APPENDIX E:

PRELIMINARY GEOTECHNICAL INVESTIGATION

REPORT OF GEOTECHNICAL INVESTIGATION
RIVERSIDE COMMUNITY HOSPITAL
RIVERSIDE, CALIFORNIA

Prepared for

HCA DESIGN AND CONSTRUCTION
One Park Plaza, Building II-E
Nashville, Tennessee 37203

Prepared by

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Project No. SD809
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HCA Design and Construction
One Park Plaza, Building II-E
Nashville, Tennessee 37203

Attention: Mr. Brian Seely

SUBJECT: REPORT OF GEOTECHNICAL INVESTIGATION
HCA Riverside Community Hospital
Riverside, California

Mr. Seely:

We are pleased to submit this geotechnical investigation report for the planned improvements to the Community Hospital campus in Riverside, California. The following report summarizes the findings of our subsurface investigation and field infiltration tests, provides our conclusions regarding the geologic constraints to development, and provides geotechnical recommendations for remedial grading, shoring, foundations, slabs, retaining walls and pavement section design.

We appreciate this opportunity to be of professional service. Please feel free to contact the office with any questions or comments, or if you need anything else.

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1.0 INTRODUCTION

The following report provides geotechnical recommendations for the planned improvements to the Community Hospital campus in Riverside, California. The site is located south of downtown Riverside and east of the Santa Ana River as shown in Figure 1A. Selected photographs of the site are shown in Figures 1B to 1E. The general locations of the planned Garage and Tower additions on the Riverside Community Hospital campus are shown in Figure 2. The approximate locations of the five Cone Penetration Test soundings, 10 exploratory borings and 12 infiltration tests that we completed on site are shown on the Exploration Plans, Figures 3A to 3C.

1.1 Scope of Services

Our geotechnical services were provided in general accordance with the provisions of the referenced proposal (GDC, 2023). The purpose of this work was to characterize the general geotechnical constraints to site development, and to provide geotechnical recommendations for grading and design of the new foundations, slabs, utilities, retaining walls and pavement sections. The recommendations provided herein are based on the findings of our subsurface investigation, laboratory tests and engineering analyses, and our previous experience with similar geologic conditions in the site vicinity. In summary, we provided the following services for this project.

- A visual and geologic reconnaissance of the surface characteristics of the site, a distress documentation, and review of the reports referenced in Section 8.0.

- A subsurface exploration of the site including five Cone Penetration Test (CPT) soundings and 10 exploratory borings. We also completed 12 borehole infiltration tests at six planned BMP locations. The approximate boring, CPT and infiltration test locations are shown on the Exploration Plans, Figures 3A to 3C. The CPT data interpretations and Boring Records are provided in Appendix A.

- Laboratory tests on soil samples collected from the explorations. Laboratory tests included sieve and hydrometer analysis, Atterberg Limits, in-situ moisture and density, Expansion Index, corrosivity, consolidation, direct shear and R-Value. The laboratory test results are presented in Appendix B.

- Engineering analysis of the field and laboratory data to help develop geotechnical recommendations for site preparation, remedial earthwork, foundation, shoring and retaining wall design, soil reactivity, site drainage and pavement sections. Our liquefaction and dynamic settlement analyses are summarized in Appendix C. The field infiltration tests and infiltration assessment are provided in Appendix D. Our site-specific seismic hazard evaluation is presented in Appendix E.

- Preparation of this report summarizing our findings, conclusions and providing geotechnical recommendations for the planned improvements.

1.2 Site Description

The site is located on the Riverside Community Hospital campus at 4445 Magnolia Avenue in downtown Riverside, California. The hospital campus is located southeast of the intersection between Brockton Avenue and 14th Street as shown on the Site Location Map, Figure 1A. Magnolia Avenue borders the eastern edge of the campus. The southern edge of the campus is bordered by two medical office buildings on the west and the Calvary Presbyterian Church on the east. Selected photographs of the site are provided in Figures 1B to 1E.

The subject site consists of the two portions of the hospital campus where the new parking Garage and Tower additions are proposed (see Figure 2). There are several existing structures on site that will need to be demolished prior to constructing these new buildings. The new Garage addition along Brockton Avenue is currently occupied by four structures, including two small one-story medical office buildings on the north, a larger Community Medical Building with a subterranean garage in the center, and the single-story Brockton Auto Clinic on the south as shown in Figure 3B. The Tower addition is located in an area currently occupied by a large four-story parking garage as shown in Figure 3C. The eastern half of this parking garage was initially built into the natural hillside. The garage was later expanded to the west (Crandall, 1981, 1985).

The Garage site along Brockton Avenue is relatively flat-lying and located about 795 to 800 feet above mean sea level (MSL). By comparison, the Tower site slopes up significantly from a low of about 795 feet MSL on the west, up to a high of roughly 840 feet MSL on the east. The proposed development areas are surrounded by various existing asphalt concrete driveway and parking areas, concrete sidewalks and landscaping areas. Existing subsurface utilities include water, sewer, storm drain, electrical and various communication conduits.

1.3 Proposed Improvements

Details of the proposed improvements are not yet available. We understand that the project will include demolishing the existing parking structure and constructing a new stand-alone 9 to 11-story 345,866 ft^2 (square feet) hospital Tower. The new Tower will consist of a steel framed building with wide-flange columns, girders and beams supporting composite concrete and steel decks. The foundation system is anticipated to consist of cast-in-drilled-hole (CIDH) or auger-cast piles with reinforced concrete caps and tie beams. Unbalanced soil conditions will require permanent shoring using soldier piles with tie-back anchors along the east wall of the Tower as well as portions of the north and south sides. Maximum shoring wall heights up to 40 feet are anticipated. A pedestrian bridge connector and drop-off canopy will also be constructed.

The existing buildings at the Garage site will also be demolished to make room for a new 3-story parking garage with a single level basement. The new Garage will replace the existing parking garage at the Tower site after demolition. The new Garage will be constructed using reinforced concrete walls with post-tensioned concrete floor slabs and beams. The foundation system for the new Garage will also consist of reinforced concrete piles, pile caps and grade beams.

Other site improvements will include various new or relocated utilities such as fire water service, domestic water, sanitary sewer and storm drain installations. Exterior concrete sidewalks and new asphalt concrete vehicular pavement areas will also be added to connect the new buildings to the existing driveway and parking areas that will remain. Various new landscaping and storm water Best Management Practice (BMP) drainage improvements are also proposed.

2.0 FIELD AND LABORATORY INVESTIGATION

Our field investigation program included five cone penetration test (CPT) soundings, 10 exploratory borings and 12 borehole infiltration tests. The field work was completed between March 22nd and April 5th, 2024. The maximum depth of exploration was about 55 feet below surrounding grades. The approximate exploratory boring, CPT and infiltration test locations are shown on the Exploration Plans, Figures 3A to 3C. The CPT data interpretations and Boring Records are provided in Appendix A. The infiltration test results are provided in Appendix D.

Various soil samples were collected from the exploratory borings, CPT soundings and infiltration boreholes for laboratory testing and analysis. The testing program included gradation, hydrometer analysis and Atterberg Limits to aid in material classification according to the Unified Soil Classification System (USCS) used by the American Society of Civil Engineers (ASCE). Tests were conducted on relatively undisturbed samples to help estimate the in-situ dry density and moisture content of the various materials we encountered on site. Expansion Index (EI) tests were conducted on bulk soil samples to help assess the potential for movement associated with soil heave. Corrosivity tests including pH, resistivity, water soluble sulfate and chloride contents were conducted to help assess the potential for corrosion of buried metals or sulfate attack of concrete structures in contact with the on-site soils. Consolidation tests were conducted to help characterize the compressibility of the Young Alluvium. Direct shear tests were conducted to aid in strength characterization. Finally, R-Value tests were conducted to aid in pavement section design. The laboratory test results are presented in detail in Appendix B.

2.1 Shear Wave Velocity

Shear wave velocity measurements were collected at the Garage site in both CPT-1 and CPT-2 at roughly 5-foot depth intervals until refusal was met within very dense Old Alluvium. The Garage site is underlain by 15 to 25-feet of loose Young Alluvium over dense Old Alluvium. The average shear wave velocity in the upper 100-feet (Vs_{30}) at the Garage site is estimated at approximately 1,390 feet per second (ft/s), corresponding to a 2022 CBC Site Class C (Soft Rock).

The Tower will be underlain by fill and compressible Young Alluvium on the west side of the pad, as well as dense Old Alluvium on the east side of the pad. The average shear wave velocity (Vs_{30}) at the Tower site is estimated at 1,600 ft/s based on measurements in CPT-5. This corresponds to a 2022 CBC Site Class C. The shear wave velocity of the Old Alluvium was previously measured by others at the Tower site at between 1,940 and 2,560 ft/s as shown in Appendix A1 (CHJ, 2008). A shear wave velocity of 1,940 ft/s was assumed for the Old Alluvium in our Vs_{30} estimates.

2.2 Previous Investigations

A total of 10 previous exploratory borings and six previous Cone Penetration Tests (CPT) were conducted by others in close proximity to the planned Garage and Tower additions. Logs for these 15 previous field explorations are included in Appendix A1 for reference. The approximate locations of these previous explorations are also shown in Figures 3B and 3C. The relevant findings from the previous geotechnical investigations are summarized below.

The proposed Tower is located in an area that is currently covered with an existing parking garage that will need to be demolished prior to the planned redevelopment (see Figure 3C). The existing parking garage was constructed in two phases. The initial phase of construction included only the eastern half of the parking garage, which was built into the natural hillside of the Tequesquito Arroyo. The initial field investigation program for this parking garage included three exploratory borings in the Tower area as described in the referenced report (Crandall, 1981).

A supplemental field investigation with three more borings was conducted when the second half of the garage was added along the western edge of the first phase (Crandall, 1985). Most of the available borings for the parking garage were completed using a large diameter bucket auger, and the blow counts provided for these borings are inappropriate for our use in geotechnical correlations. Consequently, logs for these initial parking garage borings are not included in Appendix A1. However, the supplemental 1985 geotechnical investigation did include a detailed topographic survey showing the general site conditions at that time. The Community Medical Building and Brockton Auto Clinic at the Garage site had already been constructed in 1985, as well as the eastern half of the existing parking garage at the Tower site. Much of the campus between the Tower and Garage sites consisted of asphalt concrete paved parking areas in 1985. Most of these parking lots were subsequently demolished.

CHJ Consultants completed a thorough investigation around the perimeter of the existing parking garage for a previous Tower concept that was never constructed. The relevant logs from this investigation include Borings B-1, B-2, B-6, B-7, B-8 and B-9 as well as soundings CPT-1 to CPT-6 (CHJ, 2008). A borehole seismic velocity survey was also conducted by CHJ in Boring B-1 for the 2007 field investigation. This shear wave velocity profile is attached in Appendix A1. Groundwater was measured in Borings B-1, B-2 and B-6 at elevations ranging from 760 to 764 feet MSL. This corresponds to groundwater depths of 36 to 39 feet below grade along the western edge of the Tower site. Groundwater was encountered at about 77 feet below grade in Boring B-6 along the eastern edge of the Tower, which corresponds to a similar groundwater elevation of 762 feet.

A separate geotechnical investigation was also conducted by CHJ for the existing Medical Office Building (MOB) located immediately south of the planned Garage addition (see Figure 3B). The logs for Borings B-1 and B-2 from the MOB investigation are included in Appendix A1 (CHJ, 2012). Note that groundwater was measured in Boring B-2 for the MOB investigation at a depth of about 37½ feet, corresponding to a groundwater elevation of about 755 feet MSL at that time.

Appendix A1 also includes logs for two previous Boring B-1 and B-2 conducted by AMEC for hospital Tower G that was constructed in 2015 (AMEC, 2013). Although these two borings were not located in the current development area, the conditions AMEC encountered were similar to the general conditions we observed at the site. Groundwater was encountered in Boring B-1 at a depth of roughly 31 feet below grade, corresponding to an elevation of about 764 feet MSL.

2.3 Distress Observations

As part of our investigation, we observed and photographed signs of existing distress throughout the proposed Garage and Tower sites. In general, the existing pile supported garage structure at the Tower site has performed reasonably well to date with few signs of cracking and distress (see Figures 1B and 1C). By comparison, the Community Medical Center along Brockton Avenue, the three minor structures, and the surface improvements at the Garage site show extensive signs of cracking and distress (see Figures 1D and 1E). Our distress observations are summarized below. Additional photographs of the distressed areas may be provided upon request.

Within the interior of the existing parking structure at the new Tower site, we observed relatively few cracks within the reinforced concrete columns and beams. The western portion of the slab-on-grade for the garage addition did have a few minor cracks (Crandall, 1985). Fairly extensive cracks were also observed around the perimeter of the structure. The concrete sidewalk along the eastern edge of the MOB immediately southwest of the Tower site was extensively cracked. The asphalt concrete pavements in this area also had extensive alligator cracking. Note that the western part of the Tower site is situated over compressible Young Alluvium. The patterns of cracking we observed in this area appear to be consistent with soil settlement.

The eastern portion of the existing garage at the Tower site is raised above grade and supported by concrete beams, columns and pile foundations (Crandall, 1981). This initial garage structure also appears to have performed fairly well to date, with only a few signs of cracking. However, the slope beneath this portion of the garage does show considerable signs of piping and soil erosion. One of the reinforced concrete beams in this area was longitudinally cracked with indications of seepage and corrosion of the steel reinforcement (i.e. rust stains). There are also indications of settlement of the asphalt concrete pavement located immediately east of the pile supported garage. Extensive cracks were also observed around the elevator tower in the northwest corner of the initial garage structure. Finally, the soldier pile shoring between the garage and Tower C is exposed along the staircase between these structures and appears to have been partially demolished.

In comparison to the Tower site, the existing structures at the new Garage site show numerous signs of cracking and distress. The existing 4-inch-thick concrete pavements which surround the Brockton Auto Clinic are cracked on a relatively tight spacing. The 2-inch-thick asphalt concrete parking lot in the northern portion of the Garage site is also extensively cracked (see Figure 1D). The extensive cracking of these surface improvements may be related to an inadequate initial pavement section design. However, these cracks may also be related to soil settlement.

Extensive damage was observed within the basement garage of the Community Medical Center at the proposed Garage site. The basement extends roughly 10-feet below surrounding grades and was conducted using cement masonry units (CMU). It appears that this structure is founded on shallow spread footings. The CMU walls are badly cracked beneath nearly all of the joists that support the first floor of the building (see Figure 1E). Most of the cracking was hairline in width, although a few of the cracks were 1/8th of an inch or wider. These regular cracks extended from the bottom of the joists all the way down through the CMU wall to the slab-on-grade. The concrete slab-on-grade was also extensively cracked throughout the structure, and cracks were also observed in the slab around the column footings. The pattern of distress in the basement garage of the Community Medical Center is consistent with soil and foundation settlement.

2.4 Infiltration Testing

Twelve borehole infiltration tests were conducted for this investigation, with two tests at each of the six proposed storm water Best Management Practice (BMP) improvement areas shown on the Exploration Plan, Figure 3A. The infiltration test results are presented in Appendix D. The test results are also tabulated below. A Safety Factor of 2.0 was applied to the factored infiltration rates. A correction factor was also applied for the average temperature of the water used for the infiltration testing versus an assumed average rainfall temperature of 60°F.

BMP	Test No.	Soil Description	Stabilized Rate [IN/HR]	Factored Rate [IN/HR]	Infiltration Assessment
1	I-1A	**Alluvium**: Reddish brown silty sand (SM)	1.57	0.64	Full Infiltration
	I-1B	**Alluvium**: Reddish brown silty sand (SM)	2.69	1.08	Full Infiltration
2	I-2A	**Alluvium**: Brown well-graded sand (SW)	2.92	1.36	Full Infiltration
	I-2B	**Alluvium**: Brown well-graded sand (SW)	2.63	1.32	Full Infiltration
3	I-3A	**Fill:** Dark yellow brown sandy silt (ML)	0.19	0.08	Partial Infiltration
	I-3B	**Fill:** Dark yellow brown sandy silt (ML)	0.04	0.02	Partial Infiltration
4	I-4A	**Fill:** Dark brown clayey sand (SC)	0.19	0.08	Partial Infiltration
	I-4B	**Fill:** Dark yellow brown silty sand (SM)	0.54	0.23	Partial Infiltration
5	I-5A	**Fill:** Dark yellow brown clayey sand (SC)	0.08	0.04	No Infiltration
	I-5B	**Fill:** Dark yellow brown clayey sand (SC)	0.03	0.02	No Infiltration
6	I-6A	**Alluvium**: Yellow brown silty sand (SM)	1.60	0.69	Full Infiltration
	I-6B	**Alluvium**: Yellow brown silty sand (SM)	1.95	0.83	Full Infiltration

The infiltration tests for Basins 1, 2 and 6 had factored rates from 0.64 to 1.36 and averaging 0.99 inches per hour. A factored vertical infiltration rate greater than 0.50 inches per hour is typically deemed indicative of a "Full Infiltration" condition per common guidelines. These three BMPs are located in areas with shallow surficial fill soils overlying loose Young Alluvium. The infiltration tests for Basin 2 were conducted in 8-foot-deep boreholes per the civil engineer's request.

By comparison to Basins 1, 2 and 6, the factored vertical infiltration rates for Basins 3 and 4 varied from about 0.02 to 0.23 inches per hour and averaged 0.10 inches per hour. A factored infiltration rate between 0.05 and 0.50 inches per hour is commonly considered a "Partial Infiltration" condition. The borings we excavated within Basins 3 and 4 encountered medium dense compacted fill soils. It has been our experience that permeability and infiltration rates decrease significantly with increased soil density.

The factored infiltration rates for Basin 5 varied from about 0.02 to 0.04 and averaged 0.03 inches per hour. A factored infiltration rate less than 0.05 inches per hour indicates a "No Infiltration" condition per common BMP Design guidelines. The soil we encountered in the infiltration borehole excavations for Basin 5 consisted of a dense clayey sand (SC) compacted fill.

3.0 GEOLOGY AND SUBSURFACE CONDITIONS

The site is situated within the Peninsular Ranges geomorphic province of southern California. This province, which stretches from the Los Angeles basin to the tip of Baja California, is characterized as a series of northwest trending mountain ranges separated by subparallel fault zones. The general geologic conditions in the site vicinity are depicted on the Local Geologic Map, Figure 4A. Geologic cross sections of the site are provided in Figures 4B and 4C.

The site is underlain at depth by Old Alluvial Fan Deposits associated with the Santa Ana River system (Geologic Map Symbol - Qof). These materials are referred to as "Old Alluvium" throughout this report. The Old Alluvium is covered with roughly 15 to 25 feet of loose to medium dense Young Axial Channel Deposits (Qya) beneath the planned Garage as well as the western edge of the Tower site. These compressible deposits are termed "Young Alluvium" in this report. Artificial Fill (Qaf) covers the Young Alluvium throughout much of the site, with the deepest fill soils along the eastern edge of the planned Tower. Groundwater was encountered in our borings and previous borings at depths of more than 30-feet below surface grades throughout the site. The subsurface conditions we encountered in the borings and CPT soundings are shown in Appendix A. The geologic materials encountered at the site are described in detail below.

3.1 Old Alluvium

Pleistocene-age Old Alluvium (Qof) associated with the Santa Ana River was encountered in all of our explorations at depth. The Old Alluvium is predominately granular in nature and includes well graded sand (SW), well-graded sand with silt (SW-SM), poorly-graded sand with silt (SP-SM), silty sand (SM) and clayey sand (SC). The Old Alluvium also contained occasional beds of both well-graded sand with gravel (SW) and silty sand with gravel (SM).

Corrected SPT blow counts (N_{60}) within the Old Alluvium typically ranged from 30 to 100 or more and averaged 59, which is indicative of a dense to very dense material. All five of the CPT soundings we conducted at the site met with refusal in the Old Alluvium with tip resistance in excess of 700 tons per square foot (TSF) as shown in Figures A-1 to A-5 in Appendix A.

The dry unit weight of the Old Alluvium samples we tested ranged from about 107 to 127 lb/ft^3 and averaged 116 lb/ft^3. The moisture content of the Old Alluvium ranged from 2.5 to 13.5 and averaged 7.6 percent. The average moist unit weight of the Old Alluvium above groundwater is estimated at about 125 lb/ft^3.

3.2 Young Alluvium

Holocene Young Alluvium (Qya) associated with the Tequesquito Arroyo was encountered in most of our explorations. An arroyo is a normally dry watercourse that occasionally experiences flooding after heavy rainfall events within the local watershed. The Young Alluvium conformably overlies the Old Alluvium at depths ranging from about 15 to 25 feet below grade. The Young Alluvium primarily consists of well-graded sand with silt (SW-SM), poorly-graded sand with silt (SP-SM), silty sand (SM), clayey sand (SC) or sandy silt (ML) with occasional gravel. A thick bed of lean clay (CL) was also encountered within the Young Alluvium beneath the northern portion of the Garage site. The Liquid Limit of the clay samples we tested ranged from 37 to 46, with a Plasticity Index of 18 to 24. The undrained shear strength (Su) of the lean clay bed varied from about 2 to 3 KSF.

Corrected SPT blow counts (N_{60}) within the Young Alluvium ranged from 5 to 27 and averaged 12, which is indicative of a loose to medium dense material. Note that nearly half of the SPT tests we conducted within the Young Alluvium had corrected blow counts of 9 or less, indicating a loose and highly compressible state. Many of the low blow counts were obtained within fine-grained beds of sandy silt (ML) within the Young Alluvium.

The dry unit weight of the Young Alluvium samples we tested ranged from about 101 to 117 lb/ft^3 and averaged 110 lb/ft^3. The moisture content of the Young Alluvium typically ranged from 2.6 to 16.0 and averaged 9.8 percent. The average moist unit weight of the Young Alluvium is estimated to be approximately 120 lb/ft^3. Direct shear tests suggest that the sandy Young Alluvium has a drained friction angle which generally exceeds 36^o with 100 lb/ft^2 cohesion.

3.3 Artificial Fill

Shallow Artificial Fill (Qaf) was encountered in most of our explorations. About 28 feet of fill was observed along the eastern edge of the Tower site in Boring B-10. The fill we observed generally consisted of silty or clayey sand (SM or SC) with some sandy silt (ML). Corrected SPT blow counts (N_{60}) within the Artificial Fill ranged from 21 to 33 and averaged 27, which is indicative of a medium dense material on average. Laboratory tests indicate that the fill has a very low expansion potential (EI<20) and negligible sulfate content. Direct shear tests suggest that the sandy fill typically has a drained friction angle which exceeds 36^o with 100 lb/ft^2 cohesion. Most of the fill was likely derived from excavations within the on-site alluvium and is therefore similar in composition. Direct shear tests also indicate that the sandy silt (ML) layers within the fill have a friction angle on the order of 30^o with 300 lb/ft^2 cohesion (see Figure B-5.4 in Appendix B).

3.4 Groundwater

Groundwater was encountered in Boring B-2 at a depth of 36 feet below grade which corresponds to a current groundwater surface elevation of about 762 feet MSL. Groundwater was encountered in three previous borings at the Tower site at elevations ranging from 760 to 764 feet (CHJ, 2008). Groundwater was encountered in the MOB investigation immediately south of the Garage site at an elevation of 755 feet MSL (CHJ, 2012). Groundwater was also measured in 2013 at an elevation of about 764 feet MSL between the Tower and Garage sites (AMEC, 2013).

Based on the historic groundwater measurements at the subject site, the groundwater surface elevation beneath the site is estimated to vary from about 755 to 764 feet over time. The ground surface elevation at the site is typically 793 feet or higher. Consequently, groundwater depths will typically vary from about 30 to 45 feet below grade in the lower portions of the site. The ground surface elevation along the eastern edge of the Tower site is higher at nearly 840 feet. Therefore, groundwater will typically be located more than 70 feet below grade in that area.

It should be noted that groundwater levels do fluctuate over time. Changes in rainfall, irrigation or site drainage may result in seepage or locally perched groundwater at any location within the fill or alluvial soils which underly the site. Due to the difficulty in predicting the location of perched groundwater, such conditions are typically mitigated if and where they occur.

4.0 GEOLOGIC HAZARDS

The site is located in a highly active seismic area between the San Jacinto and Elsinore fault zones, as shown on the Regional Fault Map, Figure 5A. Potential geologic and seismic hazards include ground rupture, strong ground shaking, seismic settlement and soil liquefaction, and earthquake induced flooding. Each of these hazards are described in the referenced City of Riverside Public Safety Element (City of Riverside, 2021). Each of these potential hazards is also discussed below.

4.1 Ground Rupture

Ground rupture is the result of movement on an active fault reaching the ground surface. Known faults within 100 kilometers (km) of the site are shown on the Regional Fault Map, Figure 5A. The nearest known active fault is the San Bernardino segment of the San Jacinto fault zone located about 13 km northeast of the site. The San Jacinto fault is a right-lateral strike-slip fault zone believed to be capable of producing an earthquake with a moment magnitude (M_w) up to 8.1.

The San Andreas fault zone may also produce an earthquake of about M_w 8.0, and is located about 24 km northeast of the site. The Elsinore fault zone is located about 24 km southwest of the site. The Elsinore fault is believed to be capable of producing an earthquake of M_w 7.3. The site is not located within an Alquist-Priolo Earthquake Fault Zone, and no evidence of Holocene active or potentially active faulting was encountered in our investigation or literature review. Ground rupture is not considered to be a significant geologic hazard at this site.

4.2 Seismicity

The planned structures may be subjected to strong ground shaking over their design life. The strong shaking hazard is typically managed by structural design per the applicable seismic provisions of the governing edition of the California Building Code (CBC). Based on the shear wave velocities (Vs_{30}) measured at the site by Group Delta and others, it is our opinion that a 2022 CBC Site Class C (Soft Rock) may be applied to the seismic design of both the new Garage and Tower. General Procedure acceleration response spectra for the Garage and Tower sites are provided in Tables 1 and 2. Our site-specific seismic hazard evaluation is described in detail in Appendix E.

4.3 Liquefaction and Dynamic Settlement

Liquefaction involves the loss in strength of a saturated, cohesionless soil (sand and silts) caused by the build-up of pore water pressure during cyclic loading from an earthquake. The increase in pore water pressure temporarily transforms the soil into a fluid mass, resulting in sand boils, settlement and lateral ground deformations. Typically, liquefaction occurs in areas where there are loose to medium dense sands, and where the depth to groundwater is less than 50 feet from the surface. In summary, three simultaneous conditions are required for liquefaction:

- Historic high groundwater within 50 feet of the ground surface
- Liquefiable soils such as loose to medium dense sands
- Strong shaking, such as that caused by an earthquake

The Young Alluvium is susceptible to seismic settlement due to an earthquake. Liquefaction and seismic settlement analyses were conducted using the 2022 CBC site modified MCE level Peak Ground Acceleration (PGA_M) of 0.615g, along with a maximum moment magnitude of M_w 8.1. A high groundwater level of 25 feet below grade was used for the analyses, and we assumed that remedial earthwork would be conducted per our recommendations to compact the loose surficial soils directly beneath the building pads. Our analyses indicate that the total seismic settlement may range from about ½ to 3 inches. According to state guidelines, a differential settlement equal to one-half of the total settlement may be conservatively assumed for structural design (SCEC, 1999). We estimate a differential settlement from the combined effects of seismic compaction of dry soil above groundwater and post-liquefaction settlement below groundwater of about 1½ inches across 40 feet. The analyses are provided in Appendix C.

4.4 Landslides and Lateral Spreads

Evidence of ancient landslides or slope instabilities was not observed during our literature review or site reconnaissance. Most of the site is relatively flat. The vertical cuts along the eastern edge of the Tower site will need to be shored and designed to accommodate surcharge loads. Provided that our recommendations are properly implemented during construction, and that temporary excavations are conducted in accordance with Cal-OSHA requirements, it is our opinion that slope instability should not adversely impact the proposed improvements.

4.5 Tsunamis, Seiches and Flooding

The site is located more than 60 km from the Pacific Ocean with existing surface grades which vary from about 790 to 840 feet above mean sea level (MSL). The relatively far distance between the site and the coastline, and the relatively high elevation of the site indicates that the potential for a tsunami or seiche to impact the site is negligible.

According to the City of Riverside Public Safety Element, the entire hospital campus is located within a potential Dam Inundation Zone in the event of a seismic failure of either the Sycamore Canyon or Box Springs Dam (Riverside, 2012). The California Geological Survey has indicated that the probability of seismic dam failure is relatively low. Therefore, the potential for damage to the proposed improvements due to earthquake induced flooding of the site is considered low.

The site is located in relatively close proximity to the FEMA 100-year flood zones associated with both the Santa Ana River and the tributary Tequesquito Arroyo. Most of the flow within the arroyo is contained within underground storm drain pipes south of the site. The Federal Emergency Management Agency (FEMA) 100-year flood elevations for the site are shown in Figure 5B. Note that the FEMA flood map indicates that the site may experience minor flooding due to a "one percent annual chance of flood discharge contained in structures" (FEMA, 2021).

4.6 Settlement

Most of the Young Alluvium on site is granular and should settle rapidly when subjected to new fill or foundation loads. Minor hydro-compression settlement may also occur when the Young Alluvium is wetted after heavy rainfall or floods (AMEC, 2013). In addition, a bed of lean clay (CL) was observed at the north end of the Garage site at 20 to 30-feet below grade. We tested two relatively undisturbed samples of the lean clay in accordance with ASTM D2435. The consolidation test results are presented in Figures B-4.1 and B-4.2 in Appendix B.

The lean clay samples we tested had Virgin Compression Indices (Cc) ranging from about 0.114 to 0.128 in strain domain. The Recompression or Swell Indices (Cs) ranged from about 0.026 to 0.031. The clay was mildly over-consolidated with an estimated OCR of 2.6 to 3.0. For our time rate of settlement analyses, the Coefficient of Consolidation (Cv) was estimated from the consolidation tests to range from approximately 0.01 to 0.04 in^2/min. Note that the clayey Young Alluvium was encountered above the groundwater table at about 90 percent saturation.

Our settlement analyses indicate that the lean clay bed beneath the Garage site may experience several inches of long-term consolidation settlement when subjected to new fill or foundation loads (depending on the load magnitude). The sandy Young Alluvium will settle rapidly. However, our analyses indicate that the clayey Young Alluvium may settle for between 6 and 12 months after application of new fill or foundation loads. The new Garage and Tower structures will be supported by pile foundations which should mitigate this hazard. However, Young Alluvium settlement may still impact minor structures which are supported only by shallow foundations.

5.0 CONCLUSIONS

The planned Garage and Tower additions appear to be feasible from a geotechnical perspective, provided that appropriate measures are implemented during design and construction to manage the potential for adverse settlement. The geotechnical site constraints are summarized below.

- Much of the site is underlain by 15 to 25 feet of Young Alluvium. These predominately granular soils will experience rapid settlement due to new fill or foundation loads, hydro-compression, or seismic settlement during a strong earthquake. Heavily reinforced slabs-on-grade with foundations dimensioned using a relatively low bearing pressure may help reduce settlement *for minor structures and improvements*. The new Garage and Tower structures should be supported by deep pile foundations embedded into dense Old Alluvium along with structural slabs, where needed. Geotechnical parameters for use in cast-in-drilled-hole or auger-cast pile design are provided in the following sections.

- Groundwater was encountered on site at depths of more than 30 feet below existing surface grades, corresponding to a groundwater surface elevation ranging from about 755 to 764 feet MSL. The Young Alluvium had a relatively low water content in the soil samples we tested. In general, we do not anticipate that shallow groundwater will be a significant design and construction issue. However, we experienced heaving soils in our exploratory borings due to rapid groundwater intrusion at depths of between 40 and 50 feet below grade. CIDH piles will need to be installed using wet methods including drilling slurry and temporary casing, if needed. Gamma-Gamma inspection tubes may be installed in CIDH piles which extend below groundwater in order to verify the absence of anomalies.

- The infiltration testing indicates that the loose Young Alluvium is highly permeable and should provide a “Full Infiltration” condition. However, the surficial compacted fill soils include fine grained sandy silt and clayey sand which may be medium dense to dense in consistency. These compacted fill soils will present a “Partial Infiltration” or “No Infiltration” condition based on our field infiltration test results for Basins 3, 4 and 5.

- The on-site soils are generally considered suitable for reuse as compacted fill with the exception of any expansive clays may be encountered in isolated areas. Note that Group Delta Consultants did not provide environmental testing services for this project. Laboratory tests indicate that while the on-site soils generally have a *negligible* soluble sulfate content, although they may be moderately *corrosive* to buried metals. A corrosion consultant may be contacted for specific recommendations.

- The potential for ground rupture or landslides to adversely impact the site is considered low. Seismic design should be conducted in accordance with CBC requirements. The site is located below the Sycamore Canyon and Box Springs Dams, and in close proximity to the FEMA 100-year flood zones for the Santa Ana River and Tequesquito Arroyo. The potential for flooding at the site should be evaluated by the project civil engineer.

6.0 RECOMMENDATIONS

The remainder of this report presents recommendations for earthwork construction and the design of the proposed improvements. These recommendations are based on empirical and analytical methods typical of the standards of practice in southern California. If these recommendations do not cover a specific feature of the project, please contact our office for revisions or amendments.

6.1 Plan Review

We recommend that grading, shoring and foundation plans be reviewed by Group Delta Consultants prior to finalization. We anticipate that substantial changes in the development may occur from the preliminary design concepts used for this investigation. Such changes may require additional geotechnical evaluation, which may result in substantial modifications to the remedial grading and foundation recommendations provided in this report.

6.2 Excavation and Grading Observation

Foundation, shoring and grading excavations should be observed by the geotechnical consultant. During grading, the geotechnical engineer's representative should provide observation and testing services continuously. Such observations are considered essential to identify field conditions that differ from those anticipated by this investigation, to adjust designs to the actual field conditions, and to determine that the grading is accomplished in general accordance with the recommendations presented in this report. The recommendations provided in this report are contingent upon Group Delta Consultants providing these services. Our personnel should perform sufficient testing of fill and backfill during grading and improvement operations to support our professional opinion as to compliance with the compaction recommendations.

6.3 Earthwork

Grading and earthwork should be conducted in general accordance with the requirements of the City of Riverside, the California Building Code and the earthwork recommendations provided within this report. The following recommendations are provided regarding specific aspects of the proposed earthwork. These recommendations should be considered subject to revision based on the conditions observed by the geotechnical consultant during the grading operations.

6.3.1 Site Preparation

General site preparation should begin with the removal of deleterious materials, including any existing structures, vegetation, contaminated soil, trash, pavements and demolition debris. Existing subsurface utilities that will be abandoned should be removed and the excavations backfilled and compacted as described in Section 6.3.4. Alternatively, abandoned pipes may be grouted in place using a two-sack sand-cement slurry under the observation of the project geotechnical consultant.

6.3.2 Improvement Areas

At least two feet of compacted fill with an Expansion Index of 20 or less is recommended beneath all new concrete sidewalks, exterior flatwork, pavement areas and minor structure foundations. To accomplish this objective, the upper 12-inches of subgrade soil should be excavated, cleared of deleterious materials and stockpiled. The excavation bottom should be observed by Group Delta to determine if additional removals are needed. The over-excavated area should extend at least 2-feet outside the limits of the new improvement areas (measured horizontally). Imported or on-site granular soil with an Expansion Index less than 20 should then be used to backfill the excavation in accordance with Section 6.3.4 below. Subgrade compaction should be conducted immediately prior to placing concrete (or aggregate base in new pavement areas).

6.3.3 Building Areas

Both the surficial fill and Young Alluvium are considered compressible under new fill or foundation loads. Furthermore, the entire site may also experience some seismic settlement due to a strong earthquake. Although the new Garage and Tower structures will be supported by pile foundations, the building walls and slabs-on-grade may still be damaged by such settlement.

We recommend that remedial earthwork be conducted beneath the entire Garage slab-on-grade to excavate and recompact the upper 4 feet of Young Alluvium beneath the proposed finish pad elevation of 783 feet MSL. Similarly, remedial earthwork should also be conducted beneath the western portion of the Tower site to excavate and recompact the upper 10 feet of fill and Young Alluvium in that area. The remedial excavations for the Tower should extend at least 10 feet horizontally beyond the perimeter of the building (where possible). Remedial excavations should not extend below a 1:1 plane projected down and out from the bottom outside edge of existing foundations or improvements to reduce the potential for distress. The excavations should be backfilled using very low expansion (EI<20) on-site or imported compacted fill material.

6.3.4 Fill Compaction

All fill and backfill should be placed at slightly above optimum moisture content using equipment that is capable of producing a uniformly compacted product. The minimum recommended relative compaction is 90 percent of the maximum dry density per ASTM D1557. Sufficient observation and testing should be performed by the geotechnical consultant during grading so that an opinion can be rendered as to the compaction achieved.

Rocks or concrete fragments greater than 6 inches in dimension should not be used in compacted fill. Imported fill sources should be observed prior to hauling onto the site to determine the suitability for use. In general, imported fill materials should consist of granular soil with less than 35 percent passing the No. 200 sieve based on ASTM C136 and an Expansion Index less than 20 based on ASTM D4829. Samples of the import should be tested by the geotechnical consultant in order to evaluate the suitability of these soils for their proposed use.

A two-sack sand and cement slurry may be used as an alternative to compacted fill soil. It has been our experience that slurry is often useful in confined areas which may be difficult to access with typical compaction equipment. A minimum 28-day compressive strength of 100 psi is recommended for the two-sack sand and cement slurry. A three-sack slurry with a minimum 28-day strength of 300 psi is recommended for use beneath structural foundations. Samples of the slurry should be fabricated and tested for compressive strength during construction.

6.3.5 Subgrade Stabilization

All excavation bottoms should be firm and unyielding prior to placing fill. In areas of saturated or "pumping" subgrade, a geogrid such as Tensar BX-1200 or Terragrid RX1200 may be placed directly on the excavation bottom, and then covered with at least 12 inches of minus ¾-inch aggregate base. Once the excavation is firm enough to attain the required compaction within the base, the remainder of the excavation may be backfilled using either compacted soil or aggregate base. If wet soil conditions or groundwater seepage is encountered in the excavations, an additional 12-inches of free draining open graded material (such as minus ¾-inch crushed rock) should be placed between the stabilizing geogrid and the compacted well graded aggregate base.

6.3.6 Surface Drainage

Foundation and slab performance depends greatly on how well surface runoff drains from the site. The ground surface should be graded so that water flows rapidly away from the structure and top of slope without ponding. The surface gradient needed to achieve this may depend on the prevailing landscaping. Planters should be built so that water will not seep into the foundation, slab, or pavement areas. If roof drains are used, the drainage should be channeled by pipe to the storm drain system, or discharge at least 10 feet from buildings. Irrigation should be limited to the minimum needed to sustain landscaping. Excessive irrigation, water line breaks, or rainfall may cause perched groundwater to develop within the underlying soil.

6.3.7 Storm Water Management

Various bioretention basins, swales or pervious paver block pavements will be constructed at the site in order to promote on-site infiltration for storm water Best Management Practice (BMP). All new BMPs should also be located at least 10-feet away from any building, as well as a minimum distance equal to the retained height for improvements near basements or shoring walls.

To determine the feasibility of on-site infiltration measures, the infiltration rate of the on-site soil was estimated at 12 locations. Two borehole percolation tests were conducted within each of the six planned BMP areas. The testing indicates that Basins 1, 2 and 6 within the Young Alluvium should provide a "Full-Infiltration" condition with factored infiltration rates in excess of 0.50 inches per hour. Our test indicate that Basins 3 and 4 should provide a "Partial Infiltration" condition with factored rates between 0.05 and 0.50 inches per hour. Our tests at Basin 5 indicated a "No Infiltration" condition with an average factored infiltration rate below 0.05 inches per hour. The tests were described in Section 2.4 and are presented in detail in Appendix D.

6.3.8 Temporary Excavations

Temporary excavations may be needed to construct the planned improvements. All excavations should conform to Cal-OSHA guidelines. In general, we recommended that temporary excavations be inclined no steeper than 1:1 for heights up to 10 feet. Vertical excavations should be shored. The design, construction, maintenance and monitoring of temporary slopes is the responsibility of the contractor. The contractor should have a competent person evaluate the geologic conditions during excavation to determine permissible temporary slope inclinations and other measures as required by Cal-OSHA. The following OSHA Soil Types may be assumed for temporary slope design.

Geologic Unit	Cal/OSHA Soil Type
Compacted Fill	Type B
Young Alluvium	Type C
Old Alluvium	Type B[1]

1. This assumes that no groundwater seepage or caving is encountered in the excavations.

6.4 Foundation Recommendations

Foundations should be designed by the structural engineer using the following geotechnical parameters. These are only minimum criteria, and should not be considered a structural design, or to preclude more restrictive criteria of governing agencies or the structural engineer. The following recommendations should be considered preliminary, and subject to revision based on the conditions observed by the geotechnical consultant during grading.

6.4.1 Shallow Foundations

The following recommendations assume that remedial grading will be conducted within all improvement areas as recommended in Sections 6.3.1 and 6.3.2. Conventional shallow foundations may be used to support minor structures, provided that the potential for settlement of these structures is acceptable to the design team. The Garage and Tower structures should be supported by piles as described in Section 6.4.2. The eastern edge of the Tower may be supported by shallow foundations, provided that they bear directly within dense Old Alluvium below Elevation 810 feet MSL. Shallow foundations should be at least 18-inches wide and 24-inches deep (see Figure 6). The following parameters may be used for design of shallow foundations.

Allowable Bearing: (Compacted Fill) — 2,000 lbs/ft^2 (⅓ increase for short-term loads).

Allowable Bearing: (Old Alluvium): — 3,000 lbs/ft^2 (The bearing may be increased by 500 lb/ft^2 per foot increase in width, and by 1,000 lbs/ft^2 for each additional foot of depth up to a maximum of 6,000 lbs/ft^2).

6.4.2 Deep Foundations

Deep pile foundations are recommended for support of the Garage structure to mitigate the potential for settlement of the loose Young Alluvium. We anticipate that 24 to 36-inch diameter piles may be used. Cast-In-Drilled-Hole (CIDH) or auger-cast piles embedded at least 10-feet into dense Old Alluvium are recommended. The Young Alluvium varies from about 15 to 20 feet in depth at the south side of the Garage site, with roughly 25 to 30 feet of Young Alluvium beneath the north end of the Garage site. However, the new Garage basement level will extend more than 10-feet below existing grades to a finish floor elevation of about 783 feet MSL (see Figure 4B). For our axial pile capacity analyses, each pile was assumed to be spaced at least 4 pile diameters such that group effects may be neglected. Axial capacity charts for 24 to 36-inch diameter piles located at the north and south end of the Garage are provided in Figures 7A and 7B, respectively.

Pile foundations embedded at least 10-feet into dense Old Alluvium may also be used to support the Tower. Shored excavations will be used to lower the eastern edge of the Tower site down into dense Old Alluvium with a finish floor elevation of about 798 feet MSL. Shallow foundations embedded into Old Alluvium may also be used to support the eastern edge of the Tower at the discretion of the structural engineer (see Section 6.4.1). Preliminary axial capacity charts for CIDH or auger-cast piles at the west and east edge of the Tower are provided in Figures 7C and 7D, respectively. Based on the finish floor elevation, we understand that piles located along the eastern edge of the Tower will be entirely embedded into dense Old Alluvium (see Figure 4C).

The factored axial pile capacity charts provided in Figures 7A through 7D include both skin friction and end bearing within the dense Old Alluvium. Skin friction was ignored in the Young Alluvium. By not including skin friction in the compressible Young Alluvium in the axial capacities, any drag loads imposed on the piles due to soil settlement will be accounted for in the pile design. The axial pile capacities used Load and Resistance Factor Design (LRFD) with Resistance Factors of 0.7 and 0.5 applied to the factored skin friction and end bearing, respectively. A Resistance Factor of 0.7 may be used for end bearing if a pile load test program is implemented, as discussed below.

For piles spaced less than 4 pile diameters center to center, Group Reduction Factors may apply to the single pile axial resistances shown in Figures 7A to 7D. Group reduction factors (η) may be estimated in accordance with Table 10.8.3.6-1 of the 6th Edition of the AASHTO LRFD Design Code with California Amendments. Note that for piles located along the east side of the Tower site that are entirely embedded within dense Old Alluvium, the Group Reduction Factor may be taken as 1.0 for a center to center spacing of 2 pile dimeters or greater per Table 10.8.3.6-1. Additional Group Reduction Factors for specific pile configurations may be provided upon request.

The pile excavations will extend below groundwater and may encounter unstable bottom conditions and sidewall caving. The Contractor should be prepared to use appropriate wet methods to stabilize the pile excavations. We understand that auger-cast piles will be used to control caving. A detailed pile installation plan should be provided by the contractor for review by the design team at least 30 days prior to commencing with the pile excavations.

The factored axial capacities provided in Figures 7A and 7B for the Garage site incorporate the standard Resistance Factor of 0.5 on end bearing. The factored axial capacities provided for the Tower site in Figures 7C and 7D incorporate a Resistance Factor of 0.7 on end bearing. Note that the Resistance Factor of 0.7 for end bearing is only deemed to be appropriate when a successful pile load test program is implemented prior to construction to verify the ultimate axial capacities we provided for each pile diameter that is used. We understand that load tests will be conducted for the Tower site only. The test piles should be constructed using the same means and methods proposed by the Contractor for the production piles, including the recommended remedial grading. The proposed test pile location for the Tower site is shown in Figure 3C.

The ultimate test pile capacities should be defined at 1-inch of total pile displacement. Static axial pile load tests should be conducted in general accordance with ASTM D1143. Alternatively, Bi-Directional axial load tests may be conducted using ASTM D8169. Lateral pile load tests may also be beneficial to better define the shear loads associated with ¼, ½ and 1-inch of pile head deflection. Lateral pile load tests should be conducted in general accordance with ASTM D3966. The Contractor should provide a detailed description of the proposed pile installation and testing program for review by the design team at least 30 days prior to commencing with construction.

6.4.3 Settlement

Total and differential settlement of new shallow foundations under the allowable bearing loads provided in Section 6.4.1 are not expected to exceed one inch and ¾-inch in 40 feet, respectively. In addition to the estimated static settlements, the Garage and western portion of the Tower site may experience between ½ and 3 inches of total seismic settlement due to a strong earthquake. If the potential for such settlement is intolerable to the design team, piles embedded at least 10-feet into dense Old Alluvium should be used to limit the total pile settlement to less than 1-inch. Additional pile recommendations may be provided during the design development phase.

Settlement of the Young Alluvium will induce a drag load on the pile foundations. This drag load has already been removed from the factored axial pile capacities provided in Figures 7A to 7D. However, the compressive force on the piles induced by the drag load should be evaluated by the structural engineer. Note that the drag load magnitude will vary depending on the pile diameter (D) and thickness (T) of compressible Young Alluvium at each pile location (in feet). Based on the average corrected SPT blow counts we measured in the Young Alluvium (N_{60}~12), the drag load (L_D) for each pile may be conservatively approximated from the ultimate skin friction using the following equation:

$$L_D \sim D^*(4.4^*T-12) \text{ [kips] for } T \geq 5 \text{ feet}$$

6.4.4 Lateral Resistance

Lateral loads against structures may be resisted by friction between the bottoms of footings and slabs and the underlying soil, as well as passive pressure from the portion of vertical foundation members embedded into compacted fill. A coefficient of friction of 0.30 and an allowable passive pressure of 300 psf per foot of depth may be used. These allowable friction and passive pressure values incorporate Safety Factors of 1.5 and 2.0 or more, respectively.

LPILE analyses are provided in Figures 8A to 8D which show the estimated lateral deflection, shear and moment diagrams for single 24-inch diameter piles at the Garage and Tower sites. Additional analyses are provided in Figures 8E to 8H which show the response of 30-inch diameter piles at the Garage and Tower sites. For these preliminary LPILE analyses, the piles were assumed to be 40-feet long and embedded at least 10-feet into dense Old Alluvium. We evaluated both free-head and fixed-head conditions for ¼, ½ and 1-inch of lateral displacement at the pile head.

Note that depending upon the ultimate pile diameter, spacing and direction of loading, group reduction factors may also apply to the lateral pile resistance. Group reduction P-multipliers for use in LPILE may be estimated in accordance with Table 10.7.2.4 of the 6^{th} Edition of the AASHTO LRFD Design Code with California Amendments. Additional P-Multipliers for use with the LPILE analyses at specific pile locations may be provided upon request.

6.4.5 Seismic Design

Structures should be designed in general accordance with the governing seismic provisions of the 2022 California Building Code (CBC) for Seismic Design Category D. Based on the shear wave velocities measured at the site, it is our opinion that a 2022 CBC Site Class C may be applied to the seismic design of both the new Garage and Tower.

General Procedure seismic design parameters were developed using the referenced online tool (ASCE, 2023). The 2022 CBC Design and MCE_G General Procedure spectra for Site Class C are identical for the two sites as shown in Tables 1 and 2. Our site-specific seismic hazard analyses are described in detail in Appendix E.

6.5 On-Grade Slabs

Concrete building slabs should be at least 5½ inches thick and should be reinforced with at least No. 4 bars on 18-inch centers, each way. Slab thickness, control joints, and reinforcement should be designed by the structural engineer and conform to the requirements of the current CBC.

6.5.1 Moisture Protection for Slabs

Concrete slabs constructed on grade ultimately cause the moisture content to rise in the underlying soil. This results from capillary rise and the termination of normal evapotranspiration. Because normal concrete is permeable, the moisture will eventually penetrate the slab. Excessive moisture may cause mildewed carpets, lifting or discoloration of floor tiles, or similar problems. To decrease the likelihood of these problems, suitable moisture protection measures should be used where moisture sensitive floor coverings, equipment, or other factors warrant. These measures may be omitted at the Garage site at the discretion of the property owner.

Common moisture barriers in southern California often consist of two to four inches of clean sand covered by 'visqueen' plastic sheeting. Two inches of sand are placed over the plastic to decrease concrete curing problems. Such systems may transmit approximately 6 to 12 pounds of moisture per 1000 square feet per day. The architect should review the estimated moisture transmission rates, since these values may be excessive for some applications, such as sheet vinyl, wood flooring, vinyl tiles, or carpeting with impermeable backings that use water soluble adhesives. Sheet vinyl may develop discoloration or adhesive degradation due to excessive moisture. Wood flooring may swell and dome if exposed to excessive moisture. The architect should specify an appropriate moisture barrier based on the allowable moisture transmission rate for the flooring. This may require a “vapor barrier” or a “vapor retarder”.

The American Concrete Institute provides detailed recommendations for moisture protection systems (ACI 302.1R-04). ACI defines a “vapor retarder” as having a minimum thickness of 10-mil, and a water transmission rate of less than 0.3 perms when tested per ASTM E96. ACI defines a “vapor barrier” as having a water transmission rate of 0.01 perms or less (such as a 15 mil StegoWrap). The vapor membrane should be constructed in accordance with ASTM E1643 and E1745 guidelines. All laps or seams should be overlapped at least 6 inches or per the manufacturer recommendations. Joints and penetrations should be sealed with pressure sensitive tape, or the manufacturer’s adhesive. The vapor membrane should be protected from puncture and repaired per the manufacturer’s recommendations if damaged.

Based on ACI recommendations, the concrete slab should be placed directly over the vapor membrane. The common practice of placing sand over the vapor membrane may increase moisture transmission through the slab, because it provides a reservoir for bleed water from the concrete to collect. The sand placed over the vapor membrane may also move during placement, resulting in an irregular slab thickness. When placing concrete on an impervious membrane, it should be noted that finishing delays may occur. Care should be taken to assure that a low water to cement ratio is used, and that the concrete is moist cured in accordance with ACI guidelines.

The vapor membrane is often placed over 4 inches of granular material, when required by the product manufacturer. The material should consist of a clean, fine graded sandy soil with roughly 10 to 30 percent passing the No. 100 sieve. The sand should not be contaminated with clay, silt, or organic material. The sand should be proof-rolled prior to placing the vapor membrane.

6.5.2 Exterior Slabs

Exterior slabs and sidewalks should be at least 4 inches thick and underlain by 2-feet of non-expansive compacted fill soil (EI<20). Control joints should be placed on a maximum spacing of 10-foot centers, each way, for slabs, and on 5-foot centers for sidewalks. The potential for differential movements across the control joints may be reduced by using steel reinforcement. Typical reinforcement would consist of 6x6 W2.9/W2.9 welded wire fabric placed securely at mid-height of the concrete slab or sidewalk.

6.5.3 Reactive Soils

In order to assess the sulfate exposure of concrete in contact with the site soils, samples were tested for pH, resistivity, water-soluble sulfate and chloride content, as shown in Figure B-3. The sulfate test results indicate that the on-site soils present a *negligible* potential for sulfate attack based on commonly accepted criteria. Type II cement may be used for this condition. The saturated resistivity of the on-site soils is indicative of a *moderately corrosive* soil with respect to buried metals. Typical corrosion control measures should be incorporated into the project design. A corrosion consultant may be contacted for specific recommendations.

6.6 Earth-Retaining Structures

Backfilling retaining walls with expansive soil can increase lateral pressures well beyond normal active or at-rest pressures. Retaining walls should be backfilled with granular soil with an Expansion Index of 20 or less (EI<20). The on-site soils generally meet this criterion.

Retaining wall backfill should be compacted to at least 90 percent relative compaction based on ASTM D1557. Backfill should not be placed until the retaining walls have achieved adequate strength. Heavy compaction equipment should not be used. For wall design, an allowable bearing capacity of 2,000 lbs/ft^2, a coefficient of friction of 0.35, and a passive pressure of 300 psf per foot of depth is recommended.

6.6.1 Shored Excavations

We understand that both temporary and permanent shored excavations will be needed to complete the proposed construction. Cantilever shoring may consist of steel soldier piles with wood lagging for temporary systems, or shotcrete facing for permanent shored walls. Typically, steel I-beams would be installed in pre-drilled 2 or 3-foot diameter holes spaced at 6 to 8 foot centers. The space between the hole and soldier beam would be filled with structural concrete up to about 6-inches below the bottom of the planned excavation. A two sack sand-cement slurry would then be used to backfill the remainder of the soldier pile excavations. Wood lagging or reinforced shotcrete would be placed between the I-beams flanges as the excavation proceeds.

Cantilever shoring may be used for excavations up to about 15 feet deep, provided that 1-inch lateral deflection at the top of the shoring is acceptable to the design team. Existing improvements located within the retained zone behind the shoring may be damaged by such lateral deformation. Deeper excavation should be achievable using one or more rows of tie-backs. However, any tie-backs located within 20-feet of existing structures of subsurface utilities should be evaluated by the design team for potential interaction on a case-by-case basis.

For design of cantilever shoring with level backfill, we recommend a triangular active pressure distribution approximated by a fluid with an equivalent unit weight of 35 lb/ft^3. For cantilever shoring that will retain 2:1 sloping backfill, we recommend assuming a triangular active pressure distribution approximated by a fluid with an equivalent unit weight of 55 lb/ft^3 (see Figure 9A). For soldier piles that are spaced at least two pile diameters on center, the allowable passive resistance below the excavation bottom may be approximated by a fluid with an equivalent unit weight of 300 lb/ft^3 above the groundwater table. Lateral pile resistance below groundwater may be provided on a case-by-case basis (if needed). The lateral resistance may be assumed to act across a width equal to two soldier pile diameters, as shown in Figures 9A and 9B.

For shored excavations more than 15-feet deep, or for any excavations in close proximity to existing structures, one or more levels of tie-backs will be needed (see Figure 9B). We recommend that tie-back shoring excavated within the existing granular soils which surround the Tower structure be designed assuming a uniform lateral earth pressure of 21H lb/ft^2 for level ground conditions above the groundwater table. Excavations in sloping areas may experience a higher uniform lateral pressure and should be evaluated on a case-by case basis. Shored excavations may also be subjected to lateral surcharge loads from existing nearby structures and foundations. Surcharge loads from existing foundations may be determined on a case-by-case basis during design development. Simplified surcharge pressure distributions (P_S) for both temporary shoring and permanent walls are provided in Figures 9A through 9D.

6.6.2 Permanent Walls

Permanent cantilever retaining walls with level granular backfill may be designed using an active earth pressure approximated by an equivalent fluid pressure of 35 lbs/ft^3 (see Figure 9C). For 2:1 sloping conditions, an equivalent fluid pressure of 55 lb/ft^3 is recommended. The active pressure should be used for walls free to yield at the top at least ½ percent of the wall height. For restrained basement walls with level backfill where such movement is not permissible, an equivalent at-rest earth pressure of 60 lbs/ft^3 is recommended as shown in Figure 9D.

Retaining walls that are located adjacent to vehicular traffic areas may be designed to resist a uniform lateral surcharge pressure of 100 lb/ft^2, resulting from a typical 300 lb/ft^2 traffic surcharge acting behind the wall. Permanent walls may also be subjected to lateral surcharge loads from existing nearby structural foundations, which should be determined on a case-by-case basis. All permanent retaining walls should contain adequate drainage to relieve the buildup of hydrostatic pressures. Typical cantilever retaining wall drainage details are shown in Figure 9E.

6.6.3 Seismic Wall Design

Per the provisions of the 2022 California Building Code (CBC), seismic design is required for all earth retaining structures over 6 feet in height. The site modified MCE_G level peak ground acceleration (PGA_M) for both the Garage and Tower sites is 0.615g, as shown in Tables 1 and 2. Design level loads are traditionally used for seismic design of retaining walls (PGA_M/1.5~0.41g), as described in Section 1803A.5.12 of the 2022 CBC. A fraction of the Design level peak ground acceleration is typically used for pseudo-static seismic wall design to account for yielding of the walls. We have provided seismic retaining wall design parameters based on a pseudo-static seismic load of 0.24g, corresponding to about 1 to 2 inches of seismic wall deformation. The recommended seismic increment of 23 lb/ft^3 for yielding retaining walls is depicted in the attached Figure 9C.

6.7 Pavement Design

For all pavement areas, upper 12 inches of subgrade soil should be scarified immediately prior to constructing the pavements, brought to optimum moisture, and compacted to at least 95 percent of the maximum density per ASTM D1557. Aggregate base should also be compacted to 95 percent relative compaction and should conform to the Standard Specifications for Public Works Construction (*SSPWC),* Section 200-2. Asphalt concrete should conform to Section 400-4 of the *SSPWC* and should be compacted to 91 and 97 percent of the Rice density per ASTM D2041.

6.7.1 Asphalt Concrete

Laboratory testing indicates that the predominately granular on-site soils have an R-Value ranging from about 27 to 40 as shown in Figures B-6.1 and B-6.2 of Appendix B. Additional R-Value samples may be collected from the actual pavement subgrade soil during earthwork construction. Asphalt concrete pavement design was conducted using the Caltrans Design Method (Topic 633.1). We anticipate that a Traffic Index (TI) ranging from 5.0 to 8.0 may apply to the proposed pavement areas. The project civil engineer should review the assumed Traffic Indices to determine where they may be applicable. Based on an estimated minimum subgrade R-Value of 20, and the assumed range of Traffic Indices, the following pavement sections apply.

PAVEMENT TYPE	TRAFFIC INDEX	ASPHALT SECTION	BASE SECTION
Passenger Car Parking	5.0	3 Inches	7 Inches
Light Truck Traffic Areas	6.0	4 Inches	8 Inches
Heavy Truck Traffic Areas	7.0	4 Inches	12 Inches
Fire Truck Traffic Areas	8.0	5 Inches	14 Inches

6.7.2 Portland Cement Concrete

Concrete pavement design was conducted in general accordance with the simplified design procedure of the Portland Cement Association. This methodology is based on a 20-year design life. For design, it was assumed that aggregate interlock would be used for load transfer across control joints. The concrete was assumed to have a minimum flexural strength of 600 psi. The subgrade materials were assumed to provide "low" support, based on the results of the R-Value tests. Based on these assumptions, we recommend that PCC pavement sections at this site consist of at least 6 inches of concrete placed over 6 inches of aggregate base. For heavier truck traffic areas (TI~7.0), we recommend 7 inches of concrete over 6 inches of aggregate base.

Crack control joints should be constructed for all PCC pavements on a maximum spacing of 10 feet, each way. Concentrated truck traffic areas should be reinforced with number 4 bars on 18-inch centers, each way.

6.8 Pipelines

The improvements will include various pipelines such as water, reclaimed water, sewer and storm drain systems. Geotechnical aspects of pipeline design include lateral earth pressures for thrust blocks, modulus of soil reaction, and pipe bedding. Each of these parameters is discussed below.

6.8.1 Thrust Blocks

Lateral resistance for thrust blocks may be determined by a passive pressure value of 300 lbs/ft^2 per foot of embedment, assuming a triangular distribution. This value may be used for thrust blocks embedded into compacted fill soils located above the groundwater table.

6.8.2 Modulus of Soil Reaction

The modulus of soil reaction (E') is used to characterize the stiffness of soil backfill placed along the sides of buried flexible pipelines. For the purpose of evaluating deflection due to the load associated with trench backfill over the pipe, a value of 1,500 lbs/in^2 is recommended for the general conditions, assuming granular bedding material is placed around the pipe.

6.8.3 Pipe Bedding

Typical pipe bedding as specified in the *Standard Specifications for Public Works Construction* may be used. We recommend that pipes be supported on at least 4 inches of granular bedding material such as minus ¾-inch crushed rock or disintegrated granite. Where pipeline or trench excavations exceed a 15 percent gradient, we do not recommend that open graded rock be used for bedding or backfill because of the potential for piping and internal erosion. For sloping utilities, we recommend that coarse sand or sand-cement slurry be used for the bedding and pipe zone. The slurry should consist of a 2-sack mix having a slump no greater than 5 inches.

7.0 LIMITATIONS

This report was prepared using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in similar localities. No warranty, express or implied, is made as to the conclusions and opinions included in this report.

The findings of this report are valid as of the present date. However, changes in the condition of a property can occur with the passage of time, whether due to natural processes or the work of man on this or adjacent properties. In addition, changes in applicable or appropriate standards of practice may occur from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report should not be relied upon after a period of three years.

8.0 REFERENCES

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TABLES

TABLE 1 - 2022 CBC GENERAL PROCEDURE ACCELERATION RESPONSE SPECTRA (TOWER)

INPUT

S_S=	1.500	g = short period (0.2 sec) mapped spectral response acceleration MCE Site Class B (**ASCE 7-16 Section 11.4.2 and Figure 22-1**)
S_1=	0.600	g = 1.0 sec period mapped spectral response acceleration MCE Site Class B (**ASCE 7-16 Section 11.4-2 and Figure 22-2**)
Site Class=	C	= Site Class definition based on **2022 California Bulding Code**
F_a=	1.200	= Site Coefficient applied to S_s to account for soil type (**ASCE 7-16 Table 11.4-1**)
F_v=	1.400	= Site Coefficient applied to S_1 to account for soil type (**ASCE 7-16 Table 11.4-2**)
T_L=	8.00	sec = Long Period Transition Period (**ASCE 7-16 Figure 11.4-1**)

Site Latitude:	33.9765
Site Longitude:	-117.3825
Seismic Design Category:	D
Site Modified Peak Ground Accelerations (PGA_M):	0.615

OUTPUT

S_{MS}=	1.800	= site class modified short period (0.2 sec) MCE spectral response acceleration = F_a x S_s (**ASCE 7-16 Equation 11.4-1**)
S_{M1}=	0.840	= site class modified 1.0 sec period MCE spectral response acceleration = F_v x S_1 (**ASCE 7-16 Equation 11.4-2**)
S_{DS}=	1.200	= site class modified short period (0.2 sec) Design spectral response acceleration = 2/3 x S_{MS} (**ASCE 7-16 Equation 11.4-3**)
S_{D1}=	0.560	= site class modified 1.0 sec period Design spectral response acceleration = 2/3 x S_{M1} (**ASCE 7-16 Equation 11.4-4**)
T_0=	0.093	sec = 0.2 S_{D1}/S_{DS} = Control Period (left end of peak) for ARS Curve (**ASCE 7-16 Section 11.4.6**)
T_S=	0.467	sec = S_{D1}/S_{DS} = Control Period (right end of peak) for ARS Curve (**ASCE 7-16 Section 11.4.6**)

SPECTRUM CALCULATION

T (seconds)	Design Sa (g)	MCE Sa (g)
0.000	0.480	0.720
0.093	1.200	1.800
0.467	1.200	1.800
0.500	1.120	1.680
0.600	0.933	1.400
0.700	0.800	1.200
0.800	0.700	1.050
0.900	0.622	0.933
1.000	0.560	0.840
1.100	0.509	0.764
1.200	0.467	0.700
1.300	0.431	0.646
1.400	0.400	0.600
1.500	0.373	0.560
1.600	0.350	0.525
1.700	0.329	0.494
1.800	0.311	0.467
1.900	0.295	0.442
2.000	0.280	0.420
2.100	0.267	0.400
2.200	0.255	0.382
2.300	0.243	0.365
2.400	0.233	0.350
2.500	0.224	0.336
2.600	0.215	0.323
2.700	0.207	0.311
2.800	0.200	0.300
2.900	0.193	0.290
3.000	0.187	0.280
3.100	0.181	0.271
3.200	0.175	0.263
3.300	0.170	0.255
3.400	0.165	0.247
3.500	0.160	0.240
3.600	0.156	0.233
3.700	0.151	0.227
3.800	0.147	0.221
3.900	0.144	0.215
4.000	0.140	0.210

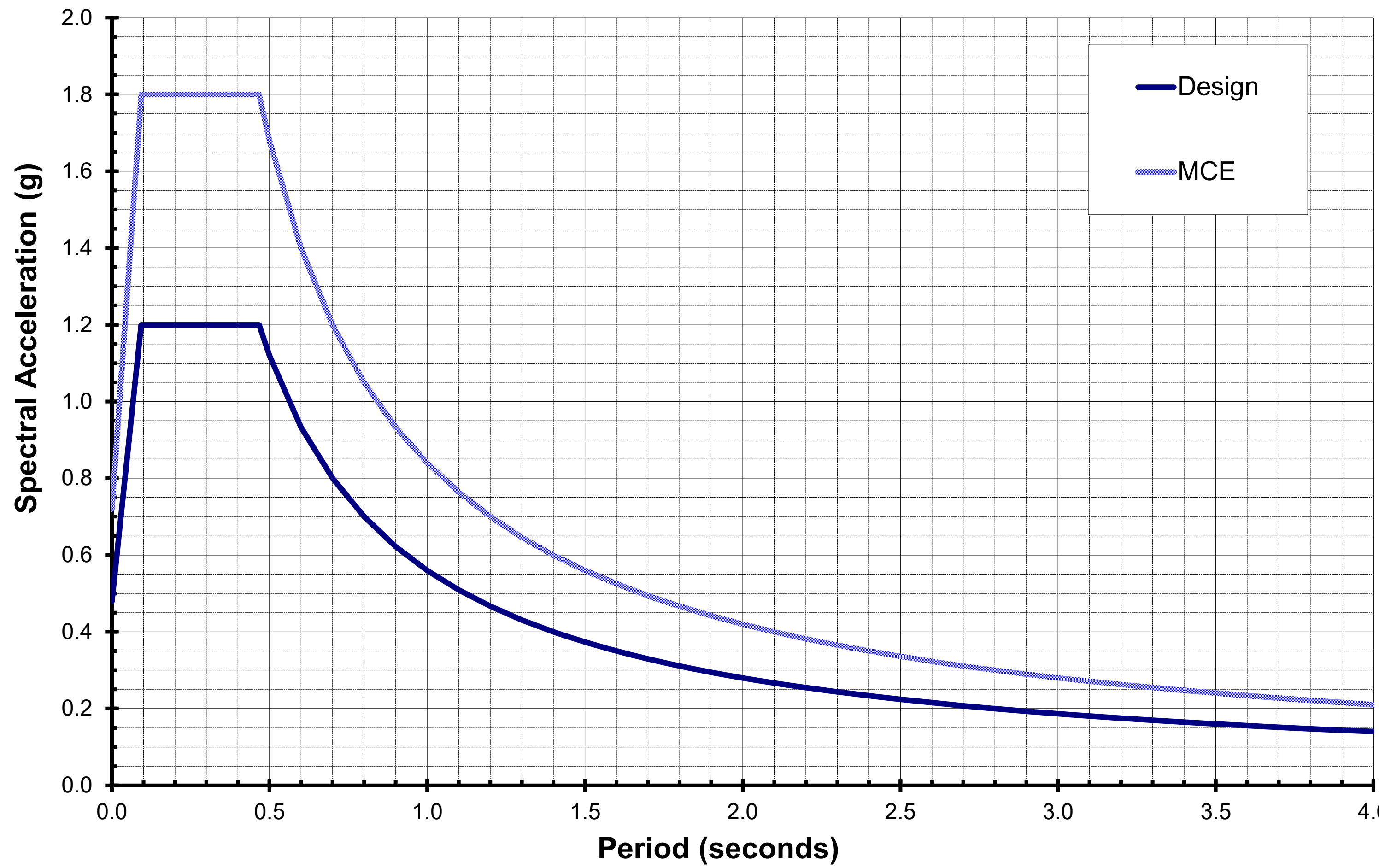


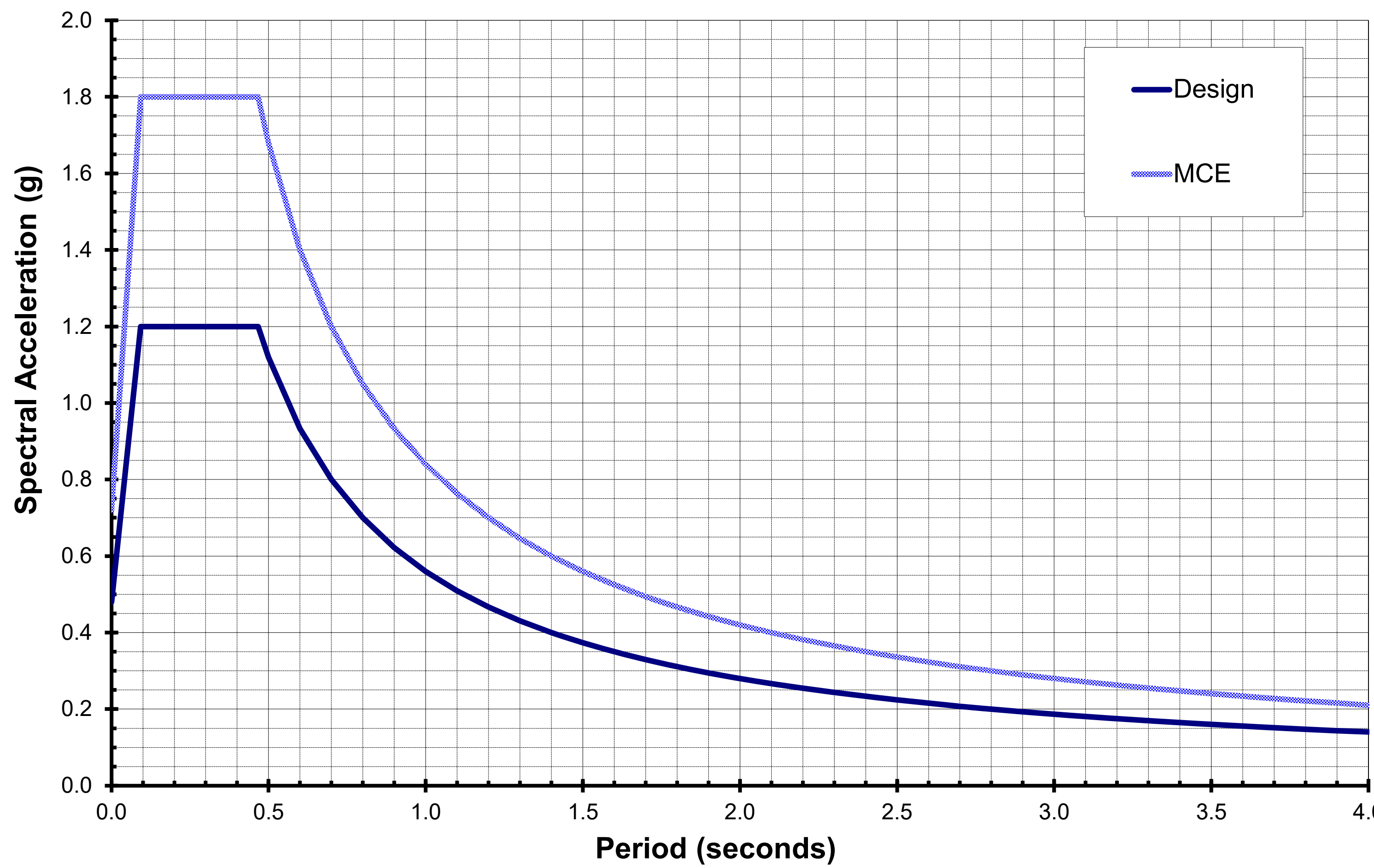
TABLE 2 - 2022 CBC GENERAL PROCEDURE ACCELERATION RESPONSE SPECTRA (GARAGE)

	Parameter	Value	Description
INPUT	S_S=	1.500	g = short period (0.2 sec) mapped spectral response acceleration MCE Site Class B (**ASCE 7-16 Section 11.4.2 and Figure 22-1**)
	S_1=	0.600	g = 1.0 sec period mapped spectral response acceleration MCE Site Class B (**ASCE 7-16 Section 11.4-2 and Figure 22-2**)
	Site Class=	C	= Site Class definition based on **2022 California Bulding Code**
	F_a=	1.200	= Site Coefficient applied to S_s to account for soil type (**ASCE 7-16 Table 11.4-1**)
	F_v=	1.400	= Site Coefficient applied to S_1 to account for soil type (**ASCE 7-16 Table 11.4-2**)
	T_L=	8.00	sec = Long Period Transition Period (**ASCE 7-16 Figure 11.4-1**)
OUTPUT	S_{MS}=	1.800	= site class modified short period (0.2 sec) MCE spectral response acceleration = F_a x S_s (**ASCE 7-16 Equation 11.4-1**)
	S_{M1}=	0.840	= site class modified 1.0 sec period MCE spectral response acceleration = F_v x S_1 (**ASCE 7-16 Equation 11.4-2**)
	S_{DS}=	1.200	= site class modified short period (0.2 sec) Design spectral response acceleration = 2/3 x S_{MS} (**ASCE 7-16 Equation 11.4-3**)
	S_{D1}=	0.560	= site class modified 1.0 sec period Design spectral response acceleration = 2/3 x S_{M1} (**ASCE 7-16 Equation 11.4-4**)
	T_0=	0.093	sec = 0.2 S_{D1}/S_{DS} = Control Period (left end of peak) for ARS Curve (**ASCE 7-16 Section 11.4.6**)
	T_S=	0.467	sec = S_{D1}/S_{DS} = Control Period (right end of peak) for ARS Curve (**ASCE 7-16 Section 11.4.6**)

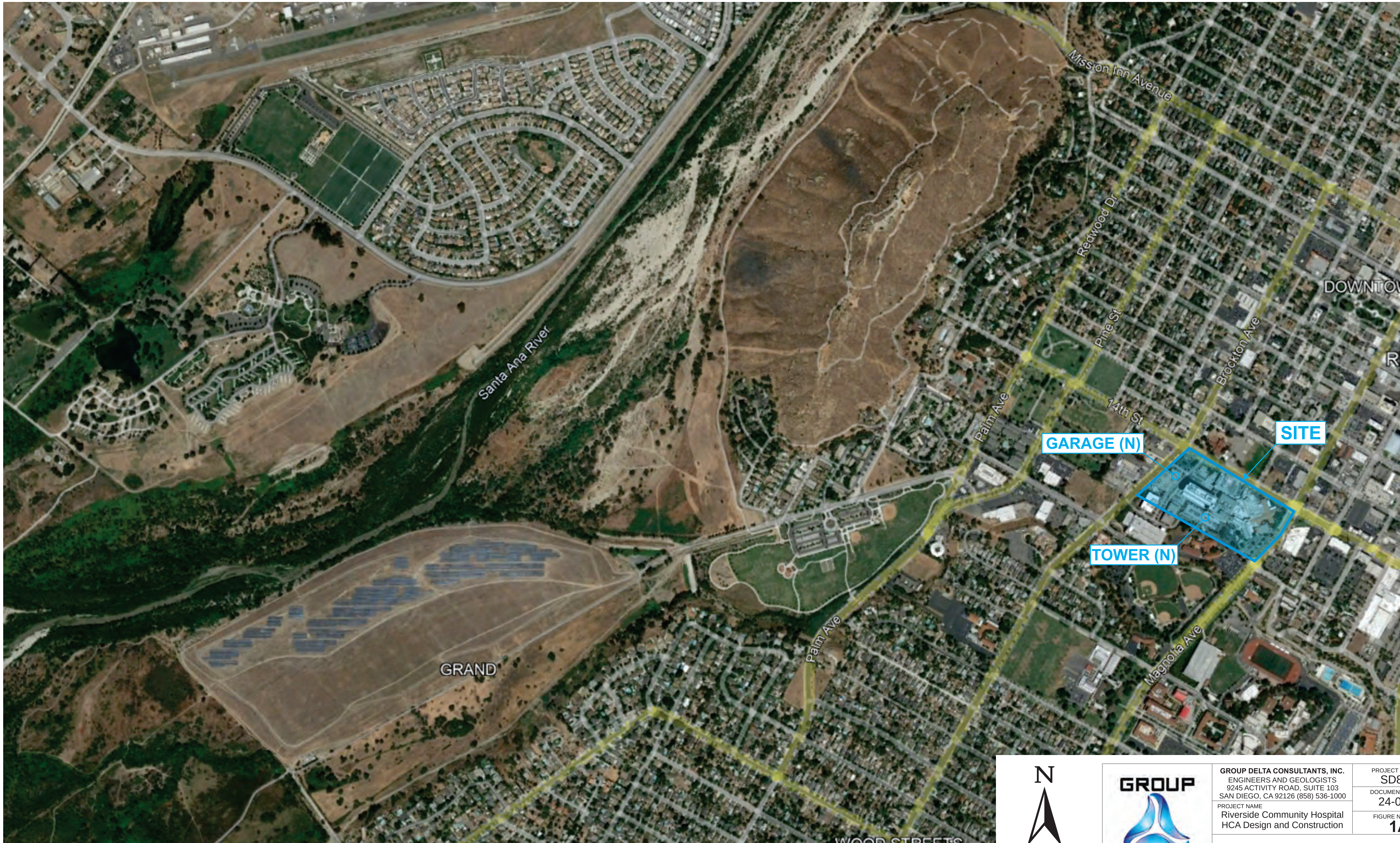
Site Latitude:	33.9776
Site Longitude:	-117.3834
Seismic Design Category:	D
Site Modified Peak Ground Accelerations (PGA_M):	0.615

SPECTRUM CALCULATION

T (seconds)	Design Sa (g)	MCE Sa (g)
0.000	0.480	0.720
0.093	1.200	1.800
0.467	1.200	1.800
0.500	1.120	1.680
0.600	0.933	1.400
0.700	0.800	1.200
0.800	0.700	1.050
0.900	0.622	0.933
1.000	0.560	0.840
1.100	0.509	0.764
1.200	0.467	0.700
1.300	0.431	0.646
1.400	0.400	0.600
1.500	0.373	0.560
1.600	0.350	0.525
1.700	0.329	0.494
1.800	0.311	0.467
1.900	0.295	0.442
2.000	0.280	0.420
2.100	0.267	0.400
2.200	0.255	0.382
2.300	0.243	0.365
2.400	0.233	0.350
2.500	0.224	0.336
2.600	0.215	0.323
2.700	0.207	0.311
2.800	0.200	0.300
2.900	0.193	0.290
3.000	0.187	0.280
3.100	0.181	0.271
3.200	0.175	0.263
3.300	0.170	0.255
3.400	0.165	0.247
3.500	0.160	0.240
3.600	0.156	0.233
3.700	0.151	0.227
3.800	0.147	0.221
3.900	0.144	0.215
4.000	0.140	0.210



FIGURES



GROUP DELTA

GROUP DELTA CONSULTANTS, INC.
ENGINEERS AND GEOLOGISTS
9245 ACTIVITY ROAD, SUITE 103
SAN DIEGO, CA 92126 (858) 536-1000

PROJECT NAME
Riverside Community Hospital
HCA Design and Construction

PROJECT NUMBER
SD809

DOCUMENT NUMBER
24-0011

FIGURE NUMBER
1A

SITE LOCATION MAP

GROUP DELTA

GROUP DELTA CONSULTANTS, INC.
ENGINEERS AND GEOLOGISTS
9245 ACTIVITY ROAD, SUITE 103
SAN DIEGO, CA 92126 (858) 536-1000

PROJECT NAME
Riverside Community Hospital
HCA Design and Construction

PROJECT NUMBER
SD809

DOCUMENT NUMBER
24-0011

FIGURE NUMBER
1B

SITE PHOTOGRAPHS
(TOWER SITE)

GROUP DELTA	GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 9245 ACTIVITY ROAD, SUITE 103 SAN DIEGO, CA 92126 (858) 536-1000	PROJECT NUMBER SD809
		DOCUMENT NUMBER 24-0011
	PROJECT NAME Riverside Community Hospital HCA Design and Construction	FIGURE NUMBER 1C
	SITE PHOTOGRAPHS (TOWER SITE)	

EXISTING PARKING LOT
(NOTE CRACKED ASPHALT
CONCRETE PAVEMENTS)

GROUP DELTA	GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 9245 ACTIVITY ROAD, SUITE 103 SAN DIEGO, CA 92126 (858) 536-1000	PROJECT NUMBER SD809 DOCUMENT NUMBER 24-0011
	PROJECT NAME Riverside Community Hospital HCA Design and Construction	FIGURE NUMBER 1D
	SITE PHOTOGRAPHS (GARAGE SITE)	

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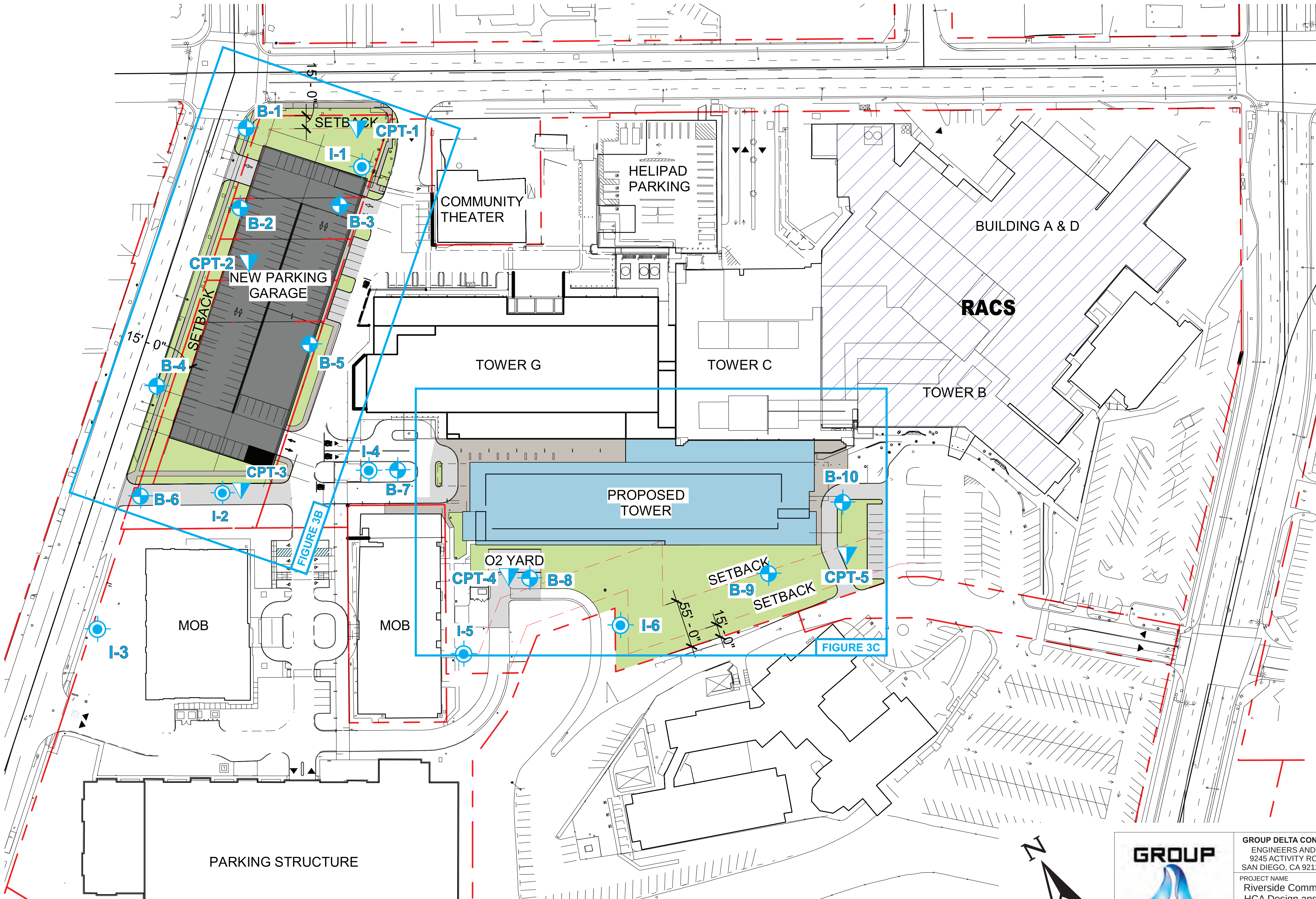
PROJECT NAME
Riverside Community Hospital
HCA Design and Construction

PROJECT NUMBER
SD809

DOCUMENT NUMBER
24-0011

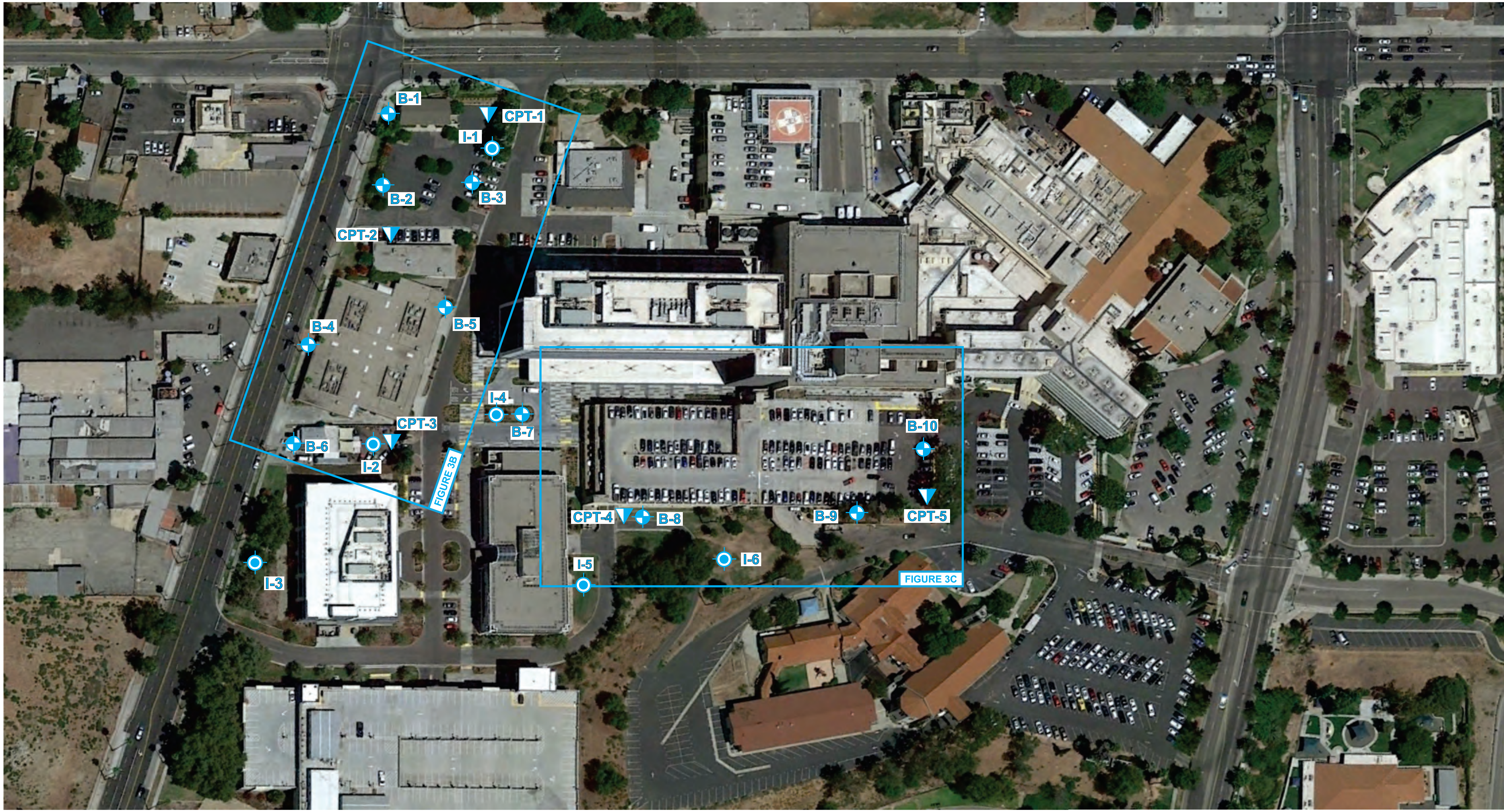
FIGURE NUMBER
1E

SITE PHOTOGRAPHS
(GARAGE SITE)



REFERENCE: HKS (2023). Riverside Commity Hospital, New Tower - Option 3, August 6.

GROUP DELTA	GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 9245 ACTIVITY ROAD, SUITE 103 SAN DIEGO, CA 92126 (858) 536-1000	PROJECT NUMBER SD809 DOCUMENT NUMBER 24-0011
	PROJECT NAME Riverside Community Hospital HCA Design and Construction	FIGURE NUMBER 2
	PROPOSED DEVELOPMENT	



EXPLANATION:

B-10 Approximate locations of the 10 supplemental hollow stem exploratory borings completed for this study (Group Delta, 2024).

CPT-5 Approximate locations of the 5 cone penetration test (CPT) soundings completed for this geotechnical investigation (Group Delta, 2024).

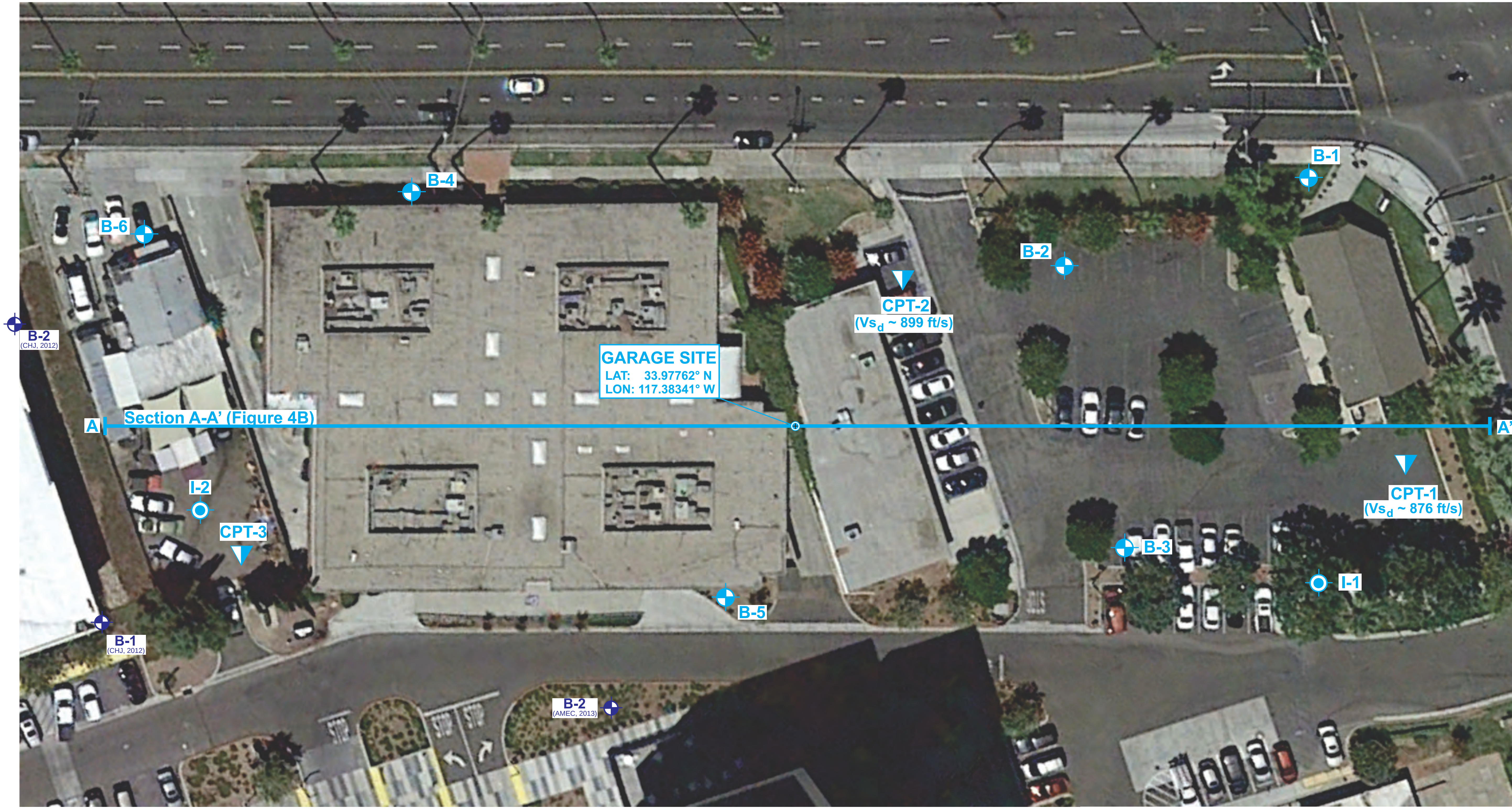
I-6 Two borehole percolation tests were performed at each of these six test locations to aid in civil BMP storm water design (Group Delta, 2024).

N

NO SCALE

GROUP DELTA

GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 9245 ACTIVITY ROAD, SUITE 103 SAN DIEGO, CA 92126 (858) 536-1000	PROJECT NUMBER SD809 DOCUMENT NUMBER 24-0011	
PROJECT NAME Riverside Community Hospital HCA Design and Construction	FIGURE NUMBER 3A	
EXPLORATION PLAN		



EXPLANATION:

B-10 — Approximate locations of the 10 exploratory borings completed for this investigation (Group Delta, 2024). Prior explorations in **dark blue**.

CPT-5 — Approximate locations of the 5 cone penetration test (CPT) soundings completed for this geotechnical investigation (Group Delta, 2024).

I-6 — Two borehole percolation tests were performed at each of the six test locations to aid in civil BMP storm water design (Group Delta, 2024).

N

NO SCALE

GROUP DELTA

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ENGINEERS AND GEOLOGISTS
9245 ACTIVITY ROAD, SUITE 103
SAN DIEGO, CA 92126 (858) 536-1000

PROJECT NAME
Riverside Community Hospital
HCA Design and Construction

PROJECT NUMBER
SD809

DOCUMENT NUMBER
24-0011

FIGURE NUMBER
3B

EXPLORATION PLAN
(GARAGE SITE)

EXPLANATION:

B-10 — Approximate locations of the 10 exploratory borings completed for this investigation (Group Delta, 2024). Prior explorations in **dark blue**.

CPT-5 — Approximate locations of the 5 cone penetration tests (CPT) completed for this investigation (Group Delta, 2024). Prior soundings in **dark blue**.

I-6 — Two borehole percolation tests were performed at each of the six test locations to aid in civil BMP storm water design (Group Delta, 2024).

N

NO SCALE

GROUP DELTA

GROUP DELTA CONSULTANTS, INC.
ENGINEERS AND GEOLOGISTS
9245 ACTIVITY ROAD, SUITE 103
SAN DIEGO, CA 92126 (858) 536-1000

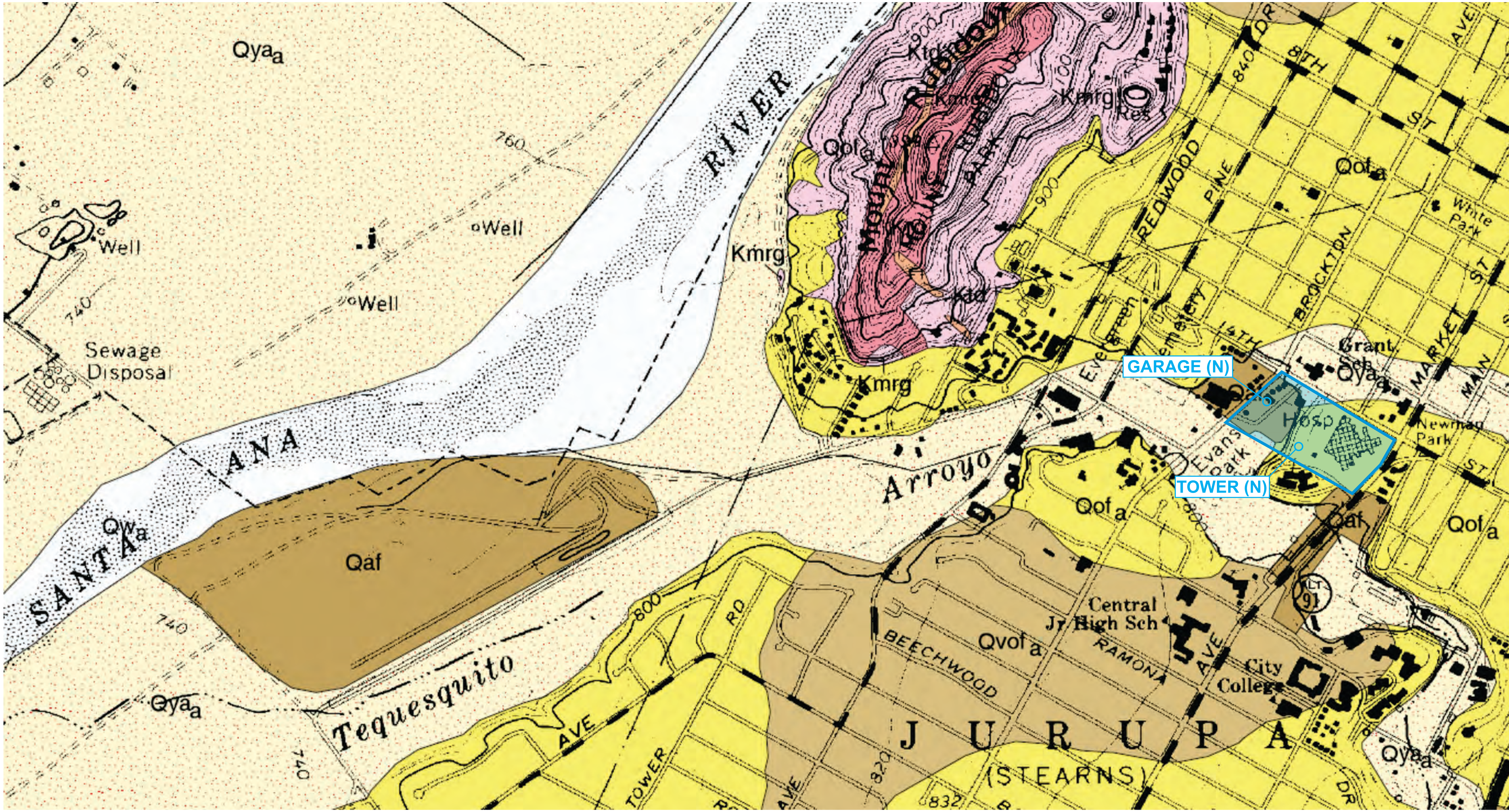
PROJECT NAME
Riverside Community Hospital
HCA Design and Construction

PROJECT NUMBER
SD809

DOCUMENT NUMBER
24-0011

FIGURE NUMBER
3C

EXPLORATION PLAN
(TOWER SITE)



EXPLANATION:

Qaf	**Artificial fill (Holocene):**	Mostly silty sand, clayey sand and sandy silt placed in previous grading operations.
Qya	**Young Alluvium (Holocene):**	Unconsolidated alluvium consisting of fine to coarse grained sand and lesser gravel and silt.
Qof	**Old Alluvium (Pleistocene):**	Slightly indurated, sandy alluvial fan deposits associated with the Santa Ana River system.

REFERENCE: Morton and Cox (2002). *Geologic Map of the Riverside West 7.5' Quadrant, Riverside, California.*

N

NO SCALE

GROUP DELTA

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9245 ACTIVITY ROAD, SUITE 103
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PROJECT NAME
Riverside Community Hospital
HCA Design and Construction

PROJECT NUMBER SD809

DOCUMENT NUMBER 24-0011

FIGURE NUMBER **4A**

LOCAL GEOLOGIC MAP