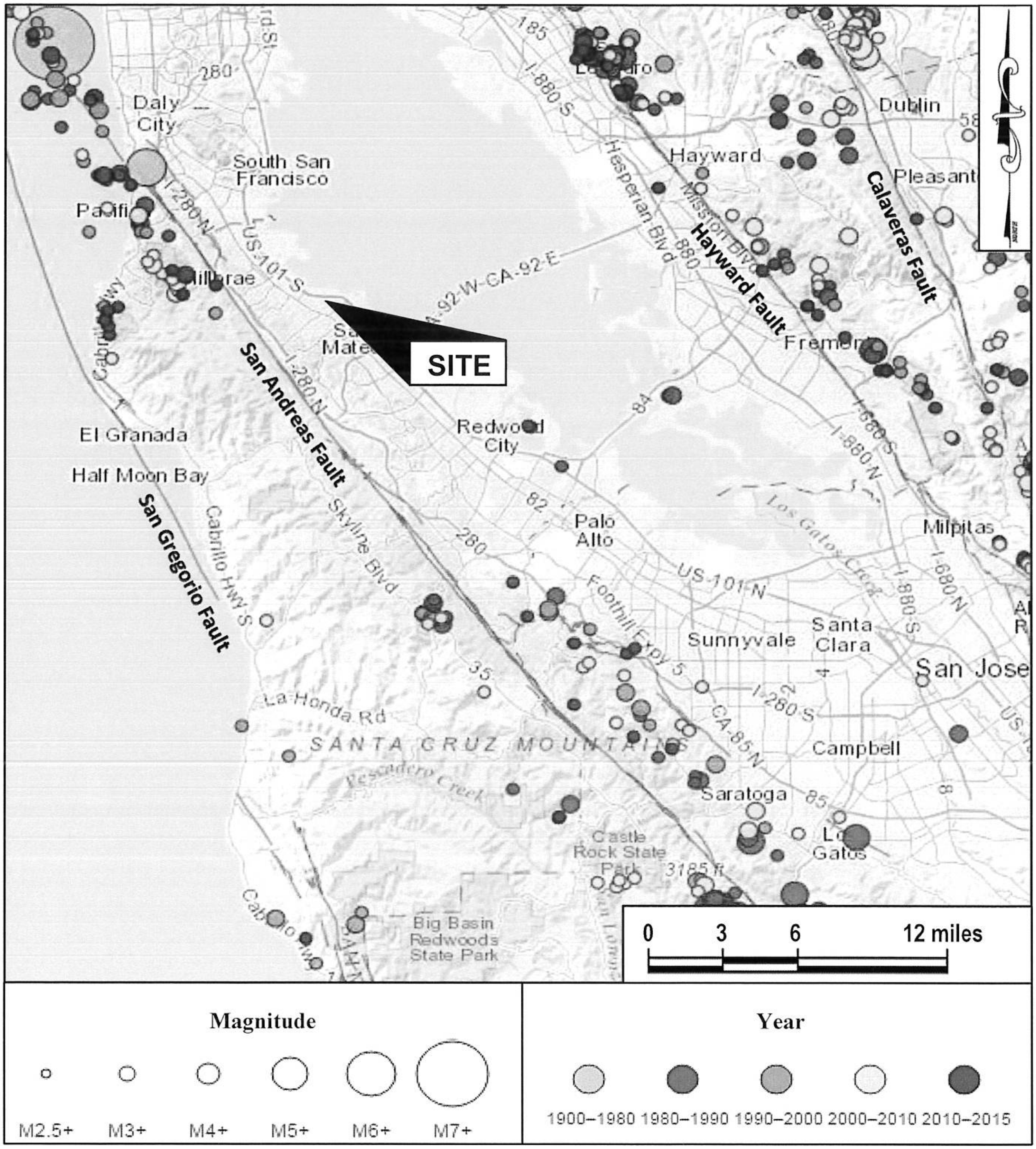


Scale: 1 inch = 6000 feet
Base is Geologic and Engineering Aspects of San Francisco Bay Fill, Special Report 97, Map of "Thickness of Younger Bay Mud", (CDMG, 1966).

CONTOUR MAP OF YOUNG BAY MUD THICKNESS
DUFFY RESIDENTIAL BUILDING
BURLINGAME, CALIFORNIA

FIGURE 4
NOVEMBER 2019
PROJECT NO. 3910-2

ROMIG ENGINEERS



Earthquakes with M5+ from 1900 to 1980, M2.5+ from 1980 to January 2015. Faults with activity in last 15,000 years. Based on data sources from Northern California Earthquake Data Center and USGS Quaternary Fault and Fold Database, accessed May 2015.

REGIONAL FAULT AND SEISMICITY MAP
DUFFY RESIDENTIAL BUILDING
BURLINGAME, CALIFORNIA

FIGURE 5
NOVEMBER 2019
PROJECT NO. 3910-2

ROMIG ENGINEERS

APPENDIX A

FIELD INVESTIGATION

The soils encountered during drilling were logged by our representative and samples were obtained at depths appropriate to the investigation. The samples were taken to our laboratory where they were examined and classified in accordance with the Unified Soil Classification System. The logs of our borings, as well as a summary of the soil classification system (Figure A-1) used on the boring logs, are attached.

Several tests were performed in the field during drilling. The standard penetration test resistance was determined by dropping a 140-pound hammer through a 30-inch free fall, and recording the blows required to drive the 2-inch (outside diameter) sampler 18 inches. The standard penetration test (SPT) resistance is the number of blows required to drive the sampler the last 12 inches, and is recorded on the borings log at the appropriate depth. Soil samples were also collected using a 2.5-inch O.D. drive sampler. The blow counts shown on the logs for the 2.5-inch sampler do not represent SPT values and have not been corrected in any way.

The locations of the borings were established by pacing, using the site plan provided to us. The locations should be considered accurate only to the degree implied by the method used.

The boring logs and related information depict our interpretation of subsurface conditions only at the specific location and time indicated. Subsurface conditions and ground water levels at other locations may differ from conditions at the locations where sampling was conducted. The passage of time may also result in changes in the subsurface conditions.

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USCS SOIL CLASSIFICATION

PRIMARY DIVISIONS			SOIL TYPE	SECONDARY DIVISIONS
COARSE GRAINED SOILS (< 50 % Fines)	GRAVEL	CLEAN GRAVEL (< 5% Fines)	GW	Well graded gravel, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravel or gravel-sand mixtures, little or no fines.
		GRAVEL with FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SAND	CLEAN SAND (< 5% Fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SAND WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS (> 50 % Fines)	SILT AND CLAY Liquid limit < 50%		ML	Inorganic silts and very fine sands, with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, lean clays.
			OL	Organic silts and organic clays of low plasticity.
	SILT AND CLAY Liquid limit > 50%		MH	Inorganic silt, micaceous or diatomaceous fine sandy or silty soil.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.
BEDROCK			BR	Weathered bedrock.

RELATIVE DENSITY

SAND & GRAVEL	BLOWS/FOOT*
VERY LOOSE	0 to 4
LOOSE	4 to 10
MEDIUM DENSE	10 to 30
DENSE	30 to 50
VERY DENSE	OVER 50

CONSISTENCY

SILT & CLAY	STRENGTH^	BLOWS/FOOT*
VERY SOFT	0 to 0.25	0 to 2
SOFT	0.25 to 0.5	2 to 4
FIRM	0.5 to 1	4 to 8
STIFF	1 to 2	8 to 16
VERY STIFF	2 to 4	16 to 32
HARD	OVER 4	OVER 32

GRAIN SIZES

BOULDERS	COBBLES	GRAVEL		SAND			SILT & CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE	

12 "   3"   0.75"   4   10   40   200

SIEVE OPENINGS   U.S. STANDARD SERIES SIEVE

Classification is based on the Unified Soil Classification System; fines refer to soil passing a No. 200 sieve.

* Standard Penetration Test (SPT) resistance, using a 140 pound hammer falling 30 inches on a 2 inch O.D. split spoon sampler; blow counts not corrected for larger diameter samplers.

^ Unconfined Compressive strength in tons/sq. ft. as estimated by SPT resistance, field and laboratory tests, and/or visual observation.

KEY TO SAMPLERS

Modified California Sampler (3-inch O.D.)
Mid-size Sampler (2.5-inch O.D.)
Standard Penetration Test Sampler (2-inch O.D.)

KEY TO EXPLORATORY BORING LOGS
DUFFY RESIDENTIAL BUILDING
BURLINGAME, CALIFORNIA

FIGURE A-1
NOVEMBER 2019
PROJECT NO. 3910-2

ROMIG ENGINEERS

DRILL TYPE: Mobile Drill B-40 with 8" Hollow Stem Auger **LOGGED BY:** RL

DEPTH TO GROUND WATER: 12 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 8/16/19

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Brown, Sandy Lean Clay, moist, fine to coarse grained sand, low plasticity, black mottling (disturbed surface soil).	Very Stiff	CL		0					
■ Liquid Limit = 25, Plasticity Index = 9.					■	29	16		3.3
Brown, Clayey Sand, moist, fine to coarse grained sand, low plasticity fines, trace fine gravel.	Dense to Very Dense	SC							
● 42% Passing No. 200 Sieve.					●		18		
						52	16		
				5					
						44	17		
Increased sand content.									
				10		51	19		
▼ Ground water measured at 12 feet after drilling.				▼					
				15		43	19		
							20		
				20		73	15		
Continued on Next Page									

EXPLORATORY BORING LOG EB-1
DUFFY RESIDENTIAL BUILDING
BURLINGAME, CALIFORNIA

NOVEMBER 2019
PROJECT NO. 3910-2

ROMIG ENGINEERS

DRILL TYPE: Mobile Drill B-40 with 8" Hollow Stem Auger **LOGGED BY:** RL

DEPTH TO GROUND WATER: 12 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 8/16/19

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Brown, Clayey Sand, moist, fine to coarse grained sand, fine sub-angular gravel, low plasticity clayey fines.	Very Dense	SC		20					
						50/6"	13		
				25					
● 32% Passing No. 200 Sieve.					●	77	15		
				30		47	21		
				35		61	18		
				40		73	19		

Continued on Next Page

DRILL TYPE: Mobile Drill B-40 with 8" Hollow Stem Auger **LOGGED BY:** RL

DEPTH TO GROUND WATER: 12 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 8/16/19

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Brown, Clayey Sand, moist, fine to coarse grained sand, fine sub-angular gravel, low plasticity clayey fines.	Dense to Very Dense	SC		40					
● 21% Passing No. 200 Sieve.				45	●	50	21		
				50		50/6"	18		
Bottom of Boring at 50 feet.									
				55					
Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual. *Measured using Torvane and Pocket Penetrometer devices.				60					

DRILL TYPE: Mobile Drill B-40 with 8" Hollow Stem Auger **LOGGED BY:** RL

DEPTH TO GROUND WATER: 12 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 8/16/19

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
4-inches of asphaltic concrete over 5-inches of aggragate base.		AC		0					
Fill: Brown, Sandy Lean Clay, moist, fine to coarse grained sand, low plasticity.	Hard	CL				54	11		>4.5
Brown, Clayey Sand, slightly moist to moist, fine to coarse sand, fine to medium grained sub-angular gravel.	Very Dense	SC				64	11		
				5		63	9		
Brown, Sandy Lean Clay, moist, fine to coarse grained sand, low plasticity. ● 55% Passing No. 200 Sieve.	Very Stiff	CL		10		25	19		2.5
▼ Ground water measured at 12 feet after drilling.				▼					
Brown, Clayey Sand, slightly moist to moist, fine to coarse sand, fine to medium grained sub-angular gravel. ● 26% Passing No. 200 Sieve.	Medium Dense to Dense	SC		15		48	18 16		
				20		20	19		
Continued on Next Page									

EXPLORATORY BORING LOG EB-1
DUFFY RESIDENTIAL BUILDING
BURLINGAME, CALIFORNIA

BORING EB-2

NOVEMBER 2019
PROJECT NO. 3910-2

ROMIG ENGINEERS

DRILL TYPE: Mobile Drill B-40 with 8" Hollow Stem Auger **LOGGED BY:** RL

DEPTH TO GROUND WATER: 12 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 8/16/19

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Brown, Clayey Sand, slightly moist to moist, fine to coarse sand, fine to medium grained sub-angular gravel.	Very Dense	SC		20					
				25		54	21		
Bottom of Boring at 25 feet.									
				30					
				35					
Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual. *Measured using Torvane and Pocket Penetrometer devices.				40					

EXPLORATORY BORING LOG EB-1
DUFFY RESIDENTIAL BUILDING
BURLINGAME, CALIFORNIA

BORING EB-2

NOVEMBER 2019
PROJECT NO. 3910-2

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APPENDIX B

LABORATORY TESTS

Samples from the subsurface exploration were selected for tests to help evaluate the physical and engineering properties of the soils that were encountered. The tests that were performed are briefly described below.

The natural moisture content was determined in accordance with ASTM D 2216 on nearly all of the samples recovered from the borings. This test determines the moisture content, representative of field conditions, at the time the samples were collected. The results are presented on the boring logs, at the appropriate sample depths.

The Atterberg Limits were determined on one sample of soil in accordance with ASTM D4318. The Atterberg limits are the moisture content within which the soil is workable or plastic. The results of this test are presented in Figure B-1 and on the log of Boring EB-1 at the appropriate sample depth.

The amount of silt and clay-sized material present was determined on five samples of soil in accordance with ASTM D422. The results of these results are presented on the boring logs at the appropriate sample depths.

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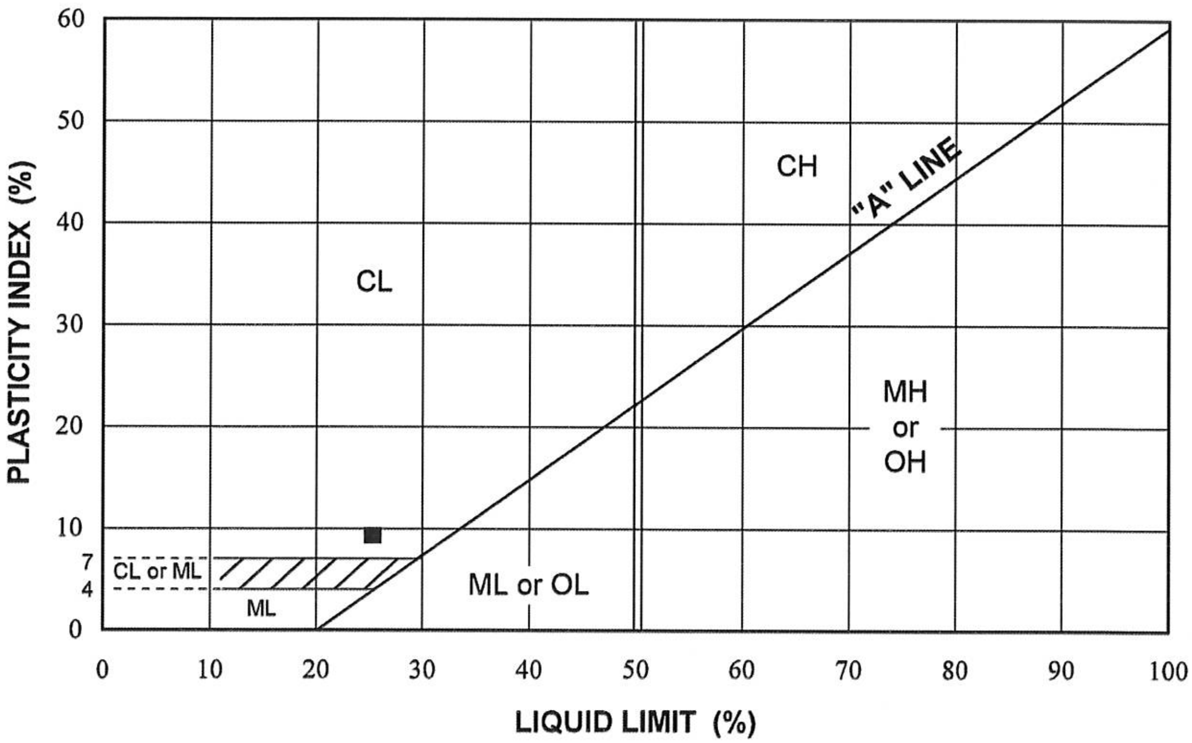

Chart Symbol	Boring Number	Sample Depth (feet)	Water Content (percent)	Liquid Limit (percent)	Plasticity Index (percent)	Liquidity Index (percent)	Passing No. 200 Sieve (percent)	USCS Soil Classification
■	EB-1	1-2.5	16	25	9	0		CL

PLASTICITY CHART
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FIGURE B-1
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