Geotechnical
Environmental
Hydrogeology
Material Testing
Construction Inspection

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Project No. 17-6513

Mr. Binh Tran
ZBT group
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Subject: Preliminary Geotechnical Investigation Report Including Percolation Testing, Proposed 5-Story Hotel, 13650 South Harbor Blvd, Garden Grove, California

Binh,

In accordance with your request and authorization, TGR Geotechnical, Inc. (TGR) has performed a preliminary geotechnical investigation for the proposed hotel at the subject site. The site is currently occupied by a former automotive building with associated repair docks and parking areas. It is our understanding that the existing structures will be demolished and the proposed development will consist of a 5-story hotel with a pool, planter areas, and associated drive lanes and parking. This report presents the findings of our geotechnical investigation, including site seismicity and provides geotechnical design recommendations for the proposed development.

Based on our investigation the proposed development is feasible from a geotechnical viewpoint provided the recommendations presented in this report are implemented during design and construction.

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

Respectfully submitted,

TGR GEOTECHNICAL, INC.

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Attachments:

Plate 1 - Boring Location Map
Figure 1 - Site Location Map
Figure 2 - Regional Geology Map
Figure 3 - Historic High Groundwater Map
Figure 4 - Regional Fault Map
Figure 5 - Seismic Hazard Zone Map

Appendix A – References
Appendix B – Log of Borings
Appendix C – Laboratory Testing Procedures and Results
Appendix D – Liquefaction Analysis

INTRODUCTION

Site Descriptions and Proposed Project Development

The subject site is located at 13650 South Harbor Boulevard, within the City of Garden Grove, California (Site Location Map, Figure 1). The subject site is currently occupied by a former automotive center with parking, repair docks, and retail building. It is our understanding that the existing structures will be demolished and the proposed development will consist of a 5-story hotel with a pool, associated drive aisles, parking lot, and planter space.

Scope of Work

The scope of work for this geotechnical investigation included the following:

- Site reconnaissance.
- Excavation of four (4) hollow stem auger borings to the approximate depths ranging from 26.5 to 51.5 feet below the existing grade.
- Percolation testing of one location to a depth of 5 feet utilizing a hollow stem auger drill rig.
- Laboratory testing of selected samples for In-situ moisture and density, maximum dry density and optimum moisture content, shear, sieve, consolidation, and corrosion.
- Engineering evaluation for liquefaction and seismic settlement, foundation and pavement design.
- Preparing this report summarizing current subsurface soil conditions, findings, and presenting our recommendations for the proposed development.

Field Investigation

Field exploration was performed on June 30, 2017 by a representative from our firm who logged the borings and obtained representative samples, which were subsequently transported to the laboratory for further review and testing. The approximate locations of the borings are indicated on the enclosed Boring Location Map (Plate 1).

The subsurface conditions were explored by drilling, sampling, and logging four (4) borings with a truck mounted hollow stem drill rig. Boring B-2 was advanced to an approximate depth of 51.5 feet below existing grade. Borings B-1, B-3, and B-4 were advanced to approximately 26.5 feet below existing grade. One additional boring, P-1, was advanced to approximately 5 feet below existing grade and utilized for percolation testing of the near-surface soils. Subsequent to drilling, all borings were backfilled with cuttings and surface repaired with cold patch asphalt. The logs of borings together with an explanation of symbols used are given in Appendix B.

The drill rig was equipped with a sampling apparatus to allow for recovery of driven modified California Ring Sampler (CRS), 3-inch outside diameter, and 2.42-inch inside diameter samples. Driven samples and bulk samples of the earth materials encountered at selected intervals were recovered from the borings. From the borings, all samples were secured in moisture-resistant

bags as soon as taken to minimize the loss of field moisture while being transported to the laboratory. The locations and depths of the soil samples recovered are indicated on the logs in Appendix B.

Laboratory Testing

Laboratory tests were performed on representative samples to verify the field classification of the recovered samples and to evaluate the geotechnical properties of the subsurface soils. The following tests were performed:

- In-situ moisture content (ASTM D2216) and dry density (ASTM D7263);
- Maximum Dry Density and Optimum Moisture Content (ASTM D1557);
- Corrosion series:
    1. Soluble Sulfate (CAL.417A);
    2. Soluble Chlorides (CAL.422);
    3. Minimum Resistivity (CAL.643); and
    4. pH
- Sieve Analysis (ASTM D6913);
- Consolidation potential (ASTM D2435); and
- Direct shear strength (ASTM D3080).

Laboratory tests for geotechnical characteristics were performed in general accordance with the ASTM procedures. The results of the in-situ moisture content and density tests are shown on the boring log (Appendix B). The results of the laboratory tests are presented in Appendix C.

GEOTECHNICAL FINDINGS

Geology

Regional Geologic Setting

The subject property is located near the southern extent of the Los Angeles Basin. The Los Angeles Basin is considered part of the Transverse Ranges geomorphic province of California. The Transverse Ranges extends about 325 miles from Point Arguello and San Miguel Island on the west into Joshua Tree National Monument on the east where it merges with the Mojave and Colorado Desert provinces. The Transverse Ranges province has an irregular east-west shape and varies from about 40 to 60 miles wide.

The Los Angeles Basin is situated at the junction of two major convergent fault systems. The first group includes the northwest-trending high angle strike slip faults of the San Andreas system projecting from the northern terminus of the Peninsular Ranges province. Faults in this group include the Palos Verdes, Newport-Inglewood and Whittier-Elsinore fault zones. The second group includes the east-west trending low angle reverse or reverse-oblique faults bounding the south margin of the Transverse Range province. Faults in this group include the Malibu-Santa Monica, Hollywood, Raymond and Sierra Madre fault zones. The Los Angeles Basin is bounded by active faults on all sides and is underlain by alluvial sediments and buried thrust faults. The seismic hazard for the Los Angeles Basin and vicinity is high. Figure 2, Regional Geologic Map,

presents the location of the project site with respect to regional geology of the project site and vicinity.

Earth Units

At the subject site, the upper 15 feet of subsurface soils are generally slightly moist to moist, brown, silty sand to sand with varying amounts of sand and silt. At approximately 15 feet below ground surface, a thin layer of silt to silty sand occurs to approximately 20 feet. The material below this is generally fine to coarse sand with varying amounts of silt in a wet and loose condition to approximately 45 feet below grade. Below this the material becomes silty sand at 45 feet and silt at 50 feet to the maximum depth explored, approximately 51.5 feet below grade. A detailed description of soils encountered is included on the boring logs, Appendix B. The site soils are generally consistent with the young alluvial fan deposits described in the geologic map for the area (Morton and Miller, 2006).

Infiltration Testing

The infiltration testing was performed in general accordance with the test procedures outlined in the Orange County Model Water Quality Management Plan (WQMP), dated May 19, 2011.

The results of our testing are as follows:

P-1 5.0 inches per hour design infiltration rate

The design infiltration rate includes a safety factor of 2.5.

Groundwater

A review of the seismic hazard report for the Anaheim quadrangle indicates that the historic high groundwater at the subject site is approximately 8 feet below existing ground surface. The Historic High Groundwater Map is presented on Figure 3, attached. Groundwater was encountered at a depth of approximately 22 feet below ground surface at the time of our investigation. Seasonal and long-term fluctuations in the groundwater may occur as a result of variations in subsurface conditions, rainfall, run-off conditions and other factors. Therefore, variations from our observations may occur. Groundwater is not anticipated to negatively impact grading operations during construction.

Seismic Review

Faulting and Seismicity

The subject site, like the rest of Southern California, is located within a seismically active region as a result of being located near the active margin between the North American and Pacific tectonic plates. The principal source of seismic activity is movement along the northwest-trending regional faults such as the San Andreas, San Jacinto and Elsinore fault zones. These fault systems produce approximately 5 to 35 millimeters per year of slip between the plates.

By definition of the State Mining and Geology Board, an active fault is one which has had surface displacement within the Holocene Epoch (roughly the last 11,000 years). The State

Mining and Geology Board has defined a potentially active fault as any fault which has been active during the Quaternary Period (approximately the last 1,600,000 years). These definitions are used in delineating Earthquake Fault Zones as mandated by the Alquist-Priolo Geologic Hazard Zones Act of 1972 and as subsequently revised in 1994 (Hart, 1997) as the Alquist-Priolo Geologic Hazard Zoning Act and Earthquake Fault Zones.

The intent of the act is to require fault investigations on sites located within Special Studies Zones to preclude new construction of certain inhabited structures across the trace of active faults.

The subject site is not included within any Earthquake Fault Zones as created by the Alquist-Priolo Earthquake Fault Zoning Act (Hart, 1997). Our review of geologic literature pertaining to the site area indicates that there are no known active or potentially active faults located within or immediately adjacent to the subject property. The nearest known active fault is the El Modeno Fault located approximately 5.8 miles northeast of the subject site. The next nearest known active fault is the Peralta Hills Fault located approximately 6.6 miles northeast of the subject site. The Newport-Inglewood Fault is located approximately 6.3 miles south of the subject site. The Regional Fault Map (Figure 4) shows the subject site in relation to the faults in the area.

Secondary Seismic Hazards

Surface Fault Rupture and Ground Shaking

Since no known faults are located within the site, surface fault rupture is not anticipated. However, due to the close proximity of known active and potentially active faults, severe ground shaking should be expected during the life of the proposed structures.

Liquefaction

Liquefaction is a seismic phenomenon in which loose, saturated, fine-grained granular soils behave similarly to a fluid when subjected to high-intensity ground shaking. Liquefaction occurs when these ground conditions exist: 1) Shallow groundwater; 2) Low density, fine, clean sandy soils; and 3) High-intensity ground motion. Effects of liquefaction can include sand boils, settlement, and bearing capacity failures below foundations.

Based on our review of Seismic Hazard Zones in California the subject site is located within a mapped liquefaction zone (Figure 5). Due to the presence of loose sandy soils below the historic high water table, the potential for liquefaction at the subject site is high.

Groundwater was observed in the borings at an approximate depth of 22 feet below ground surface. Liquefaction analysis and seismic settlement calculations were performed based on a peak ground acceleration of 0.53g (PGA_M), moment magnitude of 6.6 and a historic high groundwater of 8 feet below existing grade. Results of the liquefaction study are presented in Appendix D. The subsurface soils have a potential for liquefaction from a depth of approximately 8 feet to 45 feet. Due to excessive liquefaction induced seismic settlement ground improvement utilizing deep soil cement mixing or stone columns is recommended to reduce the future liquefaction induced settlement.

Seismically Induced Settlement

Ground accelerations generated from a seismic event can produce settlements in sands or in granular earth materials both above and below the groundwater table. This phenomenon is often referred to as seismic settlement and is most common in relatively clean sands, although it can also occur in other soil materials. The liquefaction induced settlement is estimated to be 8.2 inches. The seismic settlement of dry sand is estimated to be negligible after remedial grading. Calculations for liquefaction induced settlement are presented in Appendix D. Ground improvement comprising of deep soil cement columns or stone columns is recommended to control the seismic settlement.

Lateral Spreading

Seismically induced lateral spreading involves primarily movement of earth materials due to earth shaking. Lateral spreading is demonstrated by near-vertical cracks with predominantly horizontal movement of the soil mass involved. The topography in the vicinity of the subject site is relatively flat with a lined channel. Therefore, the potential for lateral spreading at the subject site is considered very low.

DISCUSSIONS AND CONCLUSIONS

General

Based on our field exploration, laboratory testing, and geotechnical engineering analysis, the proposed improvements are considered suitable from a geotechnical viewpoint, provided that the recommendations contained in this report are incorporated into the design and construction phases of the project.

Conclusions

Based on our findings and analyses, the subject site is likely to be subjected to moderate to severe ground shaking due to the proximity of known active and potentially active faults. This may reasonably be expected during the life of the structure and should be designed accordingly.

The engineering evaluation performed concerning site preparation and the recommendations presented are based on information provided to us and obtained by us during our office and fieldwork. This report is prepared for the proposed 5-story hotel at the site. In the event that any significant changes are made to the proposed development, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the recommendations of this report are verified or modified in writing by TGR.

RECOMMENDATIONS

Seismic Design Parameters

When reviewing the 2016 California Building Code the following data should be incorporated into the design.

Latitude (degree)	33.7651
Longitude (degree)	-117.9196
Site Class	D
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.5
Mapped Spectral Acceleration at 0.2-sec Period, S_s	1.459 g
Mapped Spectral Acceleration at 1.0-sec Period, S_1	0.534 g
Spectral Acceleration at 0.2-sec Period Adjusted for Site Class, S_{MS}	1.459 g
Spectral Acceleration at 1.0-sec Period Adjusted for Site Class, S_{M1}	0.801 g
Design Spectral Acceleration at 0.2-sec Period, S_{DS}	0.973 g
Design Spectral Acceleration at 1.0-sec Period, S_{D1}	0.534 g

The structural consultant should review the above parameters and the 2016 California Building Code to evaluate the seismic design.

Conformance to the criteria presented in the above table for seismic design does not constitute any type of guarantee or assurance that significant structural damage or ground failure will not occur during a large earthquake event. The intent of the code is “life safety” and not to completely prevent damage of the structure, since such design may be economically prohibitive.

Foundation Design Recommendations

Presented below are preliminary foundation recommendations for the proposed structures. We have assumed that the proposed 5-story Hotel will have typical column loads (dead plus live) of up to 200 kips. These recommendations assume that the near surface soils are graded in accordance with the grading requirements presented in grading section of this report.

Shallow Foundation Supported on Deep Soil-Cement Mix Columns (5-story Hotel)

The proposed 5-story hotel buildings may be supported on shallow pad footings with grade beams in two perpendicular directions supported on deep soil-cement mix columns (DSM). The purpose of DSM is to limit the total and differential seismic settlement.

The DSM columns shall be designed to limit the total and differential seismic settlement to 1.5-inches and 0.75 inches over 30 feet, respectively. A higher settlement may be allowed if acceptable to the project structural engineer and approved by the geotechnical engineer. The soil-cement mix columns will be located in a grid pattern connecting the column location. The

spacing and depth of the columns shall be designed by the specialty contractor in coordination with the geotechnical engineer and approved by the geotechnical engineer of record. The overall area replacement ratio shall be at least 42 percent within a minimum soil mix column diameter of 3-feet and a minimum overlap of 6-inches. Soil cement columns will also be located under columns with the number of soil-cement columns depending on individual column loading. The column loads will be transmitted through pad footings and will be directly and entirely supported by the soil-cement columns. The soil-cement mix columns shall have a minimum 28-day compressive strength of 150 psi. A specialty contractor with a minimum experience of 5 years in DSM of similar nature shall prepare the DSM design and layout in coordination with the project structural and geotechnical engineer. The layout shall incorporate the recommendations presented in this report.

For pad and continuous foundations supported on deep soil cement mix columns, an allowable bearing pressure of 2800 pounds per square foot may be used in design.

All shallow foundations for 5-story structures shall be embedded a minimum of thirty (30) inches below the lowest adjacent grade and supported on DSM columns. The minimum recommended footing width is twenty-four (24) inches. A minimum reinforcement of two No. 5 steel bars top and two No. 5 steel bars bottom is required for continuous footings from a geotechnical viewpoint.

A one-third (1/3) increase on the aforementioned bearing pressure may be used in design for short-term wind or seismic loads.

The post construction total and differential static settlements between adjacent footings supported on compacted fill are not anticipated to exceed 0.6- inch and 0.5-inch over 30 feet, respectively. Anticipated (design) seismic settlement is presented earlier.

Resistance to lateral loads can be assumed to be provided by passive earth pressure and by friction acting on structural components in permanent contact with the subgrade soils.

Lateral resistance on the sides of footings may be computed using a passive pressure of 275 pounds per square foot per foot of embedment, subject to a maximum of 2750 pounds per square foot. An allowable friction coefficient of 0.40 may be assumed with dead load forces between concrete and the supporting soils.

<u>Mat Foundation Supported on Stone Columns (5-story Hotel)</u>

The proposed structure may be supported on mat foundation with stone columns. The stone columns shall be designed by a specialty contractor in coordination with the geotechnical engineer for the acceptable total and differential static and seismic settlement for the mat foundation. The stone columns shall have a maximum spacing of 8 foot on center with a minimum diameter of 3-foot. The installation shall be performed by vibro-replacement method with bottom feed. The stones shall consist of hard, durable, clean, crushed rock and shall comply with the following gradation requirements:

Sieve Opening	Percent Passing
1.5 inch	100
1 inch	90 - 100
½ inch	5 - 60
N0 4	0 - 5

The specialty ground improvement contractor shall ensure that the proposed stone column meet the intent of the design (seismic total and differential settlement of less than 1.5 inch and 0.75 inch over 30 feet, respectively) and a mat foundation bearing capacity of 1500 psf with a total and differential settlement of 1.5-inch and 0.75-inch over 30 feet. The ground improvement contractor should also evaluate any impact of vibration during stone column installation on adjoining properties and structures.

Mat foundation shall be a minimum of 14-inches thick with a minimum embedment of 12-inches supported on a minimum two (2) feet of crushed aggregate base compacted to a minimum 95 percent relative compaction. If pumping conditions are observed, the base may be underlain by Tensar BX1200 (or equivalent) with an additional 12-inch of ¾-inch crushed rock.

The modulus of subgrade reaction may be taken as 150 pci (K_1) for a one (1) square foot mat founded on improved soils. This value should be reduced for change in size per the following formula.

$$K = K_1 \left(\frac{B+1}{2B}\right)^2$$

Where B = Width of Mat;
K = Coefficient of Subgrade Reaction of Footings Measuring B(ft) x B(ft).

Verification Testing of the ground improvement shall be performed by the geotechnical engineer or the contractor and approved by the geotechnical engineer. Post-improvement CPT shall be advanced to ensure that the intended settlement and bearing capacity for the project has been achieved by installation of the stone columns. If the intended design requirements are not achieved additional stone columns may be recommended.

Lateral resistance on the sides of footings may be computed using a passive pressure of 275 pounds per square foot per foot of embedment, subject to a maximum of 2750 pounds per square foot. An allowable friction coefficient of 0.40 may be assumed with dead load forces between concrete and the supporting soils.

<u>Shallow Spread Foundation for Canopies/ancillary structures (Light Weight)</u>:
An allowable vertical bearing value of 1,500 pounds per square foot (psf) may be used for design of pad and continuous footings founded on a minimum 3 feet of engineered fill. Pad (square) footings should be a minimum of 24-inches wide and 24-inches deep and continuous footings shall be a minimum of 18-inches wide and 24-inches deep. The above value may be

increased by one-third when considering short duration seismic or wind loads. The reinforcement should be designed by the project structural engineer. Significant seismic settlement (up to 8-inches) is anticipated during a seismic event. If the adverse impact of seismic settlement is unacceptable, ground improvement as stated earlier in the report may be considered to control seismic settlement.

The total and differential settlements due to footing loads are estimated to be on the order of 1-inch and ½-inch between columns, respectively.

Resistance to lateral loads including wind and seismic forces may be provided by frictional resistance between the bottom of concrete and the underlying fill soils and by passive pressure against the sides of the foundations. A coefficient of friction of 0.4 may be used between concrete foundation and underlying soil. The recommended passive pressure of the engineered fill may be taken as an equivalent fluid pressure of 275 pounds per cubic foot (2750 pcf maximum).

Cement Type and Corrosion

Concrete used should be designed in accordance with the provisions of ACI 318-11, Chapter 4 for Exposure Class S0.

Results of corrosion testing on the subsurface soils indicates that the soils are not considered corrosive to ferrous materials in contact with site soils. TGR does not practice corrosion engineering. If needed, a qualified specialist should review the site conditions and evaluate the corrosion potential of the site soil to the proposed improvements and to provide the appropriate corrosion mitigations for the project.

Building Slab-On-Grade

Building slab-on-grade for light loads should be a minimum of 6-inches thick and reinforced with a minimum of No. 5 reinforcing bar on 18-inch centers in two horizontally perpendicular directions. Reinforcing should be properly supported to ensure placement near the vertical midpoint of the slab. "Hooking" of the reinforcement is not considered an acceptable method of positioning the steel. The subgrade material should be compacted to a minimum of 90 percent of the maximum laboratory dry density (ASTM 1557) to a minimum depth of three (3) feet below existing grade. Prior to placement of concrete, the subgrade soils should be well moistened to at least near optimum moisture content and verified by our field representative.

Per CALGreen Code, a 4-inch thick base of ½-inch or larger clean aggregate shall be provided with a vapor retarder in direct contact with concrete and a concrete mix design which will address bleeding, shrinkage and curling. Alternatively, concrete floor slabs should be underlain with a moisture vapor retarder consisting of a polyvinyl chloride membrane such as 10-mil Visqueen, or equivalent. All laps within the membrane should be overlapped by at least 6 inches, and at least 2 inches of clean sand should be placed over the membrane to promote uniform curing of the concrete. To reduce the potential for punctures, the membrane should be placed on a pad surface that has been graded smooth without any sharp protrusions. If a smooth surface cannot be achieved by grading, consideration should be given to placing a 1-inch thick leveling course of sand across the pad surface prior to placement of the membrane. To prevent warped, curled, or bubbled floor coverings, the moisture emission through the

concrete and the alkalinity/ pH, and relative humidity of the concrete should meet the flooring or adhesive manufacturer's recommendations and requirements. Concrete floors to receive resilient flooring should be prepared in accordance with ASTM F710-03. Unless a qualified specialist is retained for the flooring design, the above should be incorporated in the design and construction of concrete floors to receive resilient flooring.

The actual thickness and reinforcement of the slab shall be designed by the structural engineer and should include the anticipated loading condition.

Flatwork

Flatwork should be a minimum of 4-inches thick should be reinforced with a minimum of No. 3 reinforcing bar on 18-inch centers in two horizontally perpendicular directions. Reinforcing should be properly supported to ensure placement near the vertical midpoint of the slab. "Hooking" of the reinforcement is not considered an acceptable method of positioning the steel. The subgrade material should be compacted to a minimum of 90 percent of the maximum laboratory dry density (ASTM 1557) to a minimum depth of two (2) feet. Prior to placement of concrete, the subgrade soils should be well moistened to at least near optimum moisture content and verified by our field representative. The actual thickness and reinforcement of the slab shall be designed by the structural engineer and should include the anticipated loading condition. Some movement of the flatwork is anticipated with time.

Preliminary Pavement Design

The County of Orange/Caltrans method of design was utilized to develop the following asphalt pavement section. The section was developed based on an assumed "R-Value" of 25 for silty sand subgrade soils.

A traffic index of 4.5 and 6 was assumed for use in the evaluation of automobile parking stalls and driveways, respectively. The traffic indices are subject to approval by controlling authorities and shall be approved by the project civil engineer.

ASPHALT PAVEMENT SECTION				
Pavement Utilization	Traffic Index	Asphalt (Inch)	Aggregate Base (Inch)	Total (Inch)
Parking Stalls	4.5	3.0	6.0	9
Auto Driveways	6.0	4.0	8.0	12

Aggregate base material should consist of CAB/CMB complying with the specifications in Section 200.2.2 of the current "Standard Specifications for Public Works Construction" and should be compacted to at least ninety-five (95) percent of the maximum dry density (ASTM D1557). The surface of the aggregate base should exhibit a firm and unyielding condition just prior to the placement of asphalt concrete paving. The pavement subgrade should be

constructed in accordance with the recommendations presented in the grading section of this report.

The R-value and the associated pavement section should be confirmed at the completion of site grading and pavement section updated as necessary.

Lateral Earth Pressures

The following soil parameters may be used for the design of retaining walls up to 6 feet high with level backfill:

Conditions	Equivalent Fluid Pressure (psf/ft) On-Site Materials
Active (Level)	38
At Rest	56
Passive	275 (maximum 2750 psf)

- Any import backfill shall be granular non-expansive select fill, with minimum sand equivalent of 30, tested and approved by TGR prior to backfill.
- An allowable coefficient of friction between properly compacted on-site fill soil and concrete of 0.40 may be used with the dead-load forces.
- The passive pressure in the upper 6 inches of soil not confined by slabs or pavement should be neglected.

Retaining structures should be provided with a drainage system to prevent buildup of hydrostatic pressure behind the walls. Provisions should be made to collect and dispose of excess water away from the wall. Wall drainage may be provided by a perforated pipe encased in gravel or crushed rock and enclosed by geo-synthetic filter fabric. We do not recommend omitting the drains behind walls.

In addition to the above lateral forces due to retained earth, surcharge due to improvements, such as an adjacent structure, should be considered in the design of the retaining wall. A minimum vertical surcharge load of 300 psf should be used in design of walls due to adjacent traffic unless the traffic is kept at least 6 feet from the walls. Loads applied within a 1:1 projection from any surcharging structure on the stem of the wall shall be considered as lateral surcharge. For uniform lateral surcharge conditions applied to free-to-deflect walls and restrained walls, we recommend utilizing a minimum horizontal load equal to 33 percent and 50 percent of the vertical load, respectively, and should be applied uniformly over the entire height of the wall. This horizontal load should be applied below the 1:1 projection plane. To minimize the surcharge load from an adjacent footing, deepened footings may be considered.

Retaining wall footings should have a minimum embedment of 24 inches below the lowest adjacent grade unless deeper footings are needed for other reasons. The retaining walls footings shall be supported on a minimum 3 feet of compacted engineered-fill compacted to a minimum 90 percent relative compaction as per ASTM D1557.

Pool/Spa Recommendations

The following presents our general recommendations for the proposed future pool/spa at the subject site. These recommendations do not incorporate the impact of seismic settlement on the performance of the pool/spa. The site's underlying materials have been determined to be negligible in expansion potential. Provided below are general design, construction and inspection recommendations:

1. Free standing pool walls need to be designed for an equivalent fluid pressure of 63 lbs./cu.ft. When the pool is empty the walls should be designed as recommended above.

2. The geotechnical consultant should observe the bottom of the pool excavation as pool subgrade soils are exposed.

3. The pool and spa should conform to setback criteria pertaining to slopes as established in the 2016 California Building Code.

4. Where the pool is planned near any structure, appropriate surcharge loads need to be incorporated into design and construction.

5. Hydrostatic pressure relief valves may be incorporated into the pool and spa design.

6. All fittings and pipe joints, particularly fittings in the side of the pool or spa, should be properly sealed to prevent water from leaking into the adjacent soil materials.

7. An elastic expansion joint (waterproof sealant) should be installed to prevent water from seeping into the soil at all deck joints.

8. A reinforced grade beam should be placed around the skimmers to provide support and mitigate cracking around the skimmer face.

9. The design engineer and/or contractor should determine joint and saw cut locations for the pool deck. However, spacing should not exceed 8 feet.

10. It is imperative that the homeowner incorporate into the overall improvement scheme adequate provisions for surface drainage. Ponding water, ground saturation and flows over slope faces are all situations that must be avoided.

11. All deck slabs should be minimally reinforced with no.4 steel bar at 18 inch on center both ways. All slab reinforcement should be supported to ensure proper mid-slab positioning during placement of concrete.

12. Slabs should have a minimum thickness of four inches with the perimeter edge thickened. The thickened edge shall have minimum 18-inch embedment.

Site Development Recommendations

General

During earthwork construction, all site preparation and the general procedures of the contractor should be observed, and the fill selectively tested by a representative of TGR. If unusual or unexpected conditions are exposed in the field, they should be reviewed by this office and if warranted, modified and/or additional recommendations will be offered.

Demolition and Clearing

Within the proposed development and demolition area, all foundation, slab-on-grade, vaults, utility lines, surface vegetation, trash, demolition debris, asphaltic concrete, and portland cement concrete, if encountered, should be cleared and removed from the proposed site.

Depressions resulting from the removal of objects encountered as mentioned above should be backfilled with properly compacted engineered fill under the observation of the geotechnical consultant.

Grading

All grading should conform to the guidelines presented in the California Building Code (2016 edition), except where specifically superseded in the text of this report. Prior to grading, TGR's representative should be present at the pre-construction meeting to provide grading guidelines, if needed, and review any earthwork.

It is recommended that the entire building pad be over-excavated a minimum of 4 feet below grade and replaced with engineered fill placed at 90 percent relative compaction at near optimum moisture content. The lateral extent of removals beyond the building/structure pad limits should be eight (8) feet, where possible. A minimum of two (2) feet of engineered fill is recommended under flatwork and pavement. Site soils may be reused as engineered fill provided the recommendations presented in this report are implemented. Exposed bottoms should be scarified a minimum of 8-inches, moisture conditioned and compacted to a minimum 90 percent relative compaction.

The depth of over-excavation should be reviewed by the Geotechnical Consultant during the actual construction. Any subsurface obstruction buried structural elements, and unsuitable material encountered during grading, should be immediately brought to the attention of the Geotechnical Consultant for proper exposure, removal and processing, as recommended.

Fill Placement

Prior to any fill placement TGR should observe the exposed surface soils. The site soils may be re-used as engineered fill provided they are free of organic content and particle size greater than 4-inches. Fill shall be moisture-conditioned to near optimum and compacted to a minimum relative compaction of 90 percent in accordance with ASTM D1557. Any import soils shall be non-expansive and approved by TGR Geotechnical Inc.

Compaction

Prior to fill placement, the exposed surface should be scarified to a minimum depth of eight (8) inches, fill placed in six (6) inch thick loose lifts if cohesionless and eight (8) inch thick loose lifts if cohesive fill, moisture conditioned to near optimum for cohesionless soils or a minimum of two (2) percent over optimum moisture for cohesive soils, and compacted to a minimum relative compaction of ninety (90) percent in accordance with ASTM D 1557.

Trenching

All excavations should conform to CAL-OSHA and local safety codes.

Drainage

Positive site drainage should be maintained at all times. Water should be directed away from foundations and not allowed to pond and/or seep into the ground. Pad drainage should be directed towards the street or other approved area. Drainage shall comply with the 2016 CBC.

Utility Trench Backfill

All utility trench backfill in structural areas and beneath hardscape features should be brought to near-optimum moisture content and compacted to a minimum relative compaction of 90 percent of the laboratory standard. Flooding/jetting is not recommended.

Sand backfill, (unless trench excavation material), should not be allowed in parallel exterior trenches adjacent to and within an area extending below a 1:1 plane projected from the outside bottom edge of the footing. All trench excavations should minimally conform to CAL-OSHA and local safety codes. Soils generated from utility trench excavations may be used provided it is moisture conditioned and compacted to 90 percent minimum relative compaction.

Geotechnical Review of Plans

All grading, foundation, building, and structural plans should be reviewed and accepted by the geotechnical consultant prior to construction. Based on the review, additional or amended geotechnical recommendations may be provided. If significant time elapses since preparation of this report, the geotechnical consultant should verify the current site conditions, and provide any additional recommendations (if necessary) prior to construction.

Geotechnical Observation/Testing During Construction

The geotechnical consultant should perform observation and/or testing at the following stages:

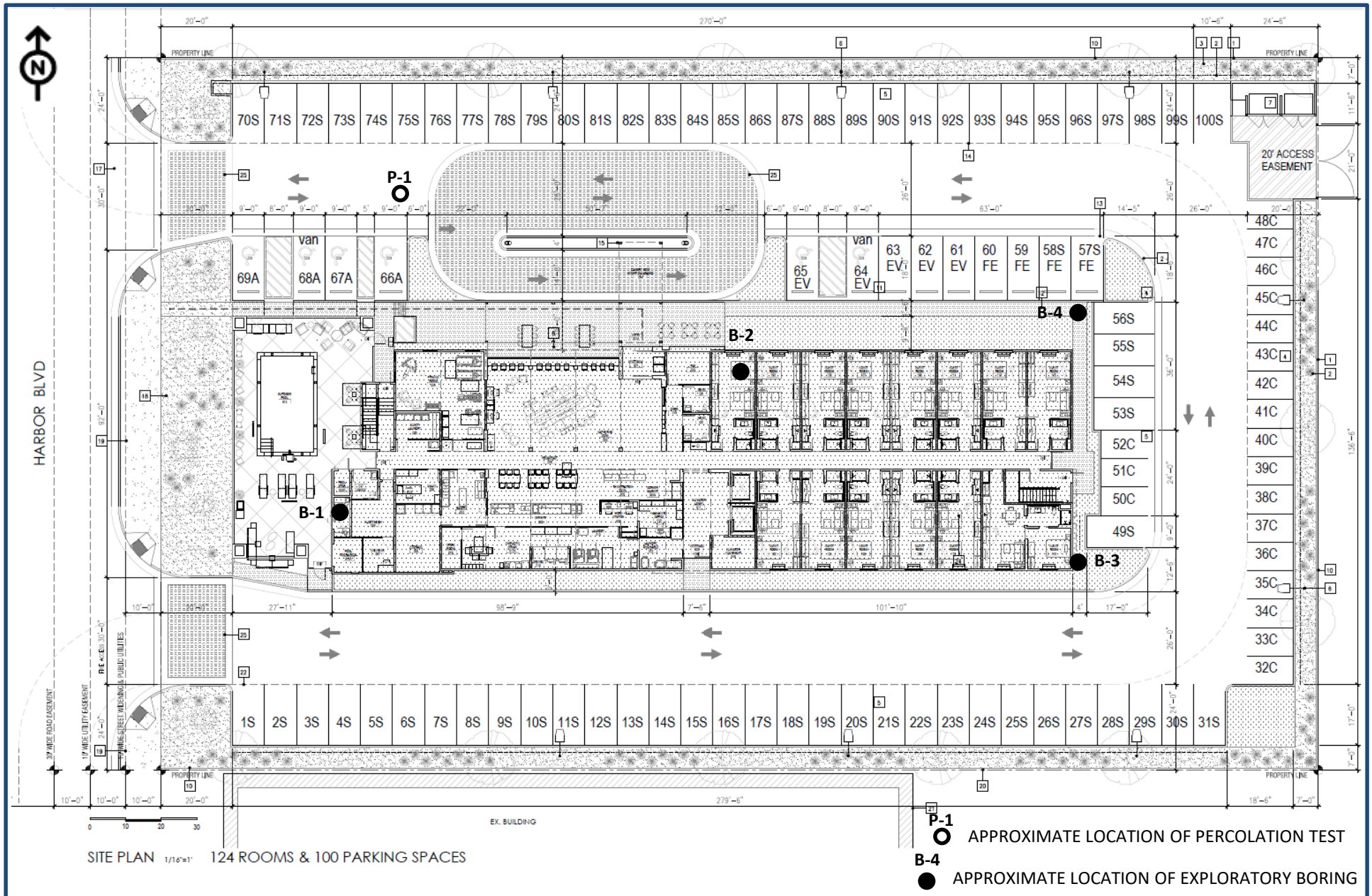
- During ground improvement
- During any grading and fill placement;
- After foundation excavation and prior to placing concrete;
- During pavement construction;
- Underground utility trench backfill;
- When any unusual soil conditions are encountered during any construction operation subsequent to issuance of this report.

Limitations

This report was prepared for a specific client and a specific project, based on the client's needs, directions and requirements at the time.

This report was necessarily based upon data obtained from a limited number of observances, site visits, soil and/or other samples, tests, analyses, histories of occurrences, spaced subsurface exploration and limited information on historical events and observations. Such information is necessarily incomplete. Variations can be experienced within small distances and under various climatic conditions. Changes in subsurface conditions can and do occur over time.

This report is not authorized for use by, and is not to be relied upon by any party except the client with whom TGR contracted for the work. Use or reliance on this report by any other party is that party's sole risk. Unauthorized use of or reliance on this report constitutes and agreement to defend and indemnify TGR from and against any liability which may arise as a result of such use or reliance, regardless of any fault, negligence, or strict liability of TGR.

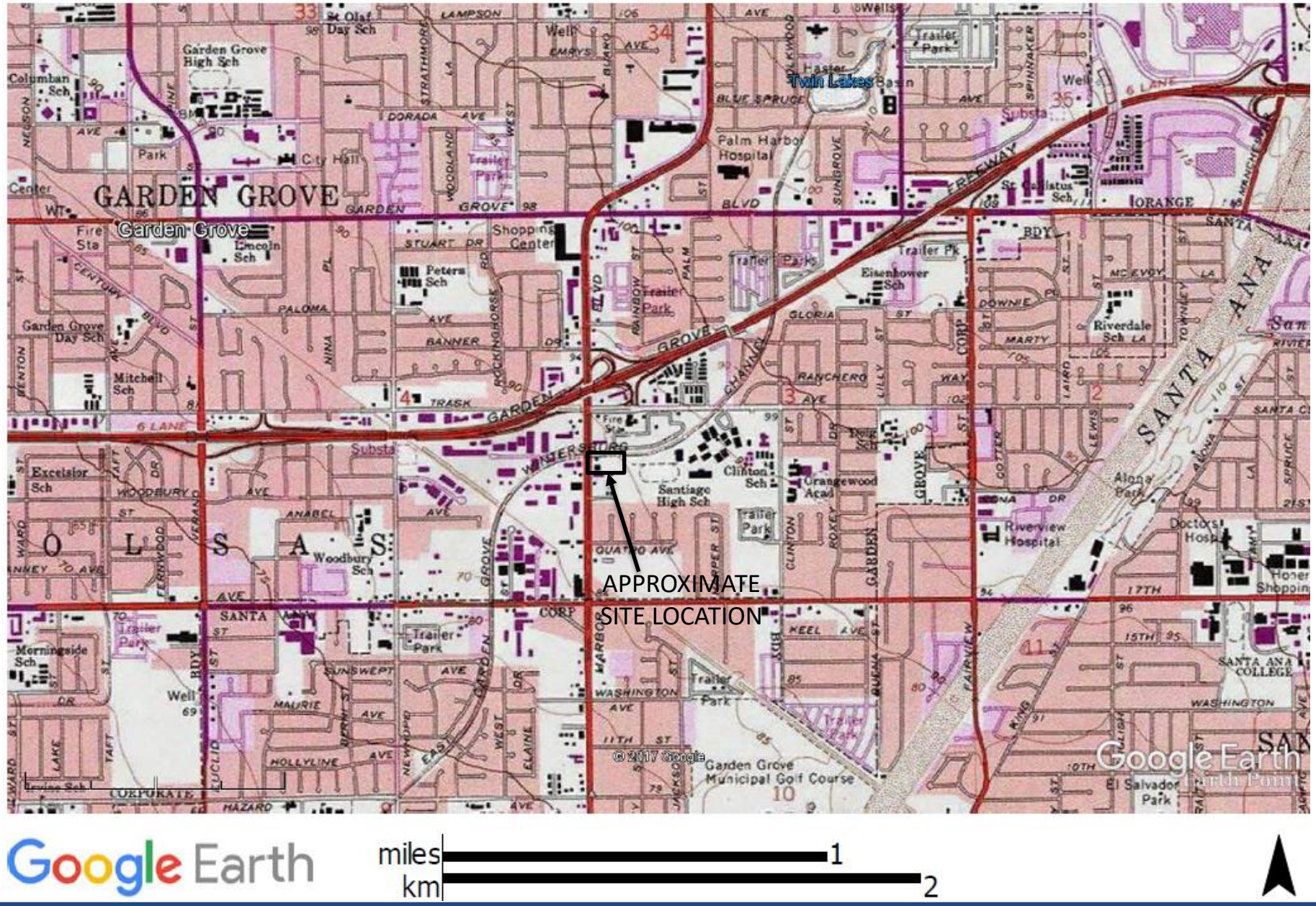


TGR Geotechnical Environmental Hydrogeology Material Testing Construction Inspection

BORING LOCATION MAP
13650 SOUTH HARBOR BOULEVARD
GARDEN GROVE, CALIFORNIA

PROJECT NO. 17-6513

PLATE 1



Google Earth

miles 1

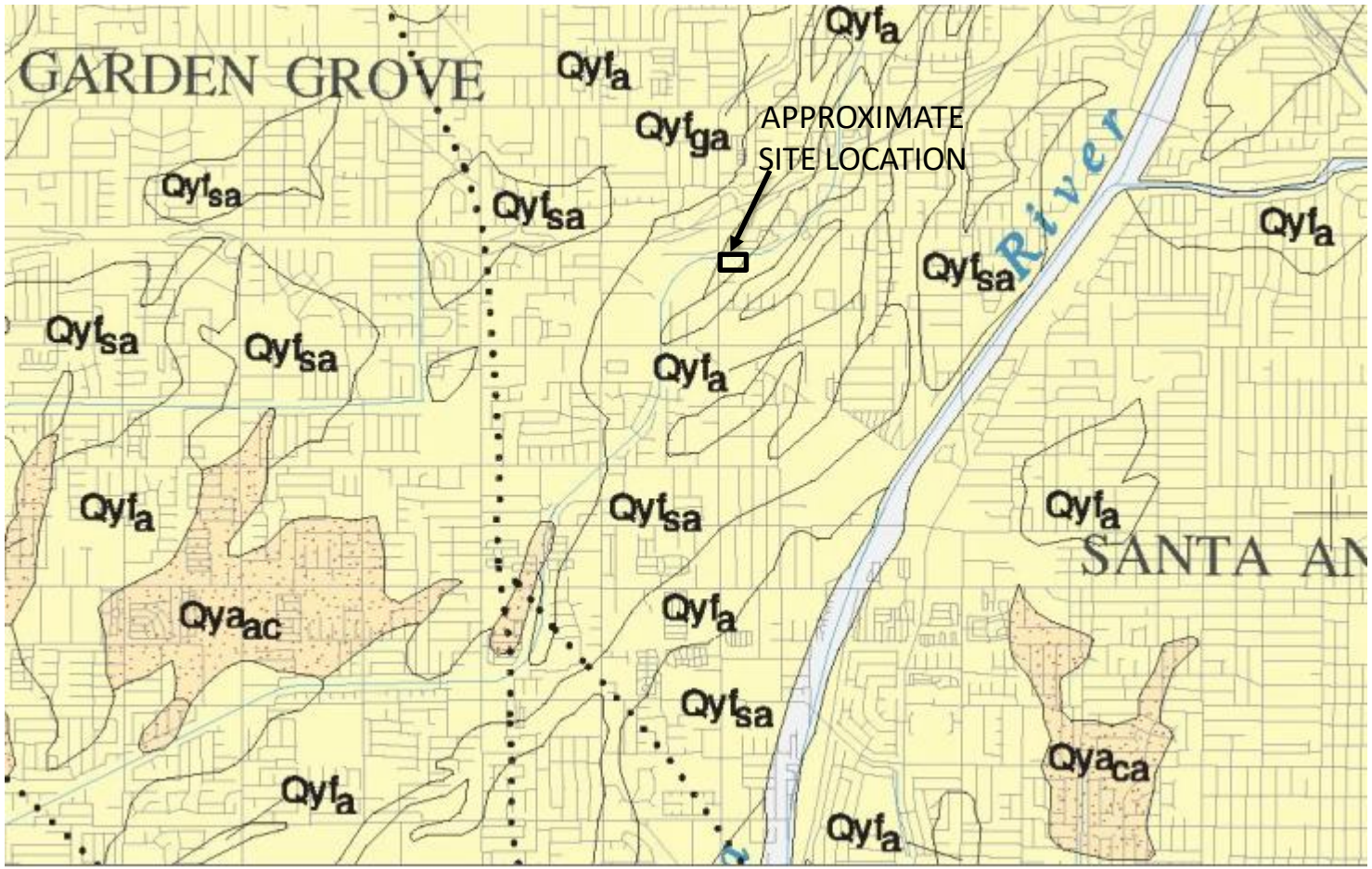
km 2

TGR Geotechnical Environmental Hydrogeology Material Testing Construction Inspection

SITE LOCATION MAP
13650 SOUTH HARBOR BOULEVARD
GARDEN GROVE, CALIFORNIA

PROJECT NO. 17-6513

FIGURE 1



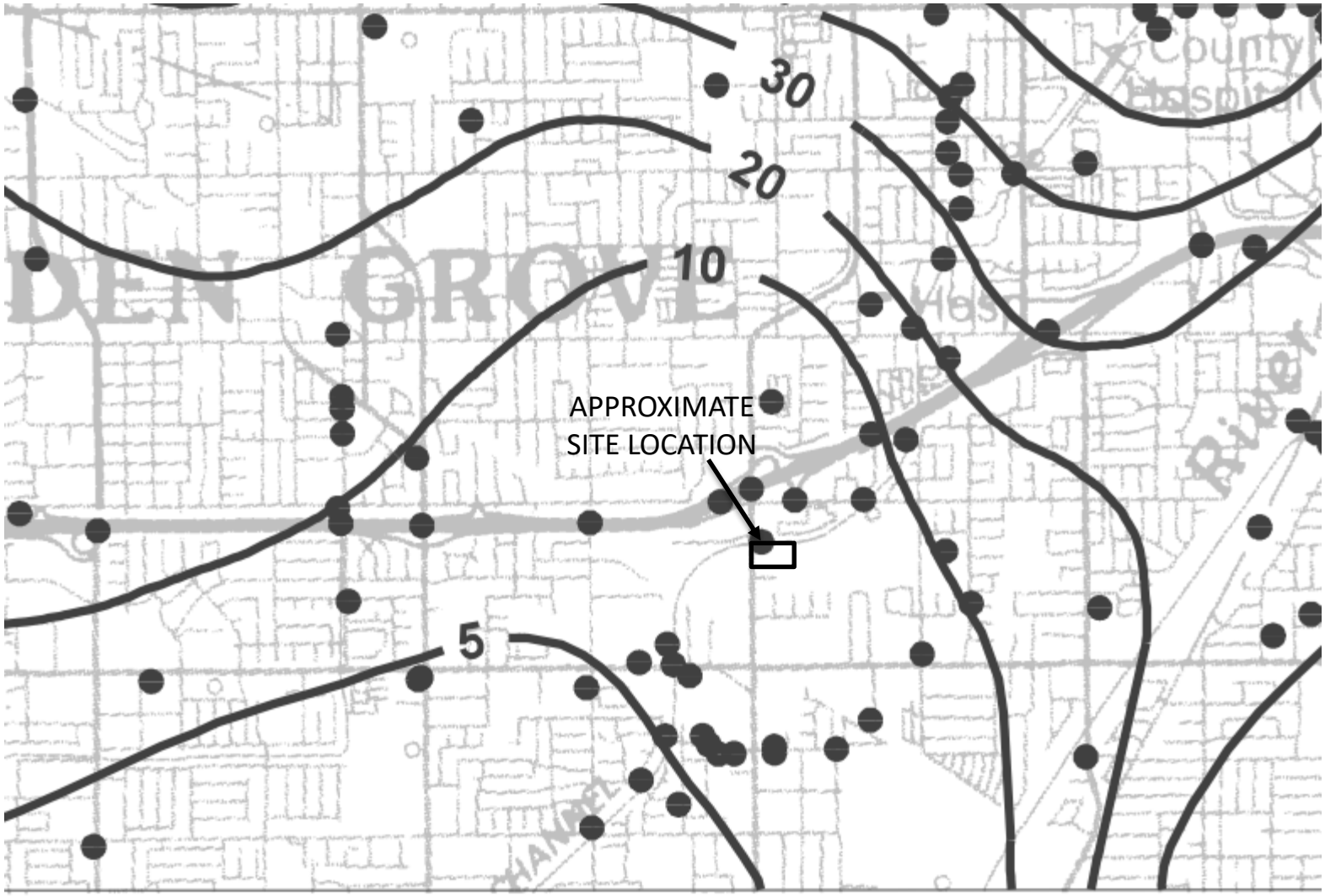
Modified From: Morton, D.M. and Miller, F.K., 2006, Geologic Map of the San Bernardino and Santa Ana 30'x60' Quadrangles, California, Version 1.0, scale 1:100,000

TGR Geotechnical Environmental Hydrogeology Material Testing Construction Inspection

REGIONAL GEOLOGY MAP
13650 SOUTH HARBOR BOULEVARD
GARDEN GROVE, CALIFORNIA

PROJECT NO. 17-6513

FIGURE 2



ONE MILE

Modified From: California Department of Conservation, Division of Mines and Geology, 1997, Seismic Hazard Zone Report for the Anaheim and Newport Beach Quadrangles, Orange County, California, Seismic Hazard Zone Report 03.

N

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HISTORIC HIGH GROUNDWATER MAP
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GARDEN GROVE, CALIFORNIA

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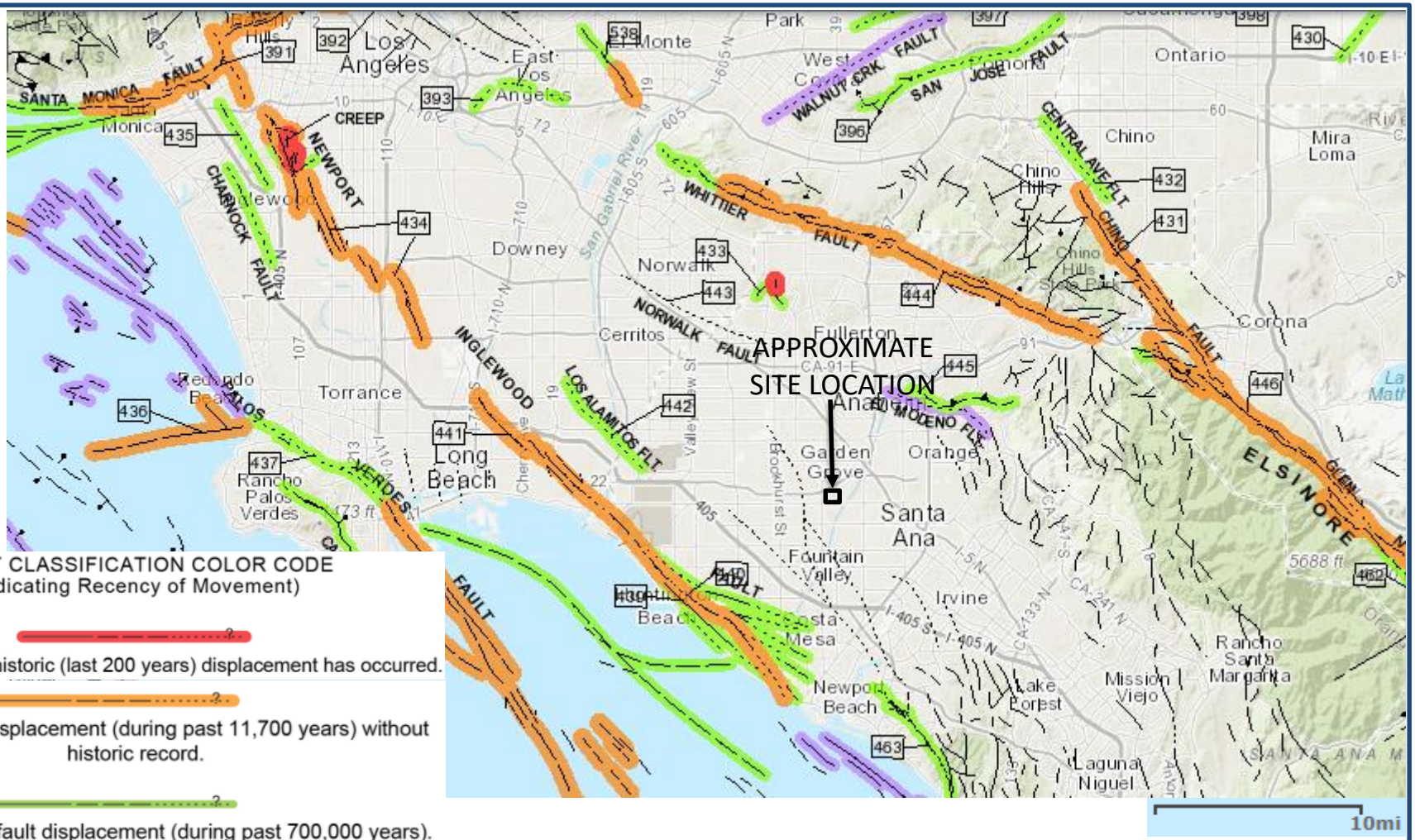
FIGURE 3



FAULT CLASSIFICATION COLOR CODE
(Indicating Recency of Movement)

Fault along which historic (last 200 years) displacement has occurred.

Holocene fault displacement (during past 11,700 years) without historic record.

Late Quaternary fault displacement (during past 700,000 years).

Quaternary fault (age undifferentiated).

Pre-Quaternary fault (older that 1.6 million years) or fault without recognized Quaternary displacement.

Modified From: California Geological Survey, 2010 Fault Activity Map of California, Geologic Data Map No. 6.

TGR Geotechnical Environmental Hydrogeology Material Testing Construction Inspection

REGIONAL FAULT MAP
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GARDEN GROVE, CALIFORNIA

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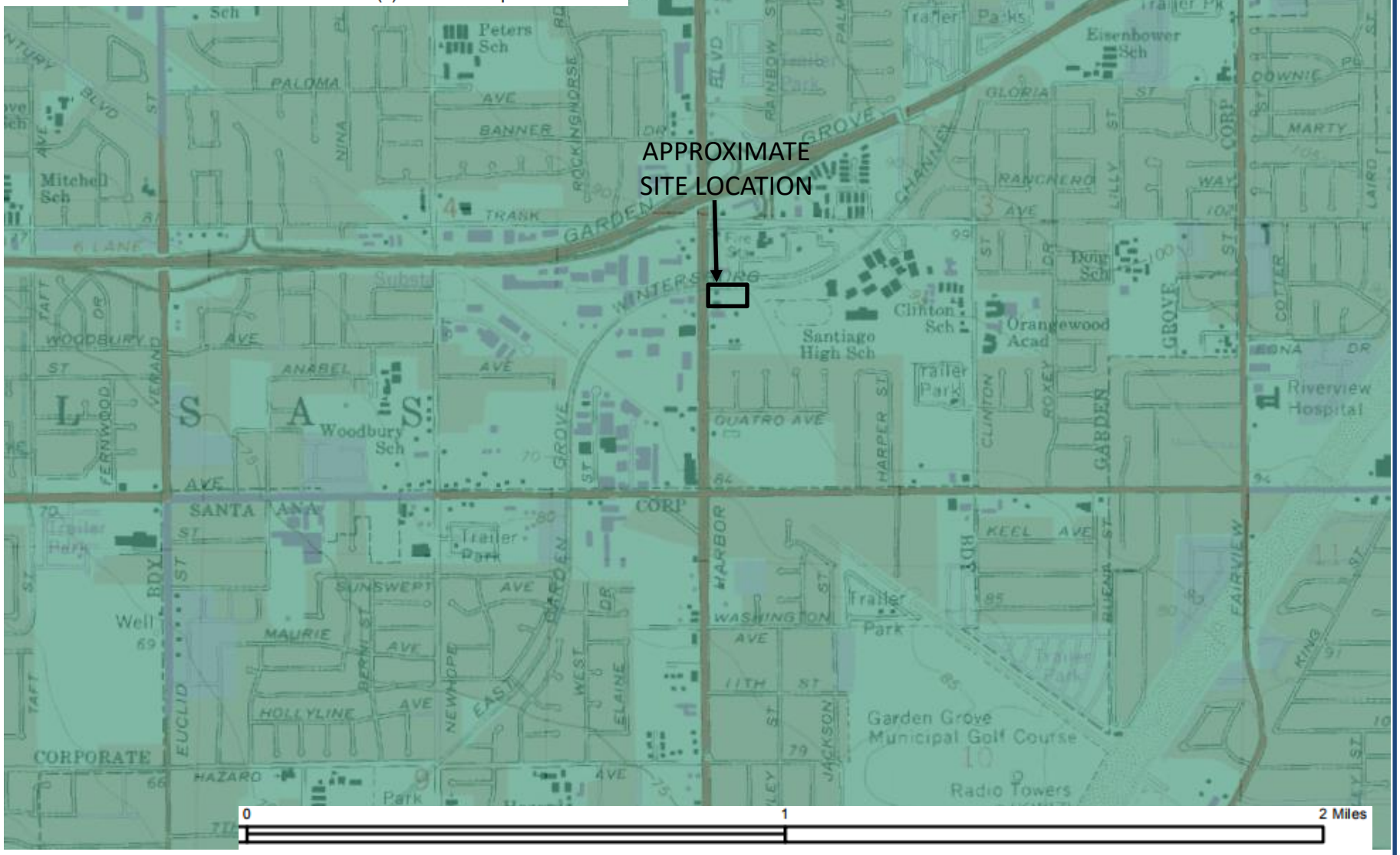
FIGURE 4

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Modified From: California Geological Survey, Earthquake Zones of Required Investigation, Anaheim Quadrangle, Official Map dated April 15, 1998, scale 1:24,000.



Geotechnical
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EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP
13650 SOUTH HARBOR BOULEVARD
GARDEN GROVE, CALIFORNIA

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FIGURE 5