Geotechnical Investigation Report

Proposed Parking Structure
Kaiser Riverside Medical Center
10800 Magnolia Avenue
Riverside, California

Prepared for:

Kaiser Foundation Hospitals
182 Granite Street
Corona, California 92879

April 12, 2021 (Revised July 21, 2021)
Project No.: 190919.3

TWINING

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April 12, 2021 (Revised July 21, 2021)
Project No.: 190919.3

Mr. Gary Richie
Director of Facilities Planning
Kaiser Foundation Hospitals
182 Granite Street
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Subject: **Geotechnical Investigation Report**
Proposed Parking Structure
Kaiser Riverside Medical Center
10800 Magnolia Avenue
Riverside, California

Dear Mr. Richie,

In accordance with your request and authorization, we are presenting the results of our geotechnical investigation for the Proposed Parking Structure project located at 10800 Magnolia Avenue in Riverside, California. The purpose of our investigation has been to evaluate the subsurface conditions at the site, to identify seismic and geologic hazards present on the site, and to provide geotechnical engineering recommendations for the proposed development. This report was prepared in accordance with the requirements of the 2019 California Building Code (2019 CBC) and ASCE 7-16 (ASCE 2017).

Based on our findings, the proposed project is geotechnically feasible, provided that the recommendations in this report are incorporated into the design and are implemented during construction of the project.

We appreciate the opportunity to be of service on this project. Should you have any questions regarding this report or if we can be of further service, please do not hesitate to contact the undersigned.

Respectfully submitted,
TWINING, INC.

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1. INTRODUCTION

This report presents the results of the geotechnical investigation performed by Twining, Inc. (Twining) for the Proposed Parking Structure project at Kaiser Permanente Riverside Medical Center (Kaiser RMC) located at 10800 Magnolia Avenue in Riverside, California. A description of the site and the proposed improvements is provided in the following section. The objectives of this investigation have been to evaluate subsurface conditions at the site, to identify seismic and geologic hazards present on the site, and to provide geotechnical recommendations for design and construction of the proposed development, including recommendations for foundations and earthwork.

This report was prepared in accordance with the requirements of the 2019 California Building Code (2019 CBC) and ASCE 7-16 (ASCE 2017).

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

Based on our communications with Kaiser and the project architect, CO Architects, proposed improvements on the Kaiser RMC campus will be developed in seven phases. Details of each phase are provided on Sheets A1.20 through A1.24 included in Appendix D – Select Project Plans. This report is primarily focused on the proposed parking structure that is a part of Phases 3 through 5.

Based on information from Kaiser, CO Architects, and the project structural engineer, John A. Martin & Associates, the proposed parking structure will consist of five stories above grade with parking on the 5th-story deck. The structure will have a gross building footprint measuring approximately 197 feet by 386 feet and approximately 76,042 square feet. The structure will be constructed with reinforced concrete supported on shallow spread footing foundations.

Other appurtenant improvements for the project are anticipated including hardscape, light poles, utility pipelines, and a stormwater infiltration system. The size and depth of the infiltration basin are to be determined, and details of the system are not yet available for our review.

Anticipated earthwork for the new parking structure will include 5,500 cubic yards (CY) of cut, 4,000 CY of fill, and 1,500 CY of import.

The project site is located at 10800 Magnolia Avenue in Riverside, California, as shown on Figure 1 – Site Location Map. The overall site plan showing the final finished entire Kaiser RMC site is depicted on Figure 2, along with the field investigation locations performed for this report. The site is currently occupied by paved surface parking, landscaping, and associated miscellaneous equipment. The site is bounded by surface parking on the north, an existing medical office building and surface parking on the west, Park Sierra Drive on the south, and a hospital campus driveway on the east.

The approximate site coordinates are latitude 33.903295°N and longitude 117.469535°W, and the site is located on the Riverside West, California 7½-Minute Quadrangle, based on the United States Geological Survey (USGS) topographic map (USGS 2018). The site is relatively level with a surface elevation of approximately 723 feet above mean sea level (msl).

3. SCOPE OF WORK

Our scope of work included review of background information, pre-field activities and field exploration, laboratory testing, engineering analyses and report preparation. These tasks are described in the following subsections.

3.1. Literature Review

We reviewed readily available background data including proposed site improvement plans, published geologic maps, topographic maps, aerial photographs, seismic hazard maps and literature, and flood hazard maps relevant to the project site. Relevant information has been incorporated into this report.

3.2. Pre-Field Activities and Field Exploration

Before starting our exploration program, we had several site meetings and performed a geotechnical site reconnaissance to observe the general surficial conditions at the site, to select field exploration locations, and to plan field logistics including traffic control, health and safety, and timing of exploration. After exploration locations were delineated, Underground Service Alert was notified of the planned locations a minimum of 72 hours prior to excavation. We also retained GEOVision Inc. of Corona, California, a private utility locating service provider, to clear proposed boring locations of underground utility lines.

The field exploration program was conducted between December 7, 2019 and February 12, 2020. It consisted of drilling, testing, sampling, and logging of 4 exploratory hollow-stem-auger (HSA) borings (PS-1 through PS-4) and percolation testing in 2 hand-auger borings (P-1 and P-2). The HSA borings were advanced to approximate depths of 31.5 to 51.5 feet below ground surface (bgs) using a CME-75 truck-mounted drill rigs equipped with 8-inch-diameter HSAs. The hand-auger borings for percolation testing were drilled to approximately 6 and 6.5 feet bgs, respectively. The approximate locations of the borings are shown on Figure 2.

Drive samples of the soils were obtained from the HSA borings using a Standard Penetration Test (SPT) sampler without liners and a modified California split spoon sampler. The samplers were driven using a 140-pound automatic hammer falling approximately 30 inches. The blow-counts to drive the samplers were recorded, and subsurface conditions encountered in the borings were logged by a Twining field engineer. Soil samples obtained from the borings were transported to Twining's geotechnical engineering laboratory for examination and testing.

Percolation tests in hand-auger borings P-1 and P-2 were performed according to the boring percolation test guidance provided in the Riverside County Design Handbook for Low Impact Development Best Management Practices. Testing was performed to provide estimates of infiltration rate of the site soils for use in preliminary design of the stormwater infiltration system.

Upon completion of drilling or percolation testing, the borings were backfilled by the drilling subcontractor using drilled soil cuttings, and the surface was repaired to match existing conditions.

Detailed descriptions of the borings, soils encountered during drilling, and the percolation tests are presented in Appendix A – Field Exploration.

3.3. Geotechnical Laboratory Testing

Laboratory tests were performed on selected samples obtained from the borings to aid in the soil classification and to evaluate the engineering properties of site soils. The following tests were performed in general accordance with ASTM standards:

- In-situ moisture and density;
- #200 Wash;
- Atterberg Limits;
- Expansion Index;
- Maximum density and optimum moisture;
- Consolidation;
- Direct shear;
- Unconfined compression;
- R-Value; and
- Corrosivity.

Detailed laboratory test procedures and results are presented in Appendix B – Laboratory Testing.

3.4. Engineering Analyses and Report Preparation

We compiled and analyzed the data collected from our field exploration and laboratory testing. We performed engineering analyses based on our literature review and data from field exploration and laboratory testing programs. Our analyses included the following:

- Site geology and subsurface conditions;
- Groundwater conditions;
- Geologic hazards and seismic design parameters;
- Liquefaction potential and seismic settlement;
- Soil corrosion potential;
- Soil collapse and expansion potential;
- Site preparation and earthwork;
- Temporary excavations;
- Project feasibility and suitability of on-site soils for foundation support;
- Foundation design parameters including bearing capacity, settlement, and lateral resistance;
- Modulus of subgrade reaction for concrete slab-on-grade design;
- Lateral earth pressures for retaining wall and shoring design;
- Concrete slab-on-grade support;

- Pavement section recommendations; and
- Stormwater infiltration rates.

We prepared this report to present our conclusions and recommendations from this investigation.

4. GEOLOGY AND SUBSURFACE CONDITIONS

4.1. Site Geology

According to the geologic mapping compiled by the California Geological Survey (CGS, 2012), the project site is underlain by Late to Middle Pleistocene Old Alluvium Fan deposits (Geologic Symbol Qof) consisting of slightly to moderately consolidated, moderately dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon. A portion of the geologic map is reproduced as Figure 3 – Regional Geologic Map.

4.2. Surface and Subsurface Conditions

The site is currently occupied by an asphalt-paved parking lot and each of the four HSA borings was performed within the pavement area. The pavement section consisted of approximately 3 to 4.5 inches of asphaltic concrete over 2.5 to 5 inches of aggregate base. Below the pavement section, boring PS-1 encountered a second layer asphaltic concrete approximately 2 inches in thickness underlying approximately 4 inches of soil.

No artificial fill was identified in our borings. No documentation for the placement and compaction of the artificial fill was available for our review. The surficial material consisted of reddish brown sandy lean clay. No evidence of previous agricultural activity was identified in the surficial material. Therefore, it is probable that surficial improvement of existing soil was performed prior to placement of Portland cement concrete, asphalt concrete and base.

Pleistocene-aged old alluvial fan deposit underlies the site to the maximum depth of the exploratory borings (approximately 51.5 feet bgs). In the upper 30 feet bgs, the old alluvium consisted primarily of reddish brown to light brown clay and silt. From 30 to 51.5 feet bgs, the old alluvium consisted primarily of poorly graded sand with silt. Locally, a sand layer was encountered between depths of 20 and 25 feet in boring PS-1 and between depths of 15 to 25 feet in boring PS-2. The clay and silt layers have a very stiff consistency, and the sand layers are mostly dense to very dense and occasionally medium dense.

Our geotechnical borings excavated at the site did not encounter bedrock. Based on the California Geological Survey (Morton & Cox, 2001), Cretaceous Gabbro (crystalline bedrock) is anticipated to be at a depth of approximately 300 feet in the vicinity of the site.

4.3. Groundwater Conditions

Groundwater was not encountered during our field exploration to a maximum depth of approximately 51.5 feet bgs. Historically high groundwater level in the vicinity of the project site is not available from CGS. We reviewed groundwater level data from Metropolitan Water District of Southern California (MWDSC), Western Municipal Water District (WMWD) and California Department of Water Resources (CDWR). According to MWDSC (2007) and WMWD (2012) maps, the project site is within the southern portion of the Arlington Basin, which is the portion of the Riverside-Arlington Groundwater Subbasin (Subbasin Number 8.2-03), as defined by the CDWR Bulletin 118-03

(CDWR, 2003). Groundwater occurrence in the Arlington Basin is generally unconfined. Groundwater flow is generally toward the southwest in the southern portion, which agrees well with groundwater levels measured in the three wells adjacent to the project site. The well locations with respect to the project site are shown on Figure 3. Historical water levels in the wells from 1960 to 2019 from WMWD (2012) and CDWR (2019) are shown in Figure 4 and summarized in Table 1. Historic high groundwater data described in WMWD (2012) indicate that pre-development groundwater elevation of approximately 705 feet above msl in the vicinity of the site.

As shown on Figure 4, the highest water levels in wells Buchanan 1 and Hole 1, located northeast and southwest of the site, respectively, occurred between 1984 and 1996. Figure 4 further indicates that the groundwater level in the area dropped continuously in the past 25 to 35 years since the highest water level year. The drops in wells Buchanan 1 and Hole 1 are approximately 50 and 40 feet, respectively. Well Daly 2 is the closest known well to the site, but no data was available before November 2011. However, it may be estimated that the highest water level in Daly 2 is between 701.5 and 711.5 feet above msl assuming a similar water level drop of 40 to 50 feet. Because the closest distance is only about 100 feet from the project site to Daly 2 which is on the south of the site, the highest water level of the site is also between 701.5 and 711.5 feet above msl.

During a recent field investigation that we performed for the proposed tower project at the northwest portion of the hospital campus, groundwater was encountered at approximately 57.5 feet bgs, and the historic high was estimated at 710.5 feet above msl. That investigation was performed at the same time as this current one, and that site was at approximately 900 feet upstream of the current project site.

Table 1 – Water Levels in Wells Adjacent to the Site

Local Well ID	Buchanan 1	Daly 2	Hole 1
California State Well Number	03S06W22K004S	03S06W13N002S	03S06W13B001S
Latitude (degrees)	33.893033	33.902396	33.91527
Longitude (degrees)	-117.495077	-117.469653	-117.457580
Approximate Location relative to Project Site	8,600 ft southwest of site	100 ft south of site	5,100 ft northeast of site
Approximate Highest Water Level Since 1960 (feet, msl)	678 in 1996	Estimated between 701.5 and 711.5	736.5 in 1984
Approximate Water Level at end of 2019 (feet, msl)	628	661.5	696.5
Water Level Drop from the Highest to the end of 2019 (feet)	50	Estimated between 40 and 50	40

Additionally, GEOBASE, Inc., (2012) reported the highest groundwater elevation since 1978 is 712.2 feet above msl in a well approximately 900 feet upstream of the current project site. It occurred in May 1979, but there is no data in other years available for our review.

It should be noted that groundwater conditions may vary across the site due to stratigraphic and hydrologic conditions and may change over time as a consequence of seasonal and meteorological fluctuations, or of activities by humans at this and nearby sites. For the purpose of this investigation, the historically highest groundwater for this project site can be assumed at approximately 710.5 feet above msl or 12.5 feet bgs.

5. GEOLOGIC HAZARDS AND SEISMIC DESIGN CONSIDERATIONS

The site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion in the project area is considered high during the design life of the proposed development. The hazards associated with seismic activity in the vicinity of the site area discussed in the following sections.

5.1. Surface Fault Rupture

The site is not located within or adjacent to an Alquist-Priolo Earthquake Fault Zone (EFZ) (CGS 2016). The boundary of the closest Alquist-Priolo EFZ is located approximately 6.9 miles (11.1 kilometers) southwest of the site associated with the Elsinore fault (Figure 5). Figure 6 shows the locations of the recognized nearby faults with respect to the site. The City of Riverside (2018) and the County of Riverside (2019) do not identify any additional hazardous faults in the immediate site vicinity.

5.2. Liquefaction Potential and Seismic Settlement

Liquefaction is the phenomenon in which loosely deposited granular soils with silt and clay contents of less than approximately 35 percent, and non-plastic silts located below the water table undergo rapid loss of shear strength when subjected to strong earthquake-induced ground shaking. Ground shaking of sufficient duration results in the loss of grain-to-grain contact due to a rapid rise in pore water pressure and causes the soil to behave as a fluid for a short period of time.

Seismic settlement can occur when loose to medium dense granular materials densify during seismic shaking and liquefaction. Seismically-induced settlement may occur in dry, unsaturated, as well as saturated soils. Liquefaction is generally known to occur in loose, saturated, relatively clean, fine-grained cohesionless soils at depths shallower than approximately 50 feet. Factors to consider in the evaluation of soil liquefaction potential include groundwater conditions, soil type, grain size distribution, relative density, degree of saturation, and both the intensity and duration of ground motion. Other phenomena associated with soil liquefaction include sand boils, ground oscillation, and loss of foundation bearing capacity.

Seismic settlement can occur when medium dense granular materials densify during seismic shaking and/or liquefaction. Seismically-induced settlement may occur in dry, unsaturated, as well as saturated soils.

The area of the project site has not been evaluated for liquefaction by CGS. According to the liquefaction zones map in the General Plan 2025 of the City of Riverside, the site has moderate to high liquefaction potential (Figure 7).

We performed site-specific liquefaction analysis for non-plastic and low plasticity alluvium layers that are susceptible to liquefaction at the site. The analysis was performed based on SPT blowcounts from the HSA borings (PS-12 through PS-4) using the computer program LiqSVs version 2.0 (Geologismiki, 2019) and the procedure of Boulanger and Idriss (2014). The analysis considered an Mw 7.7 associated with a full rupture of the Elsinore Fault Zone, a PGA_M 0.61g discussed in Section 5.9, and a groundwater level at 12.5 feet bgs during earthquake discussed in Section 4.3. Detailed input parameters and results of the liquefaction analyses are presented in Appendix C of this report.

The analysis results indicate that site soils have very low liquefaction potential except for a layer in boring PS-1 between approximately depths of 30 and 40 feet. Calculated maximum seismic settlement is approximately 2 inches. The results suggest that during strong earthquake events, liquefaction and seismic settlement would be limited to localized zones below 30 feet bgs if they were to occur, due to the lack of horizontally continuous liquefiable layers and the presence of 30 feet of overlying predominantly cohesive soils. The results indicate that the maximum seismic settlement is approximately 2 inches at the parking structure site. Based on the calculated total settlements and measured distances between borings, the maximum calculated differential settlement is approximately 0.6 inches over a horizontal distance of 50 feet.

5.3. Lateral Spread

The potential of liquefaction-induced lateral spread at the site is considered remote because the site has low liquefaction potential, does not have a sloping ground, and is not adjacent to a slope.

5.4. Landslides

The area of the project site is not within an area with the potential for earthquake-induced landslides. Considering the site is relatively flat and not close to significant slopes, the potential for earthquake-induced landslides to occur at the site is considered very low.

5.5. Flooding and Dam Inundation

According to the Flood Hazard Areas map (Figure 8) in the General Plan 2025 of the City of Riverside, the site is not located within a 100- or 500-year floodplain. However, the site is located within the inundation area associated with incidents and failures of the Harrison dam and the Mockingbird Canyon dam. It is further noted that the site is not located within the inundation area of Lake Mathews (Figure 8).

According to the flood insurance rate maps (FIRMs) of the Federal Emergency Management Agency (FEMA) for use in administering the National Flood Insurance Program, the site is located within Zone X, which is described as “Areas of 0.2% annual chance flood hazard; areas of 1% annual chance flood with average depths of less than 1 foot or drainage areas less than 1 square mile.” A portion of the FEMA flood map is reproduced in Figure 9.

5.6. Tsunamis and Seiches

Tsunamis are waves generated by massive landslides near or under sea water. The potential for the site to be adversely impacted by earthquake-induced tsunamis is considered to be remote because the site is not within the official tsunami inundation area mapped by California and the site is located tens of miles inland from the Pacific Ocean coast and has an approximate ground surface elevation of 723 feet above msl that exceeds the maximum height of potential tsunami inundation in California (USGS 2013).

Seiches are standing wave oscillations of an enclosed water body (e.g., a lake, reservoir, or bay) after the original driving force has dissipated. Resulting oscillation could cause waves up to tens of feet high, which in turn could cause extensive damage along the shoreline. The most serious consequences of a seiche would be the overtopping and failure of a dam. The potential for the site to be adversely impacted by earthquake-induced seiches would be associated with the potential of seiche-induced failure of the Harrison dam or the Mockingbird Canyon dam, because the site is within the inundation area of the two dams.

5.7. Deaggregated Seismic Source Parameters

We performed a seismic hazard de-aggregation analysis for the peak ground acceleration with a probability of exceedance of 2% in 50 years. The analysis used the USGS Unified Hazard Tool based on the 2014 USGS seismic source model. The results of the analysis indicate the controlling modal moment magnitude and fault distance are 6.47 Mw and 7.45 miles (11.99 km), respectively.

5.8. Site Class for Seismic Design

Based on the 2006 CGS Site Classification Map, the average seismic shear-wave velocity for the upper 100 feet or 30 meters (V_{S30}) at the site is approximately 387 m/s or 1,270 ft/s. Based on global V_{S30} from topographic slope (Wald & Allen 2008), the site V_{S30} is approximately 300 m/s or 984 ft/s.

A geophysical study was performed for the proposed tower project at the northwest corner of the hospital campus approximately 900 feet northwest of this site. The study obtained a V_{S30} value of approximately 348 m/s or 1,143 ft/s.

Based on the above V_{S30} values and the site subsurface conditions (Section 4.2 and Appendix A), we recommend a V_{S30} value of 348 m/s and Site Class D for the project seismic design, in accordance with Chapter 20 of ASCE 7-16.

5.9. Mapped CBC Seismic Design Parameters

Our recommendations for seismic design parameters have been developed in accordance with the 2019 CBC and ASCE 7-16 (ASCE 2017) standards. As the site is classified as seismic Site Class D and the mapped spectral acceleration parameter at period 1-second, S_1, is greater than 0.2 g, a site-specific ground motion hazard analysis is required according to Section 11.4.7 of ASCE 7-16.

As an alternative, Exception 2 in Section 11.4.8 of ASCE 7-16 may be used for the project. For structural design based on this exception, Table 2 presents the seismic design parameters for the site based on coordinates of latitude 33.903295°N and longitude 117.469535°W.

The site-specific ground motion hazard analysis and seismic design parameters are presented in Section 5.10.

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Table 2 – 2019 California Building Code Seismic Design Parameters for Design Based on Exception 2 in Section 11.4.8 of ASCE 7-16

Design Parameters	Value
Site Class	D
Mapped Spectral Acceleration Parameter at Short Periods, S_s (g)	1.5
Mapped Spectral Acceleration Parameter at a Period of 1-Second, S_1 (g)	0.582
Site Coefficient, F_a	1
Site Coefficient, F_v	1.718
Adjusted MCE_R[1] Spectral Response Acceleration Parameter, S_{MS} (g)	1.500
Adjusted MCE_R[1] Spectral Response Acceleration Parameter, S_{M1} (g)	1.0
Design Spectral Response Acceleration Parameter, S_{DS} (g)	1.0
Design Spectral Response Acceleration Parameter, S_{D1} (g)	0.667
Risk Coefficient, C_{RS}	0.943
Risk Coefficient, C_{R1}	0.921
Peak Ground Acceleration (PGA) for Liquefaction Analysis, PGA_M (g) [2]	0.606
Seismic Design Category[3]	D
Long-Period Transition Period, T_L (seconds)	8
$T_s = S_{D1} / S_{DS}$	0.667

When using the above parameters for seismic design, the seismic design coefficient C_s should be calculated as follows:

For $T \leq 1.5T_s$, $C_s = S_{DS}/(R/I_e)$

For $T_L \geq T > 1.5T_s$, $C_s = 1.5\ S_{D1}/(T\ R/I_e)$

For $T > T_L$, $C_s = 1.5\ (S_{D1}\ T_L)/(T^2\ R/I_e)$

where:
T = the fundamental period of the structure(s) determined in Section 12.8.2 of ASCE 7-16;
R = the response modification factor determined in Table 12.2-1 of ASCE 7-16; and
I_e = the importance factor determined in accordance with Section 11.5.1 of ASCE 7-16.

Notes: [1] Risk-Targeted Maximum Considered Earthquake.
[2] PGA_M is PGA adjusted for site effects for liquefaction analysis.
[3] For S_1 greater than or equal to 0.75 g, the Seismic Design Category is E for risk category I, II, and III structures and F for risk category IV structures.

5.10. Site-Specific Seismic Design Parameters

The site-specific specific seismic design parameters were developed based on a site-specific ground motion hazard analysis. The analysis was performed in accordance with Section 21.2 of ASCE 7-16 based on a 2% probability of exceedance in 50 years. To develop the site-specific design response spectrum, we performed probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA) to compute the risk-targeted maximum considered earthquake (MCE_R) response accelerations. Our PSHA and DSHA used four NGA-West2 ground motion prediction equations (GMPEs) developed by Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014), and Chiou and Youngs (2014), respectively. The analyses were based on the Uniform California Earthquake Rupture Forecast Version 3 (UCERF3) developed by the Working Group on California Earthquake Probabilities (WGCEP). UCERF3 is the California portion of the 2014 USGS national seismic source model (Petersen et al. 2014). Our analyses included treatment of maximum direction spectra and adjustment for risk targeting.

The analyses were performed using a V_{S30} value of 348 m/sec and site coordinates of latitude 33.903295°N and longitude 117.469535°W. The site-specific design response spectrum is presented in Figure 10 – Site-Specific Design Response Spectrum, along with the MCE_R ground motions from our PSHA and DSHA. The detailed analysis description and results are presented below.

5.10.1. Probabilistic Seismic Hazard Analysis

A site-specific PSHA was performed to evaluate probabilistic MCE_R ground motions. The probabilistic spectral response accelerations are taken as the spectral response accelerations in the direction of maximum horizontal response represented by a 5% damped acceleration response spectrum that is expected to achieve a 1% probability of collapse within a 50-year period. In this report, ordinates of the probabilistic ground motion response spectrum were determined by Method 1 of Section 21.2.1.1 of ASCE 7-16.

The PSHA was first performed using the Hazard Spectrum Calculator by OpenSHA.org (http://www.opensha.org/apps-HazardSpectrumLocal) to obtain an average spectrum of the geometric-mean acceleration response spectra from the four NGA-West2 GMPEs. The spectra were calculated for 5-percent damped and a 2 percent probability of exceedance within a 50-year period. The average spectrum was converted to the maximum response ground motion using scale factors described in Section 21.2 of ASCE 7-16. The scale factors are 1.1 for spectral response periods less than or equal to 0.2 s, 1.3 for a period of 1.0 s, 1.5 for periods greater than or equal to 5.0 s, and between these periods are obtained by linear interpolation. The maximum response ground motion was then multiplied by a risk coefficient C_R to obtain the probabilistic MCE_R ground motion response spectrum. The values of C_R are C_{RS} for periods less than or equal to 0.2 s and C_{R1} for periods greater than or equal to 1.0 s. For periods between periods 0.2 s and 1.0 s, C_R is based on linear interpolation of C_{RS} and C_{R1}. The values of C_{RS} and C_{R1} for this project are presented in Table 2.

5.10.2. Deterministic Seismic Hazard Analysis

A site-specific DSHA was performed to evaluate the deterministic MCE_R ground motions. The deterministic MCE_R response acceleration at specified periods was calculated as the 84th

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percentile of the maximum rotated component of ground motion computed at each period for characteristic earthquakes on known active faults within the region.

The active faults and their parameters used in our DSHA are provided in Table 3, obtained from the Caltrans ARS Online Tool version 2.3.09 (http://dap3.dot.ca.gov/ARS_Online/index.php). The DSHA was performed for each fault to obtain the 5-percent-damped deterministic pseudo-absolute acceleration response spectrum using the four NGA-West2 GMPEs implemented in a Microsoft Excel spreadsheet available from the Pacific Earthquake Engineering Research Center (https://peer.berkeley.edu/research/data-sciences/databases).

Table 3 - Seismic Source Parameters

Fault Name	Elsinore (Glen Ivy) rev	San Jacinto (San Bernardino)	San Jacinto (San Bernardino Valley section)	Elsinore fault zone (Chino section)	Elsinore (Temecula)	San Andreas (San Bernardino S)
Fault ID	365	336	310	355	378	325
Slip Sense	Strike-Slip	Strike-Slip	Strike-Slip	Strike-Slip	Strike-Slip	Strike-Slip
M_w	7.7	7.7	7.7	6.6	7.7	7.9
Dip, (deg)	90	90	90	50	90	90
Z_{TOR} (km)	0	0	0	0	0	0
Z_{BOT}, (km)	13	16	15	9.2	14	12.8
W (km)	13	16	15	12	14	12.8
R_{RUP} (km)	12.68	22.23	23.96	13.1	27.17	34.89
R_{JB} (km)	12.68	22.23	23.96	13.1	27.17	34.89
R_X (km)	12.68	22.23	23.96	13.0	18.69	34.89
F_{NM}	0	0	0	0	0	0
F_{RV}	0	0	0	0	0	0

Notes:
M_w = Moment magnitude.
Z_{TOR} = The depth to the top of the rupture plane.
Z_{BOT} = The depth to the bottom of the rupture plane.
W = Fault rupture width.
R_{RUP} = Closest distance to coseismic rupture.
R_{JB} = Closest distance to surface projection of coseismic rupture.
R_X = Horizontal distance from top of rupture measured perpendicular to fault strike.
F_{RV} = Reverse-faulting factor: 0 for strike-slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust.
F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal.

The resulting 84th percentile geometric-mean acceleration response spectra for the earthquakes were used to develop a deterministic response spectrum based on the greatest spectral acceleration at each period, and then converted into maximum rotated components of ground motion using the scale factors described in Section 21.2 of ASCE 7-16 as discussed in Section 5.10.1 of this report. The final deterministic MCE_R is taken as the maximum rotated deterministic response spectrum scaled by a single factor equal to the greater of $1.5F_a/S_{a,max,max}$ and 1, where $S_{a,max,max}$ is the maximum spectral acceleration of the maximum rotated deterministic response spectrum, and F_a is determined to be 1 using Table 11.4.1 of ASCE 7-16.

5.10.3. Site-Specific Design Response Spectrum

The site-specific MCE_R spectral response acceleration was calculated at each period to be the lesser of the spectral response accelerations from the probabilistic and deterministic MCE_R, but not less than 1.5 times 80 percent of the spectral acceleration evaluated in accordance with Sections 11.4.6 and 21.3 of ASCE 7-16. In order to calculate the 80 percent of the spectral acceleration, values of S_{DS}, S_{D1} and the design spectrum were calculated using the mapped values presented in Table 2 , except that S_{M1} and S_{D1} at this step were based on an F_v value of 2.5, in accordance with Section 21.3 of ASCE 7-16.

Finally, the site-specific design spectral response acceleration at each period was calculated as two-thirds of the site-specific MCE_R spectral acceleration. The site-specific design response spectrum and relevant response spectral data are presented in Table 4 and on Figure 10 – Site-Specific Design Response Spectrum.

Table 4 - Site-Specific Design Response Spectrum Data

Period T (sec)	General Procedure Design Response Spectrum for Exception 2 of ASCE 7-16 (g)	Risk Coefficient C_R	Site-Specific Ground Motion Analysis Spectral Accelerations (g)						
			Maximum direction 2%-in-50-years Probabilistic Spectrum	Probabilistic MCE_R	Maximum direction 84th-percentile Deterministic Spectrum	Deterministic MCE_R adjusted with Scale Factor of 1	80% General Procedure Design Response Spectrum with Fv=2.5	Site Specific MCE_R	Site-Specific Design Response Spectrum
0.01	0.445	0.943	0.852	0.803	0.651	0.651	0.345	0.651	0.434
0.02	0.490	0.943	0.858	0.809	0.654	0.654	0.369	0.654	0.436
0.03	0.535	0.943	0.906	0.854	0.679	0.679	0.394	0.679	0.452
0.05	0.625	0.943	1.084	1.022	0.775	0.775	0.444	0.775	0.517
0.075	0.738	0.943	1.379	1.300	0.940	0.940	0.506	0.940	0.627
0.1	0.850	0.943	1.625	1.533	1.092	1.092	0.567	1.092	0.728
0.133	1.000	0.943	1.817	1.713	1.246	1.246	0.650	1.246	0.831
0.15	1.000	0.943	1.912	1.803	1.323	1.323	0.691	1.323	0.882
0.194	1.000	0.943	2.025	1.909	1.450	1.450	0.800	1.450	0.966
0.2	1.000	0.943	2.040	1.924	1.467	1.467	0.800	1.467	0.978
0.25	1.000	0.942	2.097	1.975	1.565	1.565	0.800	1.565	1.043
0.3	1.000	0.940	2.114	1.988	1.679	1.679	0.800	1.679	1.119
0.4	1.000	0.938	2.017	1.891	1.761	1.761	0.800	1.761	1.174
0.5	1.000	0.935	1.889	1.766	1.803	1.803	0.800	1.766	1.177
0.667	1.000	0.930	1.636	1.521	1.696	1.696	0.800	1.521	1.014
0.75	0.889	0.928	1.509	1.400	1.642	1.642	0.800	1.400	0.933
0.9	0.741	0.924	1.348	1.245	1.582	1.582	0.800	1.245	0.830
0.97	0.687	0.922	1.273	1.173	1.554	1.554	0.800	1.200	0.800
1	0.667	0.921	1.241	1.143	1.542	1.542	0.776	1.164	0.776
1.5	0.444	0.921	0.822	0.757	1.219	1.219	0.517	0.776	0.517
2	0.333	0.921	0.605	0.557	1.005	1.005	0.388	0.582	0.388
3	0.222	0.921	0.402	0.370	0.773	0.773	0.259	0.388	0.259
4	0.167	0.921	0.300	0.277	0.613	0.613	0.194	0.291	0.194
5	0.133	0.921	0.248	0.229	0.489	0.489	0.155	0.233	0.155

5.10.4. Site-Specific Seismic Design Parameters

The site-specific seismic design parameters are provided in Table 5. These parameters were determined from the site-specific design response spectrum presented in Table 4 following Section 21.4 of ASCE 7-16.

It should be noted that for use with the equivalent lateral force procedure in structural design, the site specific design spectral acceleration, Sa (the last column in Table 4 of this report), at T may replace S_{D1}/T and $S_{D1}T_L/T^2$ in ASCE 7-16 Eqs. (12.8-3) and (12.8-4), respectively. The site-specific seismic design parameter S_{DS} shown in Table 5 of this report may be used in ASCE 7-16 Eqs. (12.8-2), (12.8-5), (15.4-1), and (15.4-3). The mapped value of S_1 in Table 2 of this report should be used in ASCE 7-16 Eqs. (12.8-6), (15.4-2), and (15.4-4).

Table 5 - Site-Specific Seismic Design Parameters

Site-Specific Seismic Design Parameters	Design Values (g)
Spectral Response Acceleration 0.2-second period, S_{MS}	1.589
Spectral Response Acceleration 1-second period, S_{M1}	1.164
Design Spectral Response Acceleration for short period, S_{DS}	1.060
Design Spectral Response Acceleration for 1-second period, S_{D1}	0.776
MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA_M	0.592

6. GEOTECHNICAL ENGINEERING RECOMMENDATIONS

Based on the results of our literature review and the field exploration, laboratory testing, and engineering analyses, it is our opinion that the proposed construction is feasible from a geotechnical standpoint, provided that the recommendations in this report are incorporated into the design plans and are implemented during construction.

6.1. General Considerations

Geotechnical engineering recommendations presented in this report for the proposed project are based on our understanding of the proposed development, subsurface conditions encountered during our field exploration, the results of laboratory testing on soil samples taken from the site, and our engineering analyses.

The following sections present our conclusions and recommendations pertaining to the engineering design for this project. If the design substantially changes, then our geotechnical engineering recommendations would be subject to revision based on our evaluation of the changes.

6.2. Soil Expansion and Collapse Potential

Based on our field exploration and laboratory test results, the risk of soil expansion and collapse is low at the site and will not adversely affect the design and construction of the project.

6.3. Corrosive Soil Evaluation

The potential for the near-surface on-site materials to corrode buried steel and concrete improvements was evaluated. Laboratory testing was performed on one selected near-surface soil to evaluate pH and electrical resistivity, as well as chloride and sulfate contents. The pH and electrical resistivity tests were performed in accordance with California Test 643, and the sulfate and chloride tests were performed in accordance with California Tests 417 and 422, respectively. These laboratory test results are presented in Appendix B.

Corrosive soil may be defined as the soil has minimum electrical resistivity less than 1,000 ohm-centimeters, or chloride concentration greater than 500 parts per million (ppm), or sulfate concentration in soils greater than 2,000 ppm, or a pH less than 5.5 (e.g., based on the County of Los Angeles criteria or the California Department of Transportation criteria).

Discussions of corrosion protection for reinforced concrete and buried metal is provided below. Further interpretation of the corrosivity test results and associated corrosion design and construction recommendations are within the purview of a corrosion specialist. It is recommended that a qualified corrosion engineer be retained to review the corrosivity test results, to evaluate the general corrosion potential with respect to construction materials at this site, and to review the proposed design.

6.3.1. Reinforced Concrete

Laboratory tests indicate that the soil has 267 ppm or 0.0267% of water soluble sulfate (SO_4) by weight. Based on ACI 318, concrete in contact with the site soils will have a sulfate exposure class S0. As a minimum, we recommend that Type II cement and a water-cement ratio of no greater than 0.50 be used on the project.

Test results indicate that the soil has 118 ppm of water soluble chlorides by weight and the potential for chloride attack of reinforcing steel in concrete structures and pipes in contact with soil is negligible. However, if needed, a corrosion specialist may be consulted for protection from chloride attack.

6.3.2. Buried Metal

A factor for evaluating corrosivity to buried metal is electrical resistivity. The electrical resistivity of a soil is a measure of resistance to electrical current. Corrosion of buried metal is directly proportional to the flow of electrical current from the metal into the soil. As resistivity of the soil decreases, the corrosivity generally increases. Test results indicate the site soils have minimum electrical resistivity value of 3,700 ohm-centimeters. Based on the criteria of the County of Los Angeles and the California Department of Transportation, the soils are considered to have low corrosion potential to buried metals.

Correlations between resistivity and corrosion potential published by the National Association of Corrosion Engineers (NACE, 1984) indicate that the soils are mildly corrosive to buried metals. Corrosion protection may include the use of epoxy or asphalt coatings. A corrosion specialist should be consulted regarding appropriate protection for buried metals and suitable types of piping.

6.4. Site Preparation and Earth Work

In general, earthwork should be performed in accordance with the recommendations presented in this report. Twining should be contacted for questions regarding the recommendations or guidelines presented herein.

6.4.1. Site Preparation

Site preparation should begin with the removal of utility lines, asphalt, concrete, vegetation, topsoil, and other deleterious debris from areas to be graded. Tree stumps and roots should be removed to such a depth that organic material is not present. Clearing and grubbing should extend to the outside edges of the proposed excavation and fill areas. We recommend that unsuitable materials such as organic matter or oversized material be removed and disposed of offsite. The debris and unsuitable material generated during clearing and grubbing should be removed from areas to be graded and disposed of at a legal dump site away from the project area.

6.4.2. Temporary Excavations

Temporary excavations are expected for the project. We anticipate that unsurcharged excavations with vertical sides less than 4 feet high will generally be stable. Where space is available, temporary, un-surcharged excavation sides over 4 feet in height should be sloped no steeper than an inclination of 1.5H:1V (horizontal:vertical).

The tops of the excavation sides should be barricaded so that vehicles and storage loads are away from the top edge of the excavated slopes with a distance at least equal to the height of the slopes. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes. Twining should be advised of such heavy vehicle loadings so that specific setback requirements can be established. If the temporary construction slopes are to be maintained during the rainy season, berms are recommended to be graded along the tops of the slopes in order to prevent runoff water from entering the excavation and eroding the slope faces.

Excavations should not undermine the existing adjacent improvements. Prior to excavation in the proximity of an existing improvement, Twining should be contacted to evaluate that there will be no loss of support for all excavations close to the existing improvement.

Personnel from Twining should observe the excavations so that any necessary modifications based on variations in the encountered soil conditions can be made. All applicable safety requirements and regulations, including CalOSHA requirements, should be met. Stability of temporary excavations is the responsibility of the contractor.

6.4.3. Over-Excavation and Subgrade Preparation

The proposed building may be supported by shallow foundations. It is recommended that the footings be founded on undisturbed competent native soils or engineered fill.

No undocumented fill was identified in our borings; however, if encountered during construction within the parking structure footprint, undocumented fill should be removed to its full depth. If undocumented fill is encountered during excavation for minor structures that are structurally separated from the building, the excavation should extend at least 2 feet below the finished grade or at least 1 foot below the bottom of the footing of the minor structures, whichever is greater. Excavation for pavements and hardscape should be over-excavated at least 1 foot as measured from the bottom of the pavement or hardscape section.

Laterally, foundation excavation should extend beyond the foundation limits a minimum distance equal to two feet or the depth of over-excavation, whichever is greater. Excavation for other improvements (e.g., concrete walkways, flatwork, pavement) should extend laterally at least two feet beyond the limits of the improvements.

The extent and depths of all removal should be evaluated by Twining's representative in the field based on the materials exposed. Should excavations expose soft soils or soils considered as unsuitable for use as fill by a Twining representative, additional removals may be recommended. For example, deeper removal may be required in areas where soft, saturated, or organic materials are encountered.

The exposed bottom of over-excavation should be evaluated and approved by Twining. The excavation bottom to receive fill should be scarified to a minimum depth of 6 inches and moisture conditioned to achieve generally consistent moisture contents approximately 2 percent above the optimum moisture content. The scarified bottom should be compacted to at least 90 percent relative compaction in accordance with the latest version of ASTM Test Method D1557 and then evaluated and approved by Twining.

Prior to placement of reinforcing steel or concrete for foundations, the bottom of footing excavations should be scarified to a minimum depth of 6 inches, moisture conditioned to achieve generally consistent moisture contents approximately 2 percent above the optimum moisture content, and recompacted to at least 90 percent of the maximum dry density as determined from ASTM D 1557.

Fill and backfill materials should be compacted fill in accordance with Sections 6.4.4 and 6.4.5 of this report. Prior to placement of any fill, the geotechnical engineer or their representative should review the bottom of the excavation for conformance with the recommendations of this report.

6.4.4. Materials for Fill

In general, on-site soils are considered as suitable for use as fill materials. All fill soils should be free of organics, debris, rocks or lumps over three inches in largest dimension, other deleterious material, and not more than 40 percent larger than ¾ inch. Larger chunks, if generated during excavation, may be broken into acceptably sized pieces or may be disposed of offsite.

Any imported fill material should consist of granular soil having a “very low” expansion potential (i.e., expansion index of 20 or less). Import material should also have low corrosion potential (that is, chloride content less than 500 ppm, soluble sulfate content of less than 0.1 percent, and pH of 5.5 or higher).

All fill soils should be evaluated and approved by a Twining representative prior to importing or filling.

6.4.5. Compacted Fill

Unless otherwise recommended, the exposed excavation bottom to receive fill should be prepared in accordance with Section 6.4.3 of this report. Prior to placement of compacted fill, the contractor should request Twining to evaluate the exposed excavation bottoms.

Compacted fill should be placed in horizontal lifts of approximately 8 to 10 inches in loose thickness, depending on the equipment used. Prior to compaction, each lift should be moisture conditioned, mixed, and then compacted by mechanical methods. The moisture content should be approximately 2 percent above the optimum moisture content. Fill materials should be compacted to a minimum relative compaction of 95 percent within the upper one foot below new vehicle trafficked pavement sections, and 90 percent in all other areas, unless indicated otherwise. The relative compaction should be determined by ASTM D1557. Successive lifts should be treated in the same manner until the desired finished grades are achieved.

6.4.6. Excavation Bottom Stability

In general, we anticipate that bottoms of the excavations will be stable and should provide suitable support for the proposed improvements. Conditions of the excavation bottom should be evaluated by Twining during the scarification and re-compaction efforts. If unstable bottom conditions are encountered, remedial measures would be required to stabilize the bottom. Soft bottom conditions can be identified by surface yielding under rubber-tired equipment loading and the inability to achieve proper compaction. Recommendations for stabilizing excavation bottoms should be based on evaluation in the field by the geotechnical consultant at the time of construction.

6.4.7. Backfill for Utility Trench

Utility trench excavations to receive backfill shall be free of trash, debris or other unsatisfactory materials at the time of backfill placement.

At locations where the trench bottom is yielding or otherwise unstable, pipe support may be improved by placing 12 inches of crushed aggregate base (CAB) or crushed miscellaneous base (CMB) as defined in the “Greenbook” Standard Specifications for Public Works Construction (SSPWC).

The trench should be bedded with clean sand extending to at least 6 inches below the bottom of the pipe and one foot over the top of pipe. Pipe bedding as specified in SSPWC can be used. Bedding material should consist of clean sand having a sand equivalent (SE) of 30 or greater. Alternative materials meeting the intent of the bedding specifications are also acceptable. Samples of materials proposed for use as bedding should be provided to the engineer for inspection and testing before the material is imported for use on the project. The onsite materials in the upper 20 feet consist of sandy lean clay and thus do not appear suitable for bedding, unless segregation of sandy materials is performed during excavation. The pipe bedding material should be placed over the full width of the trench. After placement of the pipe, the bedding should be brought up uniformly on both sides of the pipe and mechanically compacted to reduce the potential for unbalanced loads. No void or uncompacted areas should be left beneath the pipe haunches.

Above pipe bedding, trench backfill may be onsite soils and should not contain rocks or lumps over 3 inches in largest dimension. Larger chunks, if generated during excavation, may be broken into acceptably sized pieces or may be disposed of offsite. The moisture content should be approximately 2 percent above the optimum moisture content.

Backfill may be placed and compacted by mechanical means and should be compacted to 90 percent of the laboratory maximum dry density as per ASTM Standard D1557. Where pavement is planned, the top 12 inches of subgrade soils and the overlying aggregate base should be compacted to 95 percent.

Jetting or flooding of pipe bedding and backfill material is not recommended.

6.4.8. Rippability

The earth materials underlying the site should be generally excavatable with heavy-duty earthwork equipment in good working condition. Some gravels, cobbles and artificial fill (although not identified in our borings) should be anticipated.

6.4.9. Construction Dewatering

As discussed in Section 4.3, groundwater was not encountered during our field exploration to a maximum depth of 51.5 feet bgs. Construction of the project is anticipated to occur above the groundwater. The possibility to encounter groundwater is low during earthwork and foundation preparation for the proposed structures, and the need for dewatering is not anticipated for construction of foundations and utility trenches.

6.5. Foundation Recommendations

Based upon the excavation/over-excavation and backfill recommendations, the proposed parking structure may be supported on shallow foundations designed in accordance with the geotechnical recommendations presented below. Structural design of foundations should be performed by the structural engineer and should conform to the 2019 California Building Code.

6.5.1. Footing Foundation

Continuous strip footings or isolated footings for the proposed parking structure should be placed on the subgrade prepared in accordance the requirements described in 6.4. Geotechnical design parameters for these footings presented in Table 6 may be used. Twining

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should be contacted for footing dimensions, allowable bearing pressures, and settlements that are outside the indicated applicable ranges.

Lateral loads may be resisted by footing base friction and by the passive resistance of the soils based on recommendations provided in Table 6.

The total lateral resistance can be taken as the sum of the friction at the base of the footing and passive resistance. The upper one foot of soil should be neglected when calculating the passive resistance. The passive resistance value may be increased by one-third for transient loads from wind or earthquake.

Table 6 - Geotechnical Design Parameters for Footing Foundations

Minimum Footing Dimensions	• Continuous footings: 18 inches in width. • Square footings: 24 inches in width. • Minimum embedment: 24 inches measured from the lowest adjacent grade to the bottom of the footing. • Minimum thickness: 6 inches
Allowable Bearing Pressure	• An allowable bearing pressures of 2,000 pounds per square foot (psf) may be used. The allowable may be increased by 230 psf for each additional foot of width and 650 psf for each additional foot of embedment, up to a maximum allowable capacity of 5,000 psf. • The allowable bearing values correspond to a factor of safety of 3. • The allowable bearing values may be increased by one-third for transient loads from wind or earthquake.
Estimated Static Settlement	• Approximately one inch of total settlement with differential settlement estimated to be on the order of ½ inches over 30 feet. • The static settlement of the foundation system is expected to complete on initial application of loading.
Estimated Seismic Settlement	• Approximately 1.0 inches with differential settlement of less than 0.5 inches over a horizontal distance of 50 feet
Allowable Coefficient of Friction Below Footings	• An allowable coefficient of friction of 0.35 may be used at bottom of footings. • The allowable bottom friction values correspond to a factor of safety of 1.5.
Allowable Lateral Passive Resistance	• Increases with depth at a rate of 300 psf per foot (300 pcf equivalent fluid pressure) • The allowable passive resistance corresponds to a factor of safety of 2.

6.6. Surcharge from Adjacent Footings

Design of new footings or evaluation of existing footings should consider vertical surcharge from adjacent footings that are located above the 1:1 plane drawn up from the closest bottom edge of the footing being designed or evaluated. Surcharge located below the 1:1 plane may be ignored.

6.7. Modulus of Subgrade Reaction

The modulus of subgrade reaction k for design of combined footing and slabs-on-grade may be obtained from the following equation.

$$k = \frac{k_1}{B}\left(\frac{2L + B}{3L}\right)$$

where: k_1 = modulus for a 1-foot by 1-foot plate = 100 pounds per cubic inch (pci);
B = width of combined footing or slab in feet;
L = length of combined footing or slab in feet, and $L \geq B$.

6.8. Concrete Slabs-On-Grade

Concrete slabs-on-grade should be supported on non-expansive engineered fill in accordance with Section 6.4 of this report. For design of concrete slabs, the subgrade modulus k calculated from Section 6.7 may be used.

Floor slabs should be designed and reinforced in accordance with the structural engineer's recommendations. For slabs not supporting heavy loads, we recommend that the concrete should have a thickness of at least 4 inches, a 28-day compressive strength of at least 3,000 pounds per square inch (psi), a water-cement ratio of 0.50 or less, and a slump of 4 inches or less. Slabs should be reinforced with at least No. 3 reinforcing bars placed longitudinally at 18 inches on center. The reinforcement should extend through the control joints to reduce the potential for differential movement. Control joints should be constructed in accordance with recommendations from the structural engineer or architect. For slabs supporting equipment, a minimum thickness of 5 inches is recommended. Additional thickness and reinforcement recommendations may be provided by the structural engineer.

The topmost 8 inches below the slab subgrade should be maintained in a moisture condition of approximately 0 to 2 percent above optimum moisture content. The slab subgrade should be tested for moisture and compaction immediately prior to placement of the gravel or sand base, if any. All underslab materials should be adequately compacted prior to the placement of concrete. Care should be taken during placement of the concrete to prevent displacement of the underslab materials. The underslab material should be dry or damp and should not be saturated prior to the placement of concrete. The concrete slab should be allowed to cure properly and should be tested for moisture transmission prior to placing vinyl or other moisture-sensitive floor covering. In moisture sensitive areas, the floor slabs should be dampproofed in accordance with Section 1805A.2 of 2019 CBC. Specific recommendations can be provided by a waterproofing consultant.

Table 7 provides general recommendations for various levels of protection against vapor transmission through concrete floor slabs placed over a properly prepared subgrade. Care should be taken not to puncture the plastic membrane during placement of the membrane itself and the overlying silty sand.

Table 7 - Options for Subgrade Preparation below Concrete Floor Slabs

Primary Objective	Recommendation
Enhanced protection against vapor transmission	• Concrete floor slab-on-grade placed directly on a 15-mil-thick moisture vapor retarder that meets the requirements of ASTM E1745 Class C (Stego Wrap or similar) • The moisture vapor retarder membrane should be placed directly on the subgrade (ACI302.1R-67); if required for either leveling of the subgrade or for protection of the membrane from protruding gravel, then place about 2 inches of silty sand[1] under the membrane
Above-standard protection against vapor transmission	This option is available if the slab perimeter is bordered by continuous footings at least 24 inches deep, OR if the area adjacent and extending at least 10 feet from the slab is covered by hardscape without planters: • 2 inches of dry silty sand[1]; over • Waterproofing plastic membrane 10 mils in thickness; over • At least 4 inches of ¾-inch crushed rock[2] or clean gravel[3] to act as a capillary break
Standard protection against vapor transmission	• 2 inches of dry silty sand[1]; over • Waterproofing plastic membrane 10 mils in thickness • If required for either leveling of the subgrade or for protection of the membrane from protruding gravel, place at least 2 inches of silty sand[1] under the membrane.

Notes:

[1] The silty sand should have a gradation between approximately 15 and 40 percent passing the No. 200 sieve and a plasticity index of less than 4.

[2] The ¾-inch crushed rock should conform to Section 200-1.2 of the latest edition of the "Greenbook" Standard Specifications for Public Works Construction (Public Works Standards, Inc., 2012).

[3] The gravel should contain less than 10 percent of material passing the No. 4 sieve and less than 3 percent passing the No. 200 sieve.

The above recommendations are intended to reduce the potential for cracking of slabs; however, even with the incorporation of the recommendations presented herein, slabs may still exhibit some cracking. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics.

6.9. Pole Foundations

Pole foundations for flagpoles, fences, and signposts may be designed using an allowable unit skin friction of 375 psf and an allowable end bearing resistance of 2,000 psf. A factor of safety of 2 is incorporated into the allowable skin friction, and a factor of 3 is incorporated into the allowable end bearing.

Lateral resistance for conditions with and without lateral constraint provided at the ground surface conditions are provided below based on 2019 CBC.

6.9.1. Non-Constrained Ground

The embedment of pole foundations where no lateral constraint is provided at or above the ground surface should be calculated using Equation 18-1 of 2019 IBC (shown below) or a minimum 3 feet below the ground surface, whichever is deeper.

$$d = \frac{A}{2}\left(1 + \sqrt{1 + \frac{4.36h}{A}}\right) \qquad \text{(Equation 18-1 of 2019 CBC)}$$

where:

- A = $2.34P/(S_1 * b)$
- b = Diameter of round post or footing or diagonal dimension of square post or footing, feet.
- d = Depth of embedment in earth in feet but not over 12 feet for purpose of computing lateral pressure.
- h = Distance in feet from ground surface to point of application of “P”.
- P = Applied lateral force in pounds.
- S_1 = Allowable lateral soil-bearing pressure based on a depth of one-third the depth of embedment in pounds per square foot.

An allowable passive earth pressure of 300 pcf up to a maximum of 4,500 psf may be used for design provided the upper one foot of passive resistance is neglected in the structural design. Isolated pole foundations spaced at least 3 diameters of the maximum pole foundation may be designed using an allowable lateral resistance equal to 2 times of the allowable passive pressure.

6.9.2. Constrained Ground

The embedment of pole foundations where lateral constraint is provided at the ground surface, such as by a rigid floor or pavement, should be calculated using Equation 18-2 of 2019 IBC (shown below) or a minimum 3 feet below the ground surface, whichever is deeper.

$$d = \sqrt{\frac{4.25Ph}{S_3 b}} \qquad \text{(Equation 18-2 of 2019 CBC)}$$

where:

- b = Diameter of round post or footing or diagonal dimension of square post or footing, feet.
- d = Depth of embedment in earth in feet but not over 12 feet for purpose of computing lateral pressure.
- h = Distance in feet from ground surface to point of application of “P”.
- P = Applied lateral force in pounds.

S_3 = Allowable lateral soil-bearing pressure based on a depth of one-third the depth of embedment in pounds per square foot.

An allowable passive earth pressure of 300 pcf up to a maximum of 4,500 psf may be used for design provided the upper one foot of passive resistance is neglected in the structural design. Isolated pole foundations spaced at least 3 diameters of the maximum pole foundation may be designed using an allowable lateral resistance equal to 2 times of the allowable passive pressure.

6.10. Below-Grade Wall and Lateral Earth Pressure Recommendations

For walls below grade, recommendations for wall lateral loads, backfill, and drainage are provided below. Lateral resistance may be based on Section 6.5.1 of this report. Retaining walls should be designed to have a factor of safety of 1.5 for static stability and 1.1 for stability due to transient loads from wind or seismic.

6.10.1. Backfill and Drainage of Walls

The backfill material behind walls should consist of granular non-expansive material and be approved by the project geotechnical engineer. Based on the soil materials encountered during our exploration, most on-site soils will meet this requirement, provided that wall backfill is adequately drained.

Wall backfill should be adequately drained. Adequate backfill drainage is essential to provide a free-drained backfill condition and to limit water pressure buildup behind walls. Drainage behind walls may be provided by a geosynthetic drainage composite such as TerraDrain, MiraDrain, or equivalent, attached to the outside perimeter of the wall and installed in accordance with the manufacturer's recommendations. The drainage system should meet the minimum requirements of Sections 1805.4.2 and 1805.4.3 of 2019 CBC.

6.10.2. Lateral Earth Pressure

The values presented below assume that the supported grade is level and that surcharge loads are not applied. The recommended design lateral earth pressure is calculated assuming that a drainage system will be installed behind retaining walls in accordance with Sections 1805.4.2 and 1805.4.3 of 2019 CBC and that external hydrostatic pressure will not develop behind the walls. Where wall backfill does not have adequate drainage, the full hydrostatic pressure should be added to the lateral earth pressures provided below in design.

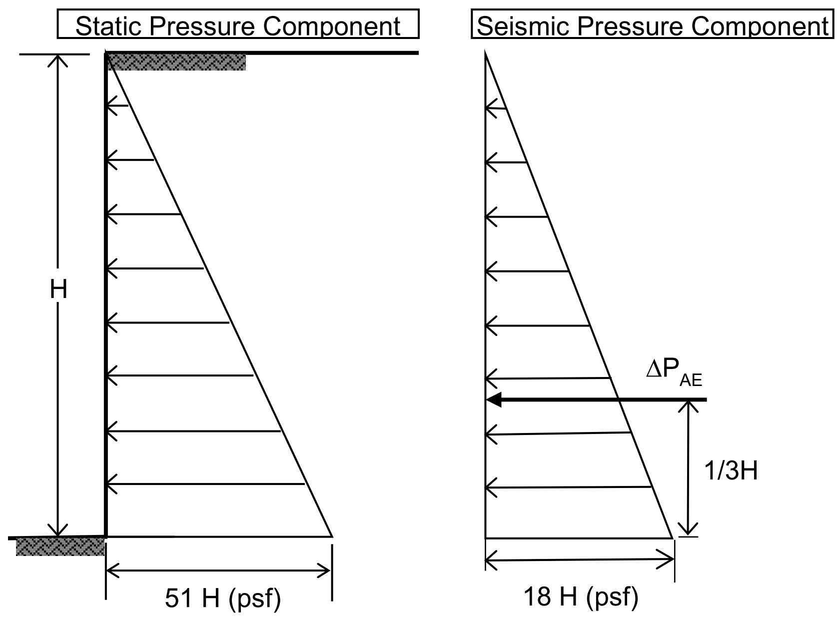
Walls that are free to move and rotate at the top (such as cantilevered walls) and have adequate drainage may be designed for the active earth pressure equivalent to a fluid weighting 51 pcf.

Walls that are restricted to move horizontally at the top (such as by a floor deck) and have adequate drainage may be designed for the "at-rest" earth pressure equivalent to a fluid weighing 72 pcf.

Vertical surcharge loads within a 1:1 plane projected from the bottom of the wall distributed over retained soils should be considered as additional uniform horizontal pressures acting on the wall. These additional pressures can be estimated as approximately 41% and 58% of the magnitude of the vertical surcharge pressures for the "active" and "at-rest" conditions, respectively.

6.10.3. Seismic Lateral Earth Pressure

Walls retaining more than 6 feet high earth should be designed for seismic lateral earth pressure. The seismic pressure distribution may be considered a triangle with the maximum pressure at the bottom. The combination of static and incremental seismic pressures shown in the following diagram may be used for seismic design for both cantilever and restrained walls.



where H is in feet

Seismic Earth Pressure Distribution on Walls

6.11. Temporary Shoring

If the project involves excavations that lack sufficient space for sloped excavations, cantilevered shoring or braced- or tieback shoring should be considered and designed.

For vertical excavations less than approximately 15 feet in height, cantilevered shoring may be used. Where cantilevered shoring is used for deeper excavations, the total deflection at the top of the wall tends to exceed acceptable magnitudes. Shoring of excavations deeper than approximately 15 feet should be accomplished with the aid of internal bracing or tieback earth anchors.

The shoring design should be provided by a California Registered Civil Engineer experienced in the design and construction of shoring under similar conditions. Once the final excavation and shoring plans are complete, the plans and the design should be reviewed by the geotechnical engineer for conformance with the design intent and recommendations. Further, the shoring system should satisfy applicable requirements of CalOSHA.

6.11.1. Lateral Earth Pressures

For design of cantilevered shoring for excavations less than 15 feet in height, a triangular distribution of lateral earth pressure may be used. It may be assumed that the drained soils, with a level surface behind the cantilevered shoring, will exert an equivalent fluid pressure of 51 pcf.

For the design of braced- or tieback-shoring, a rectangular pressure distribution where the pressure may be used. The design pressure should be 36H psf, where H is the retained soil height in feet.

Any surcharge (live, including traffic, or dead load) located within a 1:1 plane projected upward from the base of the shored excavation, including adjacent structures, should be added to the lateral earth pressures. The lateral contribution of a uniform surcharge load located immediately behind the temporary shoring may be calculated by multiplying the vertical surcharge pressure by 41% for cantilevered shoring and 58% for braced- or tieback-shoring, corresponding to the "active" and "at-rest" conditions, respectively. Lateral load contributions of surcharges located at a distance behind the shored wall may be provided once the load configurations and layouts are known. As a minimum, a 250 psf vertical uniform surcharge is recommended to account for nominal construction and/or traffic loads. More detailed lateral pressure and loading information can be provided, if needed, for specific loading scenarios as recognized through the design process.

6.11.2. Soldier Pile Design

The soldier piles for support of shoring should be designed in accordance with the geotechnical parameters presented in Table 8. Soldier piles should be spaced no closer than 3D on center, where D is the diameter of the drilled shaft for the soldier piles. Soldier piles may consist of either cast-in-place concrete caissons or pre-drilled steel beams encased in concrete (below the bottom of the excavation) and slurry (above the bottom of the excavation).

Table 8 - Geotechnical Design Parameters for Soldier Piles

The allowable lateral resistance of an isolated soldier pile drilled into the on-site soils can be calculated using equivalent fluid pressure (EFP)	300 pcf
Increase (multiplier) of the ultimate lateral passive resistance due to arching (this value is applicable for soldier piles that are spaced no closer than 3 diameters)	2

Continuous timber lagging should be used between the soldier piles. If treated timber is used, the lagging may remain in place. To develop the full lateral resistance, provisions should be taken to assure firm contact between the soldier piles and the soils; for this, we recommend that 1-½-sack sand-cement slurry infill behind the lagging be used. For drilled piles, we recommend that piles adjacent to one another be drilled alternately on different days to minimize disturbance to the open excavations.

Drilling of the soldier pile shafts can be accomplished using conventional drilling equipment. Caving should be anticipated where layers of clean sand or silty sand occurs. In the event of soil caving, it may be necessary to use casing and/or drilling mud to permit the installation of the soldier piles. Drilled holes for soldier piles should not be left open overnight. Concrete for piles should be placed immediately after the drilling of the hole and placement of the steel pile (or rebar cage) is complete. The concrete should be pumped to the bottom of the drilled shaft using a tremie. Once concrete pumping is initiated, the bottom of the tremie should remain below the surface of the concrete to prevent contamination of the concrete by soil inclusions. If steel casing is used, the casing should be removed as the concrete is placed. The concrete placed in the soldier pile excavations may be a lean mix concrete above the elevation of the bottom of the excavation. However, the concrete that is placed in the portion of the soldier pile that is below the deepest planned excavated level should have a minimum 28-day compressive strength of at least 2,500 pounds per square inch (psi). The contractor may also consider the use of driven piles or piles that are vibrated into place in lieu of drilled piles to address potential issues related to caving of drilled shafts.

6.11.3. Tieback Design

Excavations deeper than 15 feet may require tieback anchors to be used to resist lateral loads. For design purposes, it may be assumed that the failure wedge adjacent to the shoring is defined by a plane up at approximately 30 degrees from the vertical from the toe of the wall. The anchors should extend at least 15 feet beyond the potential failure wedge; however, the shoring engineer should evaluate the bonded length required beyond the failure wedge based on the loading on the shoring and the allowable skin friction provided. The bonded length should commence no less than 3 feet beyond the failure wedge.

We recommend using an allowable soil/anchor bond friction of 500 psf along the anchors in the bonded zone with a factor of safety of 1.5. Only friction developed beyond the active wedge should be considered when determining the tieback resistance. If the anchors are spaced at least 6 feet on center, no reduction in the capacity of the anchors need be considered due to group action.

As the tieback shoring system is intended for temporary use, provisions should be made in the design to de-tension and abandon the tiebacks when the subgrade walls are able to support the lateral loads.

6.11.4. Anchor Installation

The anchors may be installed at angles of 15 to 30 degrees below the horizontal. Caving may occur during the drilling of tiebacks if loose cohesionless materials are encountered. The contractor should implement appropriate measures to stabilize the drilled hole such as the installation of steel casing for loose cohesionless materials or the use of drilling mud. The anchors should be filled with concrete placed by pumping from the tip out. The portion of the anchor tendons within the failure wedge should be sleeved in plastic. If the anchor tendons are sleeved, it is acceptable to grout the entire length of the anchor.

6.11.5. Lagging and Sheeting

Continuous lagging will be required between the soldier piles. The soldier piles and anchors should be designed for the full anticipated lateral pressure. However, where lagging is relatively flexible to wales or soldier beams, the pressure on the lagging will be less due to arching in the soils. We recommend that the lagging be designed for a semi-circular distribution of earth pressure where the maximum pressure is 500 pounds per square foot at the mid-line between soldier piles, and 0 pounds per square foot at the soldier piles.

6.11.6. Lateral Deflection and Settlement

Excessive deflection could result in settlement or undermining of surrounding structures. Shoring should be adequately designed, installed, and monitored to limit the amount of lateral deflection of the shoring system and settlement behind the shoring to the allowable values of adjacent structures and improvements. The amount of deflection of the shoring system and the allowable deflections and settlements should be determined by the shoring designer. The allowable deflections and settlements should be based on the proximity of adjacent structures and improvements and the potential negative effects on those structures. If it is desired to reduce the deflection, a greater lateral pressure could be used in shoring design. If greater than anticipated deflection occurs during construction, additional bracing or tiebacks may be necessary to minimize deflection of existing adjacent improvements.

Settlement of structures or facilities founded adjacent to the shoring will occur in proportion to both the distance between the shoring and the facilities, and the amount of horizontal deflection of the shoring system. The vertical settlement will be a maximum at the shoring face and decrease as the horizontal distance from the shoring increases. Beyond a distance from the shoring equal to the height of the shoring, the settlement is expected to be negligible. The maximum vertical settlement is expected to be about 75 percent of the maximum horizontal deflection on top of the shoring system. The geotechnical engineer should review the shoring design to ensure that the recommendations provided herein are properly incorporated into the design.

6.11.7. Monitoring

For excavations in close proximity to existing improvements, some means of monitoring the performance of the shoring system is recommended. Monitoring should consist of periodic surveying of lateral and vertical locations at the tops of all soldier piles. The geotechnical engineer should review the results of the monitoring during construction.

6.12. Pavement Recommendations

Pavement section should be constructed on top of properly prepared subgrade in accordance with Section 6.4 of this report and aggregate base (AB) section compacted to 95 percent of the maximum dry density in accordance with ASTM D1557.

We performed laboratory R-value testing for preliminary pavement section design. The test indicates an R value of 18, and it was used in our pavement structural calculations. Sections 6.12.1 and 6.12.2 present our recommendations for preliminary design of flexible and rigid pavement sections, respectively. Final pavement design should be based on field observations, additional R-value tests during construction should the materials exposed differ than what is expected based on our field exploration, and the anticipated traffic index as determined by the project civil engineer.

6.12.1. Flexible Pavement Design

Our flexible pavement structural design is in accordance with Chapter 630 of the Caltrans Highway Design Manual, which is based on a relationship between the gravel equivalent (GE) of the pavement structural materials, the traffic index (TI), and the R-value of the underlying subgrade soil. For preliminary design of flexible pavement section, Table 9 provides recommended minimum thicknesses for hot mix asphalt (HMA) and aggregate base sections for different traffic indices.

Table 9 – Recommended Minimum HMA and Base Section Thicknesses

Traffic Index	5.0	6.0	7.0
HMA Thickness (in)	4	5	6
Aggregate Base Thickness (in)	6	7	10

6.12.2. Rigid Pavement Design

For preliminary design of rigid pavement section, Table 10 provides recommended minimum thicknesses for Portland cement concrete (PCC) pavement section and Class 2 Aggregate Base (AB) section for different traffic indices. The recommended values are based on a minimum 28-day concrete compressive strength of 3,500 psi. Positive drainage should be provided away from all pavement areas to prevent seepage of surface and/or subsurface water into the pavement base and/or subgrade.

Table 10 – Recommended Minimum Rigid Pavement Thicknesses

Traffic Index	5.0	6.0	7.0
PCC Thickness (in)	6	6.5	7.0
Aggregate Base Thickness (in)	6	6	6

6.13. Stormwater Infiltration Facility

Percolation testing will be required based on the actual location and depth of the planned system. The design of stormwater infiltration facility should be based on percolation test results with an appropriate factor of safety.

Our percolation test results may be used in preliminary design. Details of the percolation tests are presented in Appendix A. Infiltration rates with a factor of safety of 3 from our percolation tests are summarized in Table 11.

Any proposed infiltration facility should have a minimum setback from property lines and foundations recommended in Table 12. In addition, the bottom of the infiltration facility should be at least 10 feet above the seasonal high groundwater. We recommend that we review the proposed groundwater infiltration system prior to implementation or finalizing design.

Table 11 – Infiltration Rate with a Factor of Safety of 3

Test Location	Depth of Test Borehole (feet)	Infiltration Rate (inch/hour)
P-1	6	0.3
P-2	6.5	0.7

Table 12 – Recommended Minimum Infiltration Facility Setback

Setback from	Distance
Property lines & public right of way	5 feet
Foundations	the greater of 15 feet or a 1:1 plane drawn up from the bottom of foundation
Seasonal high groundwater	10 feet minimum depth from invert of infiltration device
Face of slope	the greater of 5 feet or one half of the slope height
Water wells	100 feet

6.14. Drainage Control

The control of surface water is essential to the satisfactory performance of the structure and site improvements. Surface water should be controlled so that conditions of uniform moisture are maintained beneath the improvements, even during periods of heavy rainfall. The following recommendations are considered minimal:

- Ponding and areas of low flow gradients should be avoided.
- If bare soil within 5 feet of the structure is not avoidable, then a gradient of 5 percent or more should be provided sloping away from the improvement. Corresponding paved surfaces should be provided with a gradient of at least 1 percent.
- The remainder of the unpaved areas should be provided with a drainage gradient of at least 2 percent.
- Positive drainage devices, such as graded swales, paved ditches, and/or catch basins should be employed to accumulate and to convey water to appropriate discharge points.
- Concrete walks and flatwork should not obstruct the free flow of surface water.
- Brick flatwork should be sealed by mortar or be placed over an impermeable membrane.
- Area drains should be recessed below grade to allow free flow of water into the basin.
- Enclosed raised planters should be sealed at the bottom and provided with an ample flow gradient to a drainage device. Recessed planters and landscaped areas should be provided with area inlet and subsurface drain pipes.
- Planters should not be located adjacent to the structures wherever possible. If planters are to be located adjacent to the structures, the planters should be positively sealed, should incorporate a subdrain, and should be provided with free discharge capacity to a drainage device.
- Planting areas at grade should be provided with positive drainage. Wherever possible, the grade of exposed soil areas should be established above adjacent paved grades. Drainage devices and curbing should be provided to prevent runoff from adjacent pavement or walks into planted areas.
- Gutter and downspout systems should be provided to capture discharge from roof areas. The accumulated roof water should be conveyed to off-site disposal areas by a pipe or concrete swale system.

Landscape watering should be performed judiciously to preclude either soaking or desiccation of soils. The watering should be such that it just sustains plant growth without excessive watering. Sprinkler systems should be checked periodically to detect leakage and they should be turned off during the rainy season.

7. DESIGN REVIEW AND CONSTRUCTION MONITORING

Geotechnical review of plans and specifications is of paramount importance in engineering practice. The poor performance of many structures has been attributed to inadequate geotechnical review of construction documents. Additionally, observation and testing of the subgrade will be important to the performance of the proposed development. The following sections present our recommendations relative to the review of construction documents and the monitoring of construction activities.

7.1. Plans and Specifications

The design plans and specifications should be reviewed by Twining, Inc. prior to bidding and construction, as the geotechnical recommendations may need to be reevaluated in the light of the actual design configuration and loads. This review is necessary to evaluate whether the recommendations contained in this report and future reports have been properly incorporated into the project plans and specifications. Based on the work already performed, this office is best qualified to provide such review.

7.2. Preconstruction Surveys

We recommend that preconstruction surveys be performed on the adjacent improvements prior to commencement of excavation activities for the subject project. The surveys should include written and photographic (or videographic) documentation of the existing conditions, as well as performance of floor level surveys or establishment of elevation monuments. Documentation of other structures and sensitive instruments within approximately 50 feet of the excavation(s) should also be performed.

7.3. Construction Monitoring

Site preparation, removal of unsuitable soils, assessment of imported fill materials, fill placement, foundation installation, and other site grading operations should be observed and tested, as appropriate. The substrata exposed during the construction may differ from that encountered in the test excavations. Continuous observation by a representative of Twining, Inc. during construction allows for evaluation of the soil conditions as they are encountered and allows the opportunity to recommend appropriate revisions where necessary.

8. LIMITATIONS

The recommendations and opinions expressed in this report are based on Twining, Inc.'s review of available background documents, on information obtained from field explorations, and on laboratory testing. It should be noted that this study did not evaluate the possible presence of hazardous materials on any portion of the site. In the event that any of our recommendations conflict with recommendations provided by other design professionals, we should be contacted to aid in resolving the discrepancy.

Due to the limited nature of our field explorations, conditions not observed and described in this report may be present on the site. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation and laboratory testing can be performed upon request. It should be understood that conditions different from those anticipated in this report may be encountered during grading operations, for example, the extent of removal of unsuitable soil, and that additional effort may be required to mitigate them.

Site conditions, including groundwater elevation, can change with time as a result of natural processes or the activities of man at the subject site or at nearby sites. Changes to the applicable laws, regulations, codes, and standards of practice may occur as a result of government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Twining, Inc. has no control.

Twining's recommendations for this site are, to a high degree, dependent upon appropriate quality control of subgrade preparation, fill placement, and foundation construction. Accordingly, the recommendations are made contingent upon the opportunity for Twining to observe grading operations and foundation excavations for the proposed construction. If parties other than Twining are engaged to provide such services, such parties must be notified that they will be required to assume complete responsibility as the geotechnical engineer of record for the geotechnical phase of the project by concurring with the recommendations in this report and/or by providing alternative recommendations.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Twining should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report has been prepared for the exclusive use by the client and its agents for specific application to the proposed project. Land use, site conditions, or other factors may change over time, and additional work may be required with the passage of time. Based on the intended use of this report and the nature of the new project, Twining may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the Client or anyone else will release Twining from any liability resulting from the use of this report by any unauthorized party.

Twining performed its evaluation using the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical professionals with experience in this area in similar soil conditions. No other warranty, either express or implied, is made as to the conclusions and recommendations contained in this report.

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TWINING

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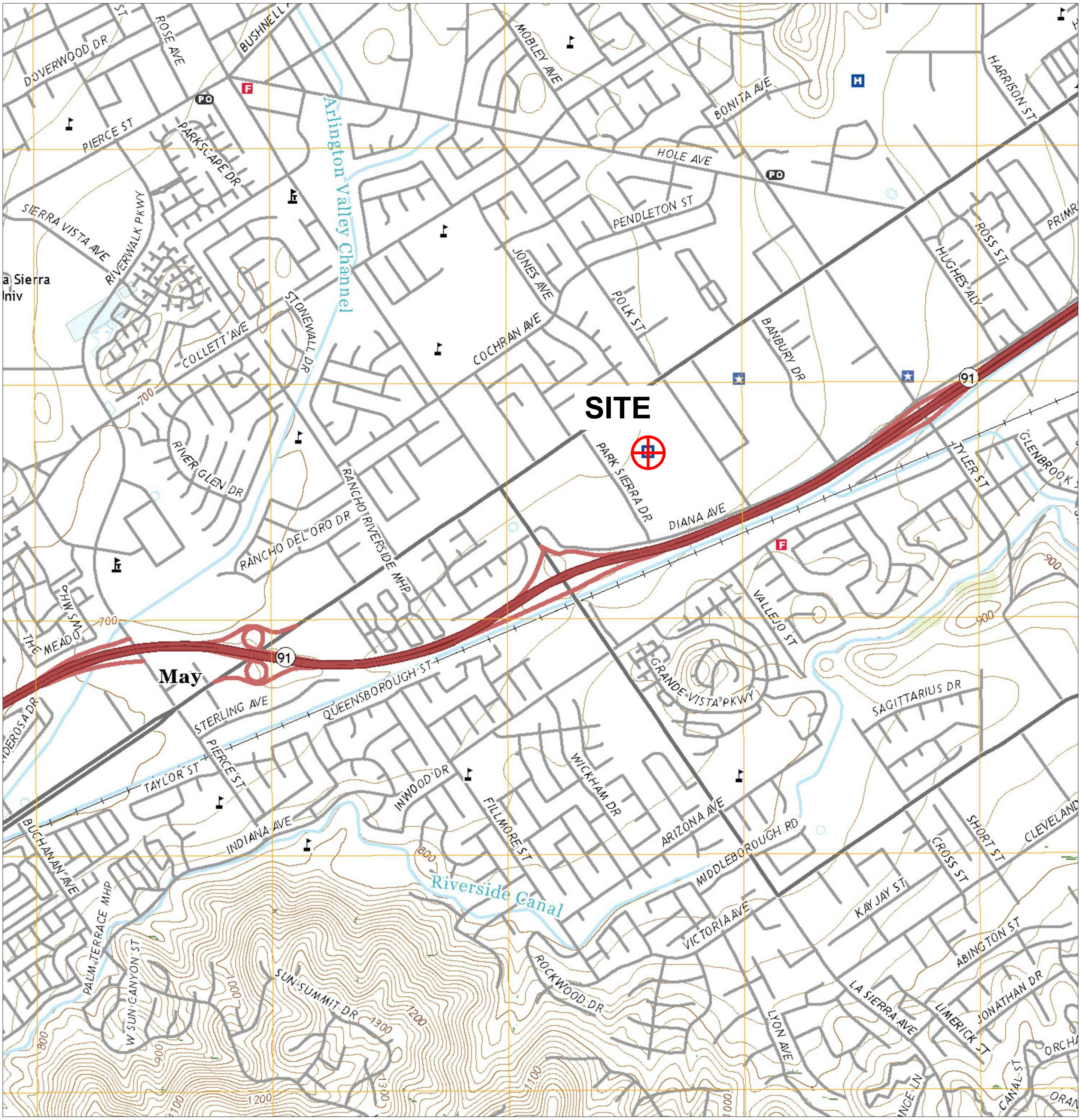
FIGURES



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SCALE IN FEET

0 2000 4000

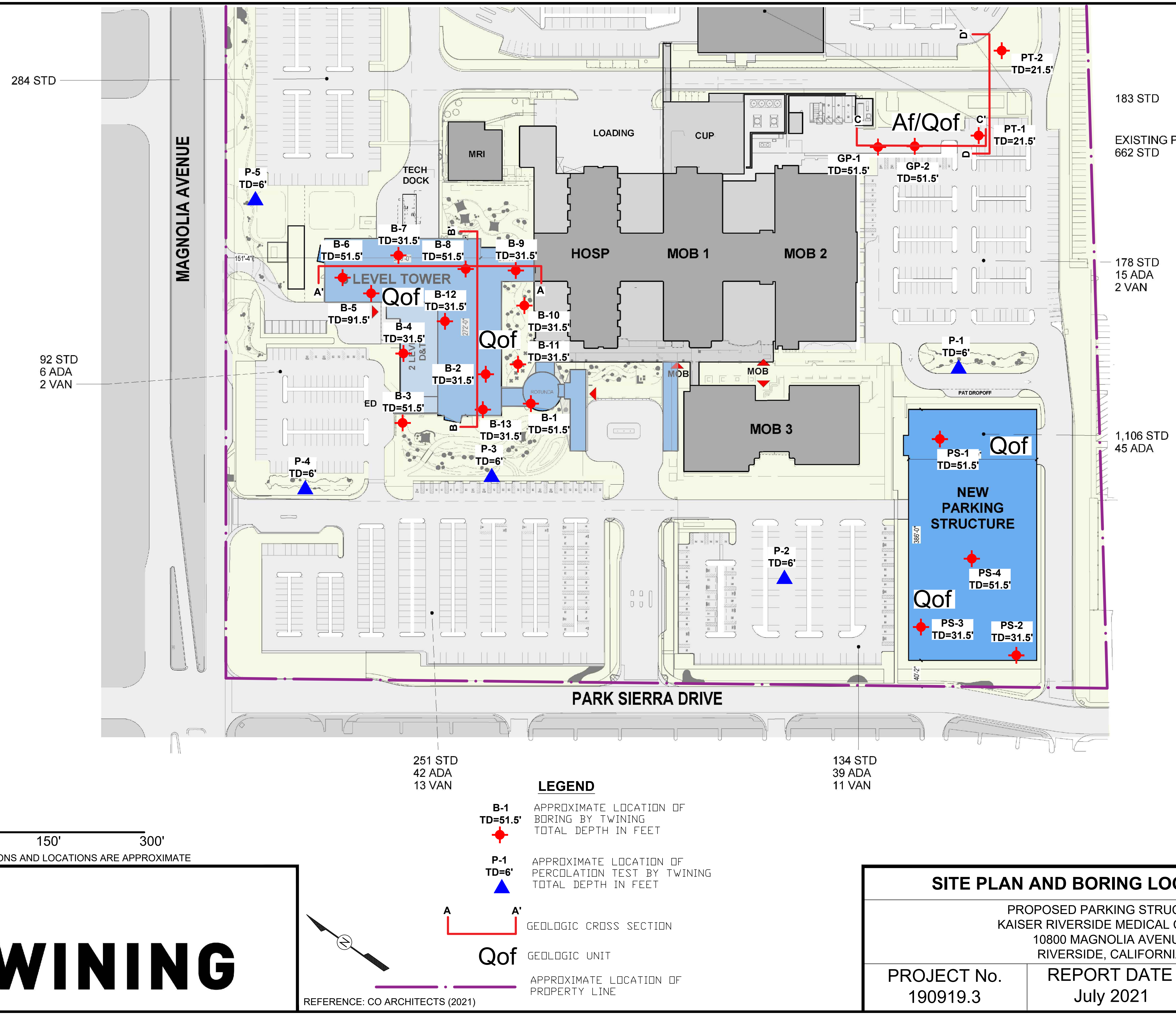
REFERENCE: USGS (2018)

TWINING

SITE LOCATION MAP

PROPOSED PARKING STRUCTURE
KAISER RIVERSIDE MEDICAL CENTER
10800 MAGNOLIA AVENUE
RIVERSIDE, CALIFORNIA

PROJECT NO.	REPORT DATE	FIGURE 1
190919.3	March 2021	



NOTE: ALL DIMENSIONS AND LOCATIONS ARE APPROXIMATE

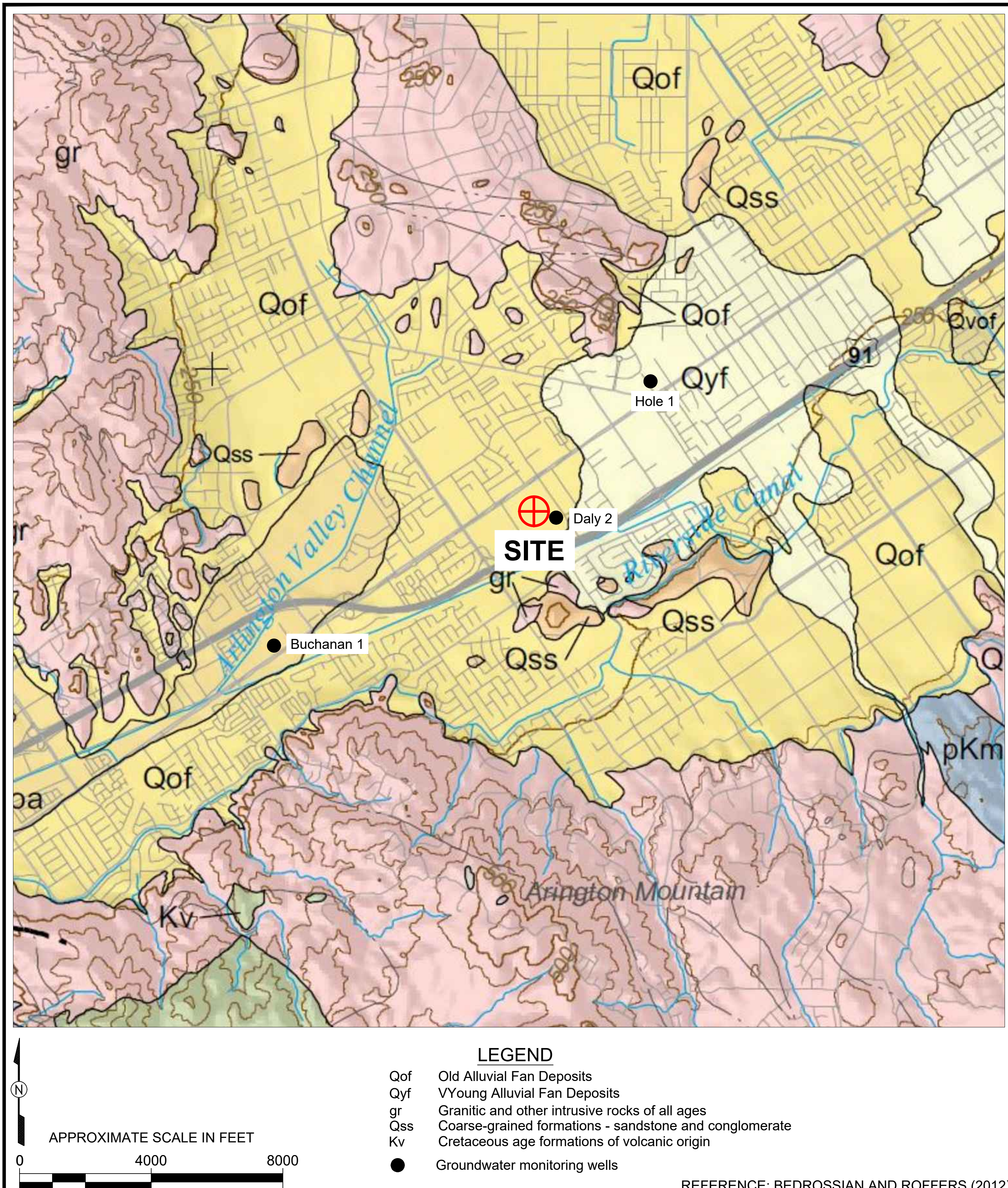
TWINING

REFERENCE: CO ARCHITECTS (2021)

SITE PLAN AND BORING LOCATION MAP

PROPOSED PARKING STRUCTURE
KAISER RIVERSIDE MEDICAL CENTER
10800 MAGNOLIA AVENUE
RIVERSIDE, CALIFORNIA

PROJECT No.	REPORT DATE	FIGURE 2
190919.3	July 2021	



LEGEND

Qof Old Alluvial Fan Deposits
Qyf VYoung Alluvial Fan Deposits
gr Granitic and other intrusive rocks of all ages
Qss Coarse-grained formations - sandstone and conglomerate
Kv Cretaceous age formations of volcanic origin
● Groundwater monitoring wells

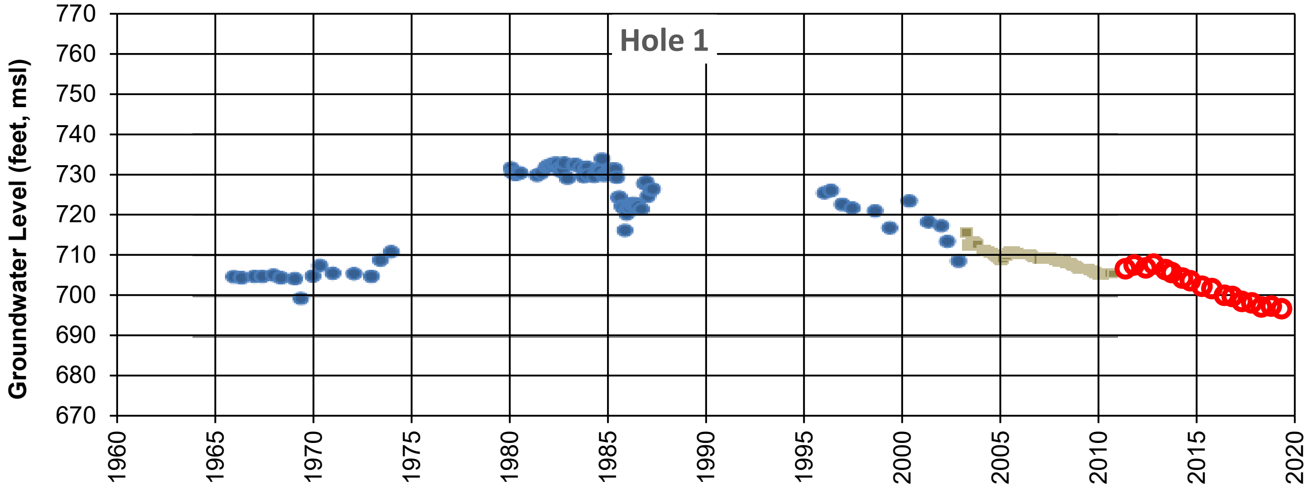
REFERENCE: BEDROSSIAN AND ROFFERS (2012)

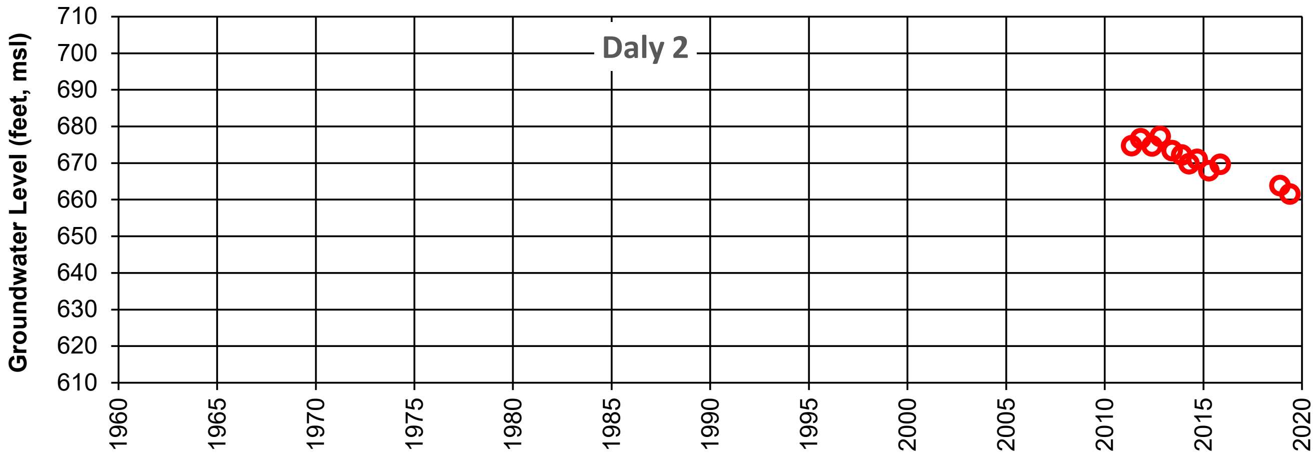
TWINING

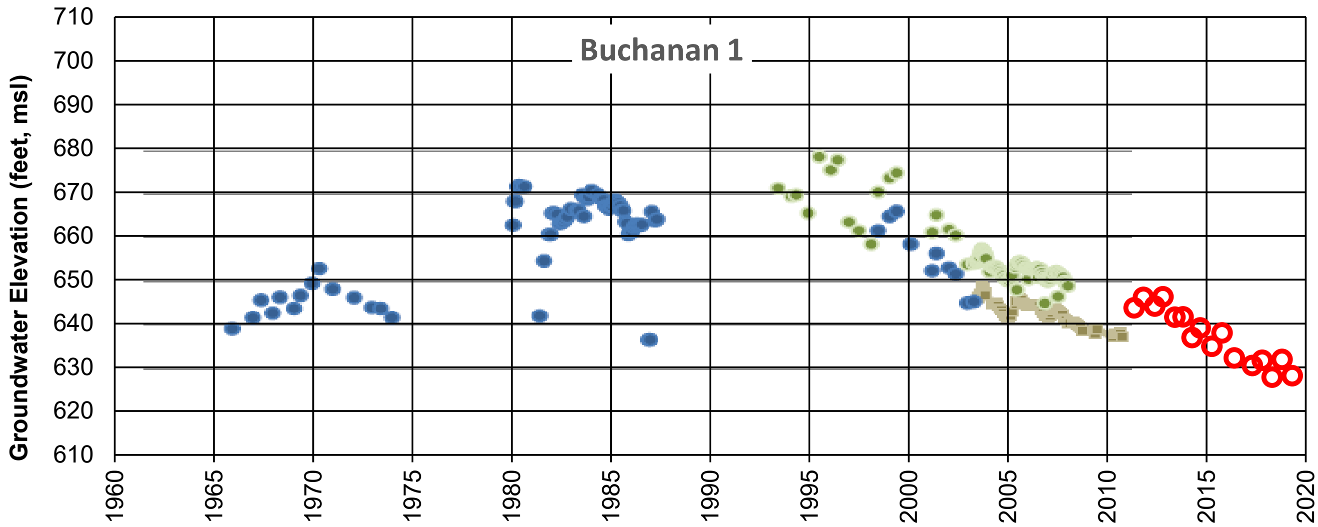
REGIONAL GEOLOGIC MAP

PROPOSED PARKING STRUCTURE
KAISER RIVERSIDE MEDICAL CENTER
10800 MAGNOLIA AVENUE
RIVERSIDE, CALIFORNIA

PROJECT NO.	REPORT DATE	FIGURE 3
190919.3	March 2021	







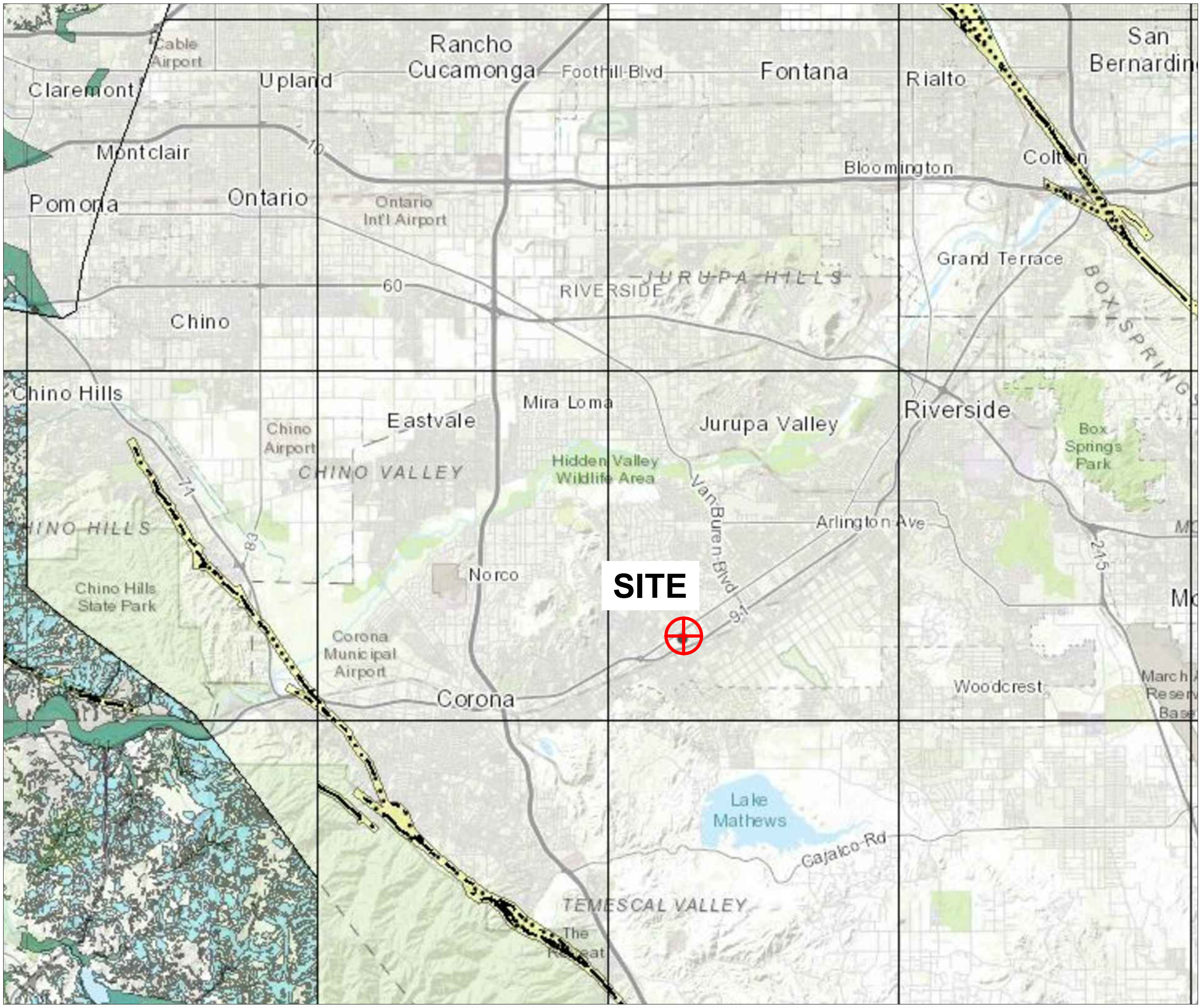
Notes: 1. See Figure 3 and Table 1 for well locations.
2. Data is from WMWD (2012) and CDWR (2019) before and after the end of 2011, respectively.

TWINING

WATER LEVELS IN WELLS ADJACENT TO THE SITE

PROPOSED PARKING STRUCTURE
KAISER RIVERSIDE MEDICAL CENTER
10800 MAGNOLIA AVENUE
RIVERSIDE, CALIFORNIA

PROJECT NO.	DATE	FIGURE 4
190191.3	March 2021	



MAP EXPLANATION

EARTHQUAKE FAULT ZONES

Earthquake Fault Zones
Zone boundaries are delineated by straight-line segments; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

Active Fault Traces
1906 C ? ?
Faults considered to have been active during Holocene time and to have potential for surface rupture: Solid Line in Black or Red where Accurately Located; Long Dash in Black or Solid Line in Purple where Approximately Located; Short Dash in Black or Solid Line in Orange where Inferred; Dotted Line in Black or Solid Line in Rose where Concealed; Query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.

SEISMIC HAZARD ZONES

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.

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

N

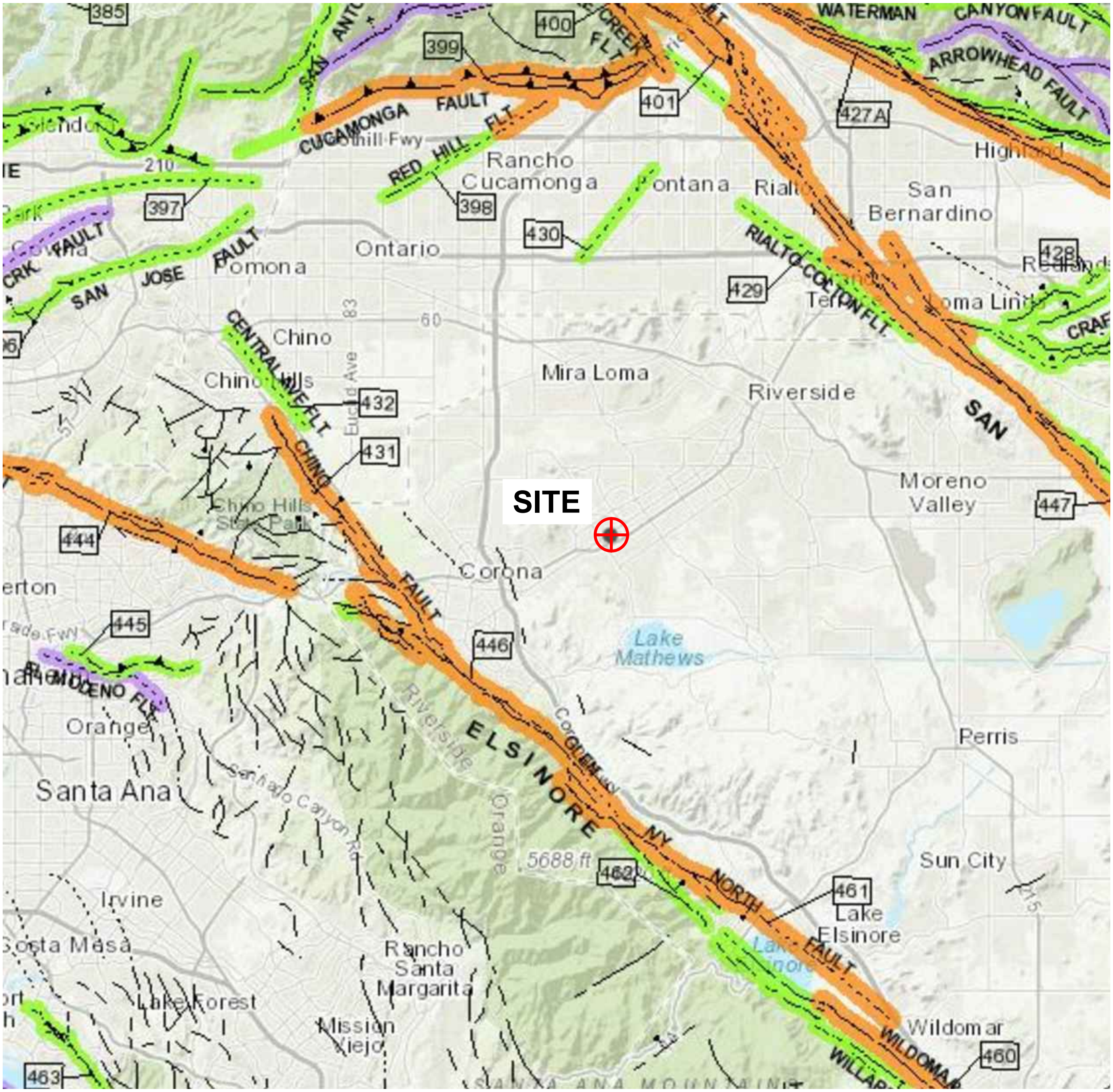
APPROXIMATE SCALE IN MILES
0 4 8

TWINING

SEISMIC HAZARD ZONES MAP

PROPOSED PARKING STRUCTURE
KAISER RIVERSIDE MEDICAL CENTER
10800 MAGNOLIA AVENUE
RIVERSIDE, CALIFORNIA

PROJECT NO.	REPORT DATE	FIGURE 5
190919.3	March 2021	



FAULT ALONG WHICH HISTORIC DISPLACEMENT HAS OCCURRED

HOLOCENE FAULT DISPLACEMENT

LATE QUATERNARY FAULT DISPLACEMENT

QUATERNARY FAULT DISPLACEMENT

PRE-QUATERNARY FAULT DISPLACEMENT

N

SCALE IN MILES

0 4 8

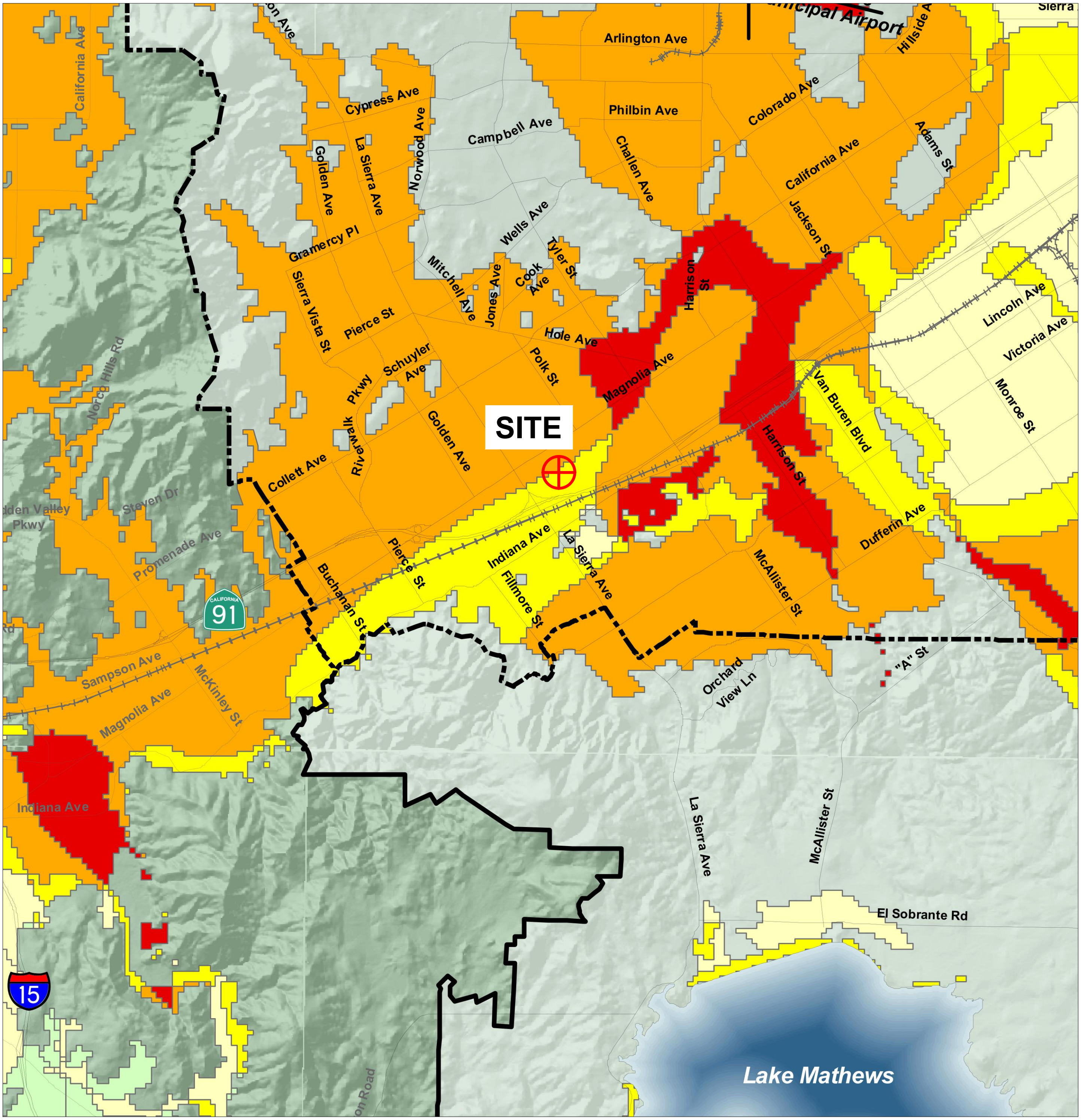
REFERENCE: JENNINGS AND BRYANT (2010)

TWINING

REGIONAL FAULT MAP

PROPOSED PARKING STRUCTURE
KAISER RIVERSIDE MEDICAL CENTER
10800 MAGNOLIA AVENUE
RIVERSIDE, CALIFORNIA

PROJECT NO.	REPORT DATE	FIGURE 6
190919.3	March 2021	



LEGEND

- VERY LOW
- LOW
- MODERATE
- HIGH
- VERY HIGH
- RIVERSIDE CITY BOUNDARY
- RIVERSIDE PROPOSED SPHERE OF INFLUENCE

APPROXIMATE SCALE IN MILES

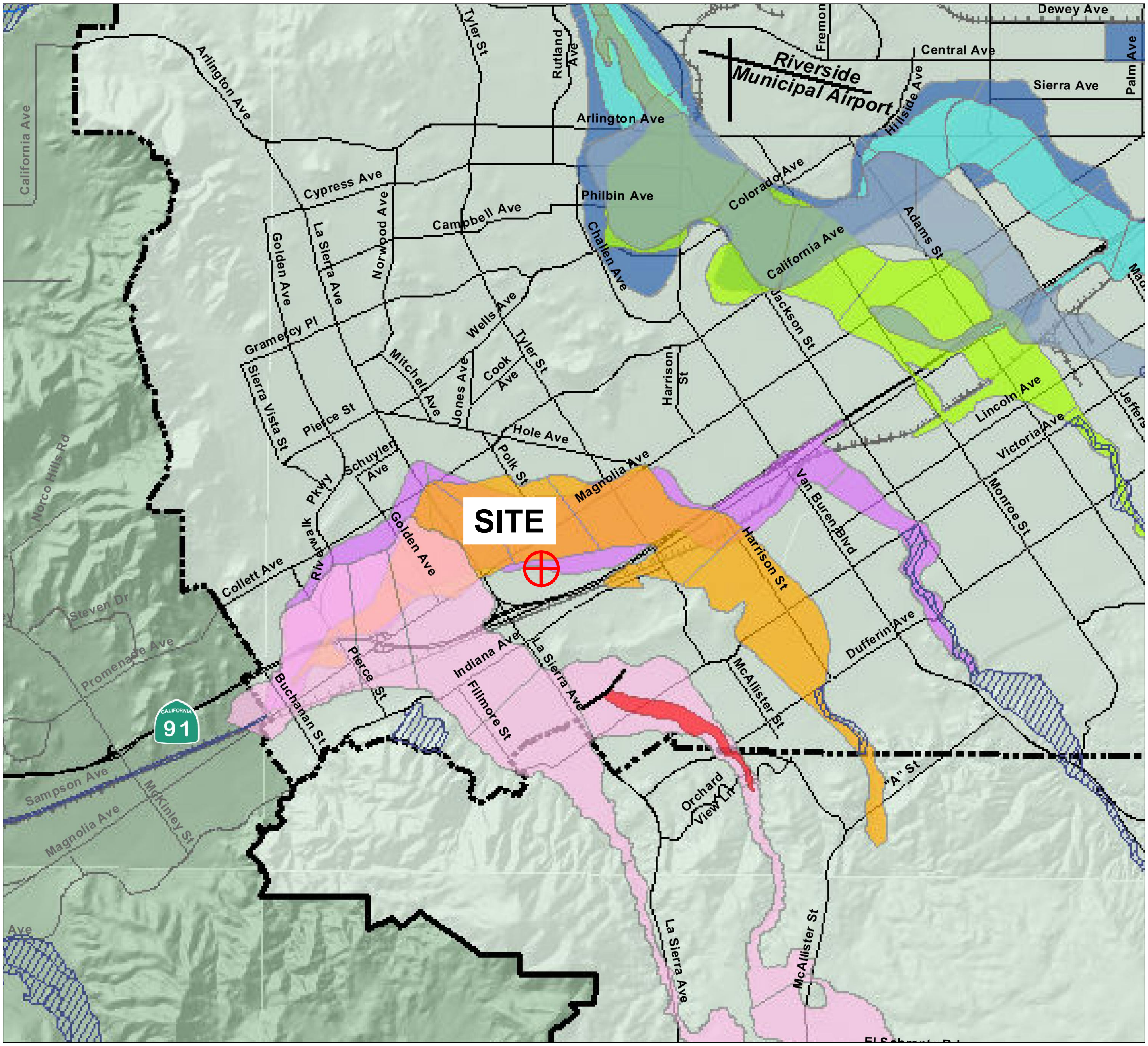
0 1 2

TWINING

LIQUEFACTION ZONES MAP

PROPOSED PARKING STRUCTURE
KAISER RIVERSIDE MEDICAL CENTER
10800 MAGNOLIA AVENUE
RIVERSIDE, CALIFORNIA

PROJECT NO.	REPORT DATE	FIGURE 7
190919.3	March 2021	



LEGEND

FLOOD HAZARD AREAS

- 1 % ANNUAL CHANCE OF FLOOD
- 0.2 % ANNUAL CHANCE OF FLOOD
- DEPARTMENT OF WATER RESOURCES AWARENESS FLOODPLAIN

DAM INUNDATION AREAS

- SYCAMORE CANYON DAM
- BOX SPRINGS DAM
- PRENDA DAM
- WOODCREST DAM
- MARY ST DAM
- ALESSANDRO DAM
- LAKE MATHEWS DAM (FORMERLY CAJALCO DAM)
- HARRISON DAM
- MOCKINGBIRD CANYON DAM
- FAIRMOUNT DAM
- RIVERSIDE CITY LIMITS
- RIVERSIDE SPHERE OF INFLUENCE

N

APPROXIMATE SCALE IN MILES

0 1 2

TWINING

DAM INUNDATION AREAS MAP

PROPOSED PARKING STRUCTURE
KAISER RIVERSIDE MEDICAL CENTER
10800 MAGNOLIA AVENUE
RIVERSIDE, CALIFORNIA

PROJECT NO.	REPORT DATE	FIGURE 8
190919.3	March 2021	

TWINING

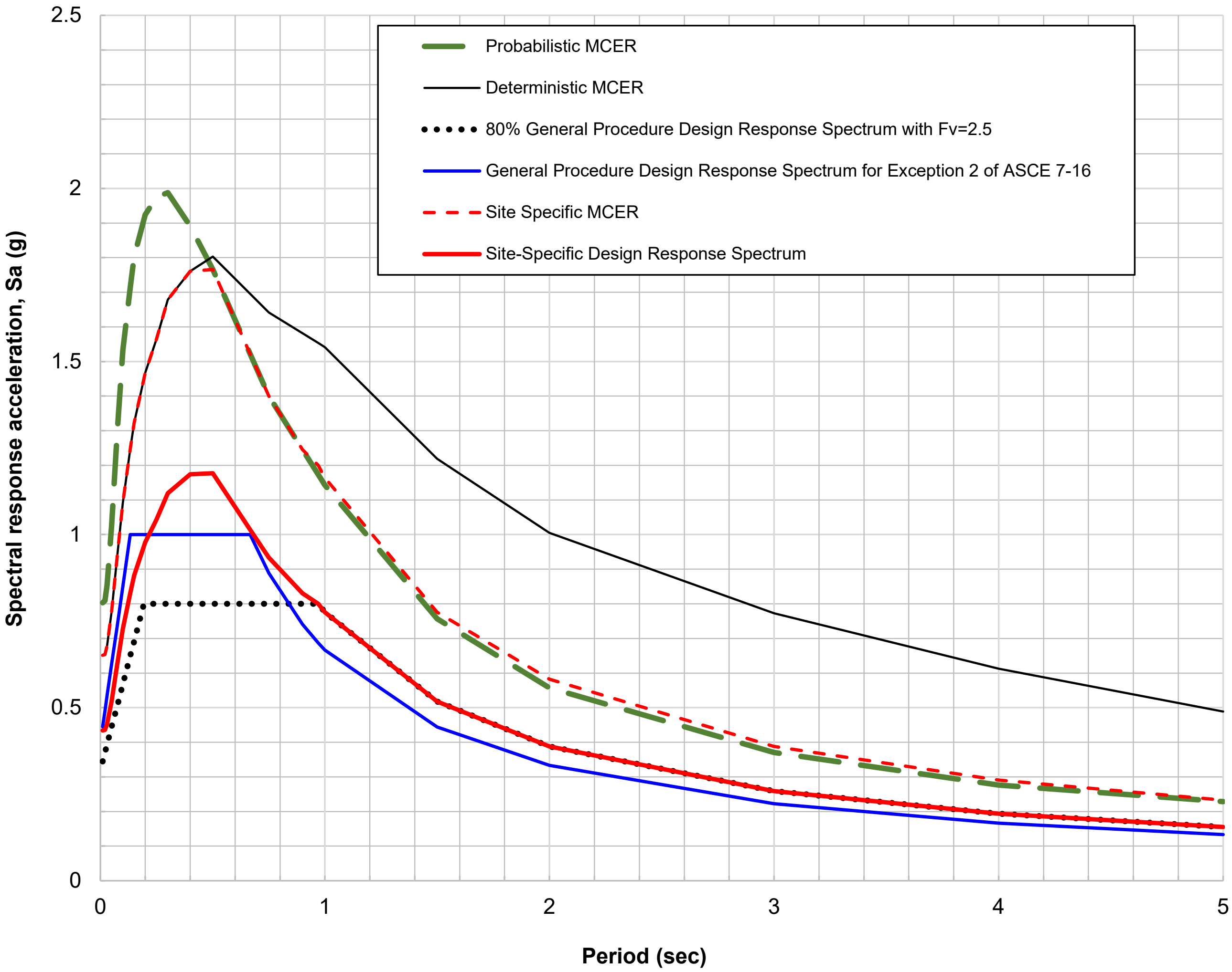
FEMA FLOOD MAP

PROPOSED PARKING STRUCTURE
KAISER RIVERSIDE MEDICAL CENTER
10800 MAGNOLIA AVENUE
RIVERSIDE, CALIFORNIA

PROJECT NO.	REPORT DATE	FIGURE 9
190919.3	March 2021	

PERIOD (seconds)	SITE-SPECIFIC DESIGN SPECTRAL ACCELERATION Sa, (g)
0.01	0.434
0.02	0.436
0.03	0.452
0.05	0.517
0.075	0.627
0.1	0.728
0.133	0.831
0.15	0.882
0.194	0.966
0.2	0.978
0.25	1.043
0.3	1.119

PERIOD (seconds)	SITE-SPECIFIC DESIGN SPECTRAL ACCELERATION Sa, (g)
0.4	1.174
0.5	1.177
0.667	1.014
0.75	0.933
0.9	0.830
0.97	0.800
1	0.776
1.5	0.517
2	0.388
3	0.259
4	0.194
5	0.155



Note: See Table 4 of the report for ordinates of the various curves.

TWINING

SITE-SPECIFIC DESIGN RESPONSE SPECTRUM

PROPOSED PARKING STRUCTURE
KAISER RIVERSIDE MEDICAL CENTER
10800 MAGNOLIA AVENUE
RIVERSIDE, CALIFORNIA

PROJECT NO.	DATE	FIGURE 10
190191.3	March 2021	

TWINING

2883 East Spring Street
Suite 300
Long Beach CA 90806

Tel 562.426.3355
Fax 562.426.6424

APPENDIX A
FIELD EXPLORATION

TWINING

2883 East Spring Street
Suite 300
Long Beach CA 90806

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Appendix A
Field Exploration

General

The subsurface exploration program for the proposed project consisted of drilling, testing, sampling and logging four hollow-stem-auger (HSA) exploratory borings (PS-1 through PS-4) and percolation testing in two hand-auger borings (P-1 and P-2) at the site between December 7, 2019 and February 12, 2020.

The HSA Borings (PS-1 through PS-4) were advanced to depths of approximately 31½ to 51½ feet below ground surface (bgs). Drilling operation for the HSA borings was performed by 2R drilling of Chino, California using a truck-mounted CME-75 drill rig equipped with 8-inch diameter hollow-stem-auger. Borings P-1 and P-2 were advanced to depths of approximately 6.5 and 6 feet bgs, respectively, using a 5-inch diameter hand auger.

The approximate locations of the borings are shown on Figure 2.

Drilling and Sampling

An explanation of the boring logs is presented as Figure A-1. The boring logs are presented as Figures A-2 through A-7. The boring logs show the earth materials encountered, samples obtained, and the field and laboratory tests performed. The logs also show the boring number, drilling date, and the name of the logger and drilling subcontractor. The borings were logged by a Twining field engineer using the Unified Soil Classification System under the supervision of a registered California Geotechnical Engineer. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Drive and bulk samples of representative earth materials were obtained from the borings.

Disturbed samples were obtained from select depths using a Standard Penetration Test (SPT) sampler. This sampler consists of a 2-inch O.D., 1.4-inch I.D. split barrel shaft without room for liner. Soil samples obtained by the SPT sampler were retained in plastic bags. A California modified sampler was also used to obtain drive samples of the soils from select depths. This sampler consists of a 3-inch outside diameter (O.D.), 2.4-inch inside diameter (I.D.) split barrel shaft. The samples were retained in brass rings for laboratory testing.

When the boring was drilled to a select depth, the sampler was lowered to the bottom of the boring and then driven a total of 18-inches into the soil using an automatic hammer weighing 140 pounds dropped from a height of 30 inches. The number of blows required to drive the samplers the final 12 inches is presented on the boring logs.

No groundwater was encountered in any of the borings. Upon completion of the borings or percolation testing, the boreholes were backfilled with drilled soil cuttings, and the surface was repaired to match existing conditions.

Percolation Testing

Percolation testing was performed on February 12, 2020 in the hand auger borings (P-1 and P-2) in accordance with the procedures of the Riverside County Design Handbook for Low Impact Development Best Management Practices. After installing pipe and filter rock, the boreholes were filled with water to approximately one foot bgs and presoaked for two consecutive 25-minute sessions prior to testing. At the end of each presoak session, more than 6 inches of water level drop was observed in the borings.

After presoaking, the boreholes were filled with water again, and measurements were recorded. The last reading was used to determine the percolation rate at each test location.

Our calculated infiltration rates with a factor safety of 3 are presented in Table A-1 below. Detailed test data is attached at the end of this appendix.

Table A-1 – Infiltration Rates with a Factor of Safety of 3

Test Location	Depth of Test Borehole (feet)	Infiltration Rate (inch/hour)
P-1	6	0.3
P-2	6.5	0.7

UNIFIED SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	**GRAVEL AND GRAVELLY SOILS** MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	**CLEAN GRAVELS** (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	**CLEAN SANDS** (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	**SILTS AND CLAYS**	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

COARSE-GRAINED SOILS			FINE-GRAINED SOILS	
Relative Density	**SPT (blows/ft)**	**Relative Density (%)**	**Consistency**	**SPT (blows/ft)**
Very Loose	<4	0 - 15	Very Soft	<2
Loose	4 - 10	15 - 35	Soft	2 - 4
Medium Dense	10 - 30	35 - 65	Medium Stiff	4 - 8
Dense	30 - 50	65 - 85	Stiff	8 - 15
Very Dense	>50	85 - 100	Very Stiff	15 - 30
			Hard	>30

NOTE: SPT blow counts based on 140 lb. hammer falling 30 inches

Sample Symbol	Sample Type	Description
	SPT	1.4 in I.D., 2.0 in. O.D. driven sampler
	California Modified	2.4 in. I.D., 3.0 in. O.D. driven sampler
	Bulk	Retrieved from soil cuttings
	Thin-Walled Tube	Pitcher or Shelby Tube

LABORATORY TESTING ABBREVIATIONS

ATT	Atterberg Limits
C	Consolidation
CORR	Corrosivity Series
DS	Direct Shear
EI	Expansion Index
GS	Grain Size Distribution
K	Permeability
MAX	Moisture/Density (Modified Proctor)
O	Organic Content
RV	Resistance Value
SE	Sand Equivalent
SG	Specific Gravity
TX	Triaxial Compression
UC	Unconfined Compression

TWINING

EXPLANATION FOR LOG OF BORINGS

Proposed Parking Structure
Kaiser Permanente Riverside Medical Center
Riverside, California

PROJECT NO.	REPORT DATE	
190919.3	March 2021	FIGURE A-1

DATE DRILLED	12/8/19	LOGGED BY	DHC	**BORING NO.**	**PS-1**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	N/E
DRILLING METHOD	8" HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	723 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	U.S.C.S. CLASSIFICATION	DESCRIPTION
	0	X					CORR, RV	CL	3 inches of asphalt concrete over 4 inches of base over 4 inches of soil over 2 inches of asphalt concrete Sandy lean CLAY; reddish brown; slightly moist
718	5		X	36	13.5	118.8	C, DS	CL	-- same; very stiff; with some tan mottling
713	10		X	15			ATT	CL	-- same; very stiff
708	15		X	39	18.8	108.3		ML	SILT with sand; very stiff; brown with khaki mottling; slightly moist; some caliche nodules
703	20		X	26				SM	Silty SAND; medium dense; light brown; slightly moist; some interbedded clay
698	25		X	35	15.1	118.5	C	CL	Sandy lean CLAY; very stiff; brown; slightly moist
693	30		X	14				SP-SM	Poorly graded SAND with silt; medium dense; tan with black, red, and brown; slightly moist; some thin interbedded clay
688	35								

BORING LOG 190919.3 - KAISER RIVERSIDE TOWER.GPJ TWINING LABS.GDT 3/24/21

TWINING

LOG OF BORING

Proposed Parking Structure
Kaiser Permanente Riverside Medical Center
Riverside, California

PROJECT NO.	REPORT DATE	
190919.3	March 2021	FIGURE A - 2

DATE DRILLED	12/8/19	LOGGED BY	DHC	**BORING NO.**	**PS-1**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	N/E
DRILLING METHOD	8" HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	723 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Driven	28	15.5	107.3			ML	SILT with sand; very stiff; khahki with orange oxidation staining; slightly moist
683	40	Driven	30					SM	Silty SAND; medium dense; light brown; slightly moist
678	45	Driven	80	1.9	109.7			SP-SM	Poorly graded SAND with silt; very dense; tan with orange, black, and red; slightly moist
673	50	Driven	34					SP-SM	-- same; dense
									Total Depth = 51.5 feet Backfilled on 12/8/2019 Groundwater not encountered. Borehole filled with cuttings at completion.
668	55								
663	60								
658	65								
653	70								

TWINING

LOG OF BORING

Proposed Parking Structure
Kaiser Permanente Riverside Medical Center
Riverside, California

PROJECT NO.	REPORT DATE	FIGURE A - 2
190919.3	March 2021	

DATE DRILLED	12/7/19	LOGGED BY	DHC	**BORING NO.**	**PS-2**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	N/E
DRILLING METHOD	8" HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	723 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
										4 inches of asphalt concrete over 4 inches of base
									CL	Sandy lean CLAY; reddish brown; dry to slightly moist
718	5		X	--	5.7	110.3	C		CL	-- same; hammer interrupted
713	10		▯	18			#200, ATT		CL	-- same; very stiff; yellowish tan
708	15		X	71	1.1	122.4			SP	Poorly graded SAND; very dense; light tan; dry to slightly moist; with approximately 5% gravel
703	20		▯	31			#200, ATT		SM	Silty SAND; dense; brown; dry to slightly moist
698	25		X	80	6.6	117.9			CL	Sandy lean CLAY; hard; brown; slightly moist
693	30		▯	33					SP-SM	Poorly graded SAND; with silt; dense; light brown; slihgtly moist
										Total Depth = 31.5 feet Backfilled on 12/7/2019 Groundwater not encountered. Borehole filled with cuttings at completion.
688	35									

TWINING

LOG OF BORING

Proposed Parking Structure
Kaiser Permanente Riverside Medical Center
Riverside, California

PROJECT NO.	REPORT DATE	
190919.3	March 2021	FIGURE A - 3

DATE DRILLED	12/8/19	LOGGED BY	DHC	**BORING NO.**	**PS-3**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	N/E
DRILLING METHOD	8" HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	723 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
										4.5 inches of asphalt concrete over 2.5 inches of base
									CL	Lean CLAY with sand; reddish brown; slightly moist
718	5		X	41	12.9	119.8	UC		CL	-- same; very stiff; reddish brown with some grey mottling
713	10		|	21			#200, ATT		CL	-- same; very stiff
708	15		X	26	11.6	94.2	C		ML	Sandy SILT; very stiff; tan with white mottling; dry to slightly moist; some caliche nodules
703	20		|	26			#200, ATT		ML	-- same; very stiff; some interbedded silty sand
698	25		X	41	12.6	120.4	UC		CL	Lean CLAY with sand; hard; brown with tan mottling; slightly moist
693	30		|	27					SM	Silty SAND; medium dense; light brown; dry to slightly moist
										Total Depth = 31.5 feet Backfilled on 12/8/2019 Groundwater not encountered. Borehole filled with cuttings at completion.
688	35									

BORING LOG 190919.3 - KAISER RIVERSIDE TOWER.GPJ TWINING LABS.GDT 3/24/21

TWINING

LOG OF BORING

Proposed Parking Structure
Kaiser Permanente Riverside Medical Center
Riverside, California

PROJECT NO.	REPORT DATE	
190919.3	March 2021	FIGURE A - 4

DATE DRILLED	12/7/19	LOGGED BY	DHC	**BORING NO.**	**PS-4**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	N/E
DRILLING METHOD	8" HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	723 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
	0									3 inches of asphalt concrete over 5 inches of base
		X			12.2		DS, EI, MAX		CL	Sandy lean CLAY; reddish brown; dry to slightly moist
718	5		X	27			ATT		CL	-- same; very stiff
713	10		X	50 for 5"	8.6	126.2			ML	Sandy SILT; hard; light brown with some white mottling; some caliche nodules
708	15		X	24					ML	-- same; very stiff
									SM	Silty SAND; light brown
703	20		X	71	11.1	111.9			ML	Sandy SILT; hard; brown with some orange oxidation; slightly moist
698	25		X	26					CL	Sandy lean CLAY; very stiff; brown with tan mottling; slightly moist; some caliche nodules
693	30		X	33	21.2	105.5			ML	Sandy SILT; very stiff; brown with tan mottling; slightly moist; some caliche nodules
688	35									

BORING LOG 190919.3 - KAISER RIVERSIDE TOWER.GPJ TWINING LABS.GDT 3/24/21

TWINING

LOG OF BORING

Proposed Parking Structure
Kaiser Permanente Riverside Medical Center
Riverside, California

PROJECT NO.	REPORT DATE	
190919.3	March 2021	FIGURE A - 5

DATE DRILLED	12/7/19	LOGGED BY	DHC	**BORING NO.**	**PS-4**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	N/E
DRILLING METHOD	8" HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	723 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
				15					CL	Sandy lean CLAY; stiff; brown; slightly moist
683	40			68	1.8	109.3			SP-SM	Poorly graded SAND with silt; dense; light brown with black, tan, and orange; dry to slightly moist; with some fine gravel
678	45			41			#200		SP-SM	-- same; dense
673	50			40/50 for 5"	2.2	112.8			SP-SM	-- same; very dense
										Total Depth = 51.5 feet Backfilled on 12/7/2019 Groundwater not encountered. Borehole filled with cuttings at completion.
668	55									
663	60									
658	65									
653	70									

BORING LOG 190919.3 - KAISER RIVERSIDE TOWER.GPJ TWINING LABS.GDT 3/24/21

TWINING

LOG OF BORING

Proposed Parking Structure
Kaiser Permanente Riverside Medical Center
Riverside, California

PROJECT NO.	REPORT DATE	FIGURE A - 5
190919.3	March 2021	

DATE DRILLED	2/12/2020	LOGGED BY	DHC	**BORING NO.**	**P-1**
DRIVE WEIGHT		DROP		DEPTH TO GROUNDWATER (ft.)	N/E
DRILLING METHOD	5" Hand Auger	DRILLER	Twining, Inc.	SURFACE ELEVATION (ft.)	723 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
								CL	Sandy lean CLAY; reddish brown; slightly moist
718	5								
									Total Depth = 6.0 feet Backfilled on 2/12/2020 Groundwater not encountered. Hole backfilled with cuttings.
713	10								
708	15								
703	20								
698	25								
693	30								
688	35								

TWINING

LOG OF BORING

Proposed Parking Structure
Kaiser Permanente Riverside Medical Center
Riverside, California

PROJECT NO.	REPORT DATE	FIGURE A - 6
190919.3	March 2021	

DATE DRILLED	2/12/2020	LOGGED BY	DHC	**BORING NO.**	**P-2**
DRIVE WEIGHT		DROP		DEPTH TO GROUNDWATER (ft.)	N/E
DRILLING METHOD	5" Hand Auger	DRILLER	Twining, Inc.	SURFACE ELEVATION (ft.)	723 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
								CL	Sandy lean CLAY; reddish brown; slightly moist
718	5								
									Total Depth = 6.5 feet Backfilled on 2/12/2020 Groundwater not encountered. Hole backfilled with cuttings.
713	10								
708	15								
703	20								
698	25								
693	30								
688	35								

TWINING

LOG OF BORING

Proposed Parking Structure
Kaiser Permanente Riverside Medical Center
Riverside, California

PROJECT NO.	REPORT DATE	
190919.3	March 2021	FIGURE A - 7

Infiltration Rate Calculation Sheet

Project :	Kaiser Riverside Medical Cntr		Project No. :	190919.3		Date :	3/31/2021
Test Hole No.:		P-1	Tested by :	DHC			
Depth of Test Hole, D_T (in):		72	USCS Soil Classification :		CL		
Test Hole Dimension (inches)					Length	Width	
Diameter (if round) (inches) =		6.5	Sides (if rectangular) =				

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6" ? (Y/N)
1	9:00 AM	9:25 AM	25	12.0	30.0	18.0	Y
2	9:25 AM	9:40 AM	15	13.2	27.6	14.4	Y

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

			Δt	H_o	H_f	ΔH	
Trial No.	**Start Time**	**Stop Time**	**Time Interval (min.)**	**Initial Water Height (inches)**	**Final Water Height (inches)**	**Change in Water Level (inches)**	**Tested Infiltration Rate**
1	9:40 AM	9:50 AM	10	58.20	50.40	7.80	1.36
2	9:50 AM	10:00 AM	10	60.00	54.00	6.00	1.00
3	10:00 AM	10:10 AM	10	60.00	55.20	4.80	0.79
4	10:10 AM	10:20 AM	10	60.00	55.20	4.80	0.79
5	10:20 AM	10:30 AM	10	60.00	55.20	4.80	0.79
6	10:30 AM	10:40 AM	10	61.20	56.40	4.80	0.77
7							
8							
9							
10							
11							
12							
13							
14							
15							

Infiltration Rate with a factor of safety of 3 = 0.3 inch /hr

Infiltration Rate Calculation Sheet

Project :	Kaiser Riverside Medical Cntr		Project No. :	190919.3		Date :	3/31/2021
	Test Hole No.:	P-2	Tested by :	DHC			
Depth of Test Hole, D_T (in):		78	USCS Soil Classification :		CL		
Test Hole Dimension (inches)					Length	Width	
Diameter (if round) (inches) =		6.5	Sides (if rectangular) =				

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6" ? (Y/N)
1	10:55 AM	11:20 AM	25	12.0	30.0	18.0	Y
2	11:20 AM	11:45 AM	25	13.2	31.8	18.6	Y

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

			Δt	H_o	H_f	ΔH	
Trial No.	**Start Time**	**Stop Time**	**Time Interval (min.)**	**Initial Water Height (inches)**	**Final Water Height (inches)**	**Change in Water Level (inches)**	**Tested Infiltration Rate**
1	11:50 AM	12:00 PM	10	60.00	48.00	12.00	2.10
2	12:02 PM	12:12 PM	10	70.80	54.00	16.80	2.56
3	12:12 PM	12:22 PM	10	67.20	51.60	15.60	2.49
4	12:22 PM	12:32 PM	10	67.20	54.00	13.20	2.07
5	12:32 PM	12:42 PM	10	66.00	54.00	12.00	1.90
6	12:42 PM	12:52 PM	10	68.40	54.00	14.40	2.23
7							
8							
9							
10							
11							
12							
13							
14							
15							

Infiltration Rate with a factor of safety of 3 = 0.7 inch /hr

TWINING

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APPENDIX B
LABORATORY TESTING

TWINING

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Appendix B
Laboratory Testing

Laboratory Moisture Content and Density Tests

The moisture content and dry densities of selected driven samples obtained from the exploratory borings were evaluated in general accordance with the latest version of ASTM D 2937. The results are shown on the boring logs in Appendix A, and also summarized in Table B-1.

No. 200 Wash Sieve

The amount of fines passing the No. 200 sieve was evaluated in accordance with ASTM D 1140. The results are presented in Table B-2.

Atterberg Limits

Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System. The test results are summarized in on Figure B-1 and Table B-3.

Resistance Value (R-value)

R-value testing was performed on a select bulk sample of the near-surface soils encountered at the site. The test was performed in general accordance with ASTM D 2844. The result is summarized in Table B-4.

Expansion Index

The expansion index of a select soil sample was evaluated in general accordance with ASTM D 4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimen was loaded with a surcharge of 144 pounds per square foot and was inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The result of expansion index test is presented in Table B-5.

Maximum Density and Optimum Moisture

Modified Proctor testing was performed on near-surface soils to determine the maximum dry density and optimum water content for compaction. The tests were performed in accordance with ASTM D 1557 Method A. The curves are attached to this appendix as Figure B-2.

Direct Shear

Direct shear tests were performed on a remolded sample and a select modified-California soil sample in general accordance with the latest version of ASTM D 3080 to evaluate the shear strength characteristics of the selected materials. The remolded sample was prepared to a relative compaction of 90% according to the maximum density as determined by ASTM D1557. The samples were inundated during shearing to represent adverse field conditions. Test results are presented on Figures B-3 and B-4.

Consolidation

Consolidation tests were performed on select modified-California soil samples in general accordance with the latest version of ASTM D2435. The samples were inundated during testing to represent adverse field conditions. The percent consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The testing was performed by the geotechnical laboratory of Twining and the laboratory of Hushmand Associates, Inc. (HAI) of Irvine, California. The results of the tests by Twining are presented in Figure B-5 and those by HAI in HAI's laboratory test sheets and graphs included in this appendix.

Unconfined Compression

Unconfined compression (UC) testing was conducted to assess unconfined compression strength of site soils. The testing was performed using strain-controlled application of the axial load on representative relatively undisturbed samples. The testing was performed by the laboratory of Hushmand Associates, Inc. (HAI) of Irvine, California in general accordance with ASTM D2166. At the time of testing, the moisture content and dry density of each sample were measured. Stress-strain measurements were also plotted for the UC tests. Test results are presented in HAI's laboratory test sheets and graphs included in this appendix. The UC strengths of the samples are summarized on Table B-6.

Corrosivity

Soil pH and resistivity tests were performed by Anaheim Test Lab, Inc. (ATLI) of Anaheim, California on a representative soil sample. The resistivity of the soil assumes saturated soil conditions. The chloride and sulfate contents of the selected samples were evaluated in general accordance with the latest versions of Caltrans test methods CT417, CT422, and CT 643. The test results are presented on Table B-7 and the ATLI report included in this appendix.

TWINING

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Table B-1
Moisture Content and Dry Density

Boring No.	Depth (feet)	Moisture Content (%)	Dry Density (pcf)
PS-1	5	13.5	118.1
PS-1	15	18.8	108.3
PS-1	25	15.1	118.5
PS-1	35	15.5	107.3
PS-1	45	1.9	109.7
PS-2	5	5.7	110.3
PS-2	15	1.1	122.4
PS-2	25	6.6	117.9
PS-3	5	12.9	119.8
PS-3	15	11.6	94.2
PS-3	25	12.6	120.4
PS-4	10	8.6	126.2
PS-4	20	11.1	111.9
PS-4	30	21.2	105.5
PS-4	40	1.8	109.3
PS-4	50	2.2	112.8

Table B-2
Number 200 Wash Results

Boring No.	Depth (feet)	Percent Passing #200
PS-2	10	55.3
PS-2	20	28.5
PS-3	10	71.8
PS-3	20	50.5
PS-4	45	5.2

Table B-3
Atterberg Limits Results

Boring No.	Depth (feet)	Liquid Limit	Plastic Limit	Plasticity Index	U.S.C.S. Classification
PS-1	10	33	17	16	Sandy Lean Clay (CL)
PS-2	10	33	22	11	Sandy Lean Clay (CL)
PS-2	20	NP	NP	NP	Silty Sand (SM)
PS-3	10	36	17	19	Lean Clay with Sand (CL)
PS-3	20	NP	NP	NP	Sandy Silt (ML)
PS-4	5	35	15	20	Sandy Lean Clay (CL)

Table B-4
Resistance Value (R-value)

Boring No.	Depth (feet)	R Value
PS-1	0 – 5	18

Table B-5
Expansion Index

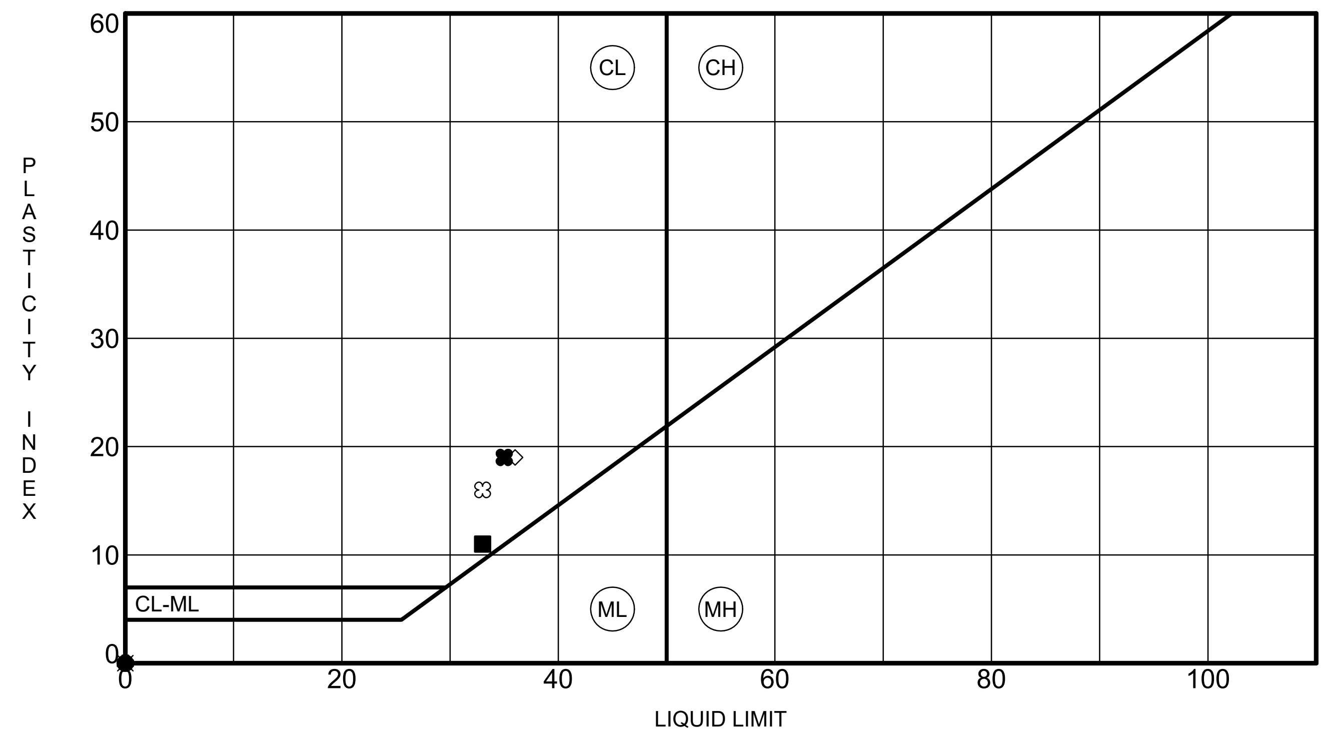
Boring No.	Depth (feet)	Expansion Index	Expansion Potential
PS-4	0 – 5	2	Very low

Table B-6
Unconfined Compression Test Results

Boring No.	Depth (feet)	Soil Classification	Unconfined Compression Strength, q_u, (psf)
PS-3	5	Lean Clay with Sand (CL)	9,435
PS-3	25	Lean Clay with Sand (CL)	7,114

Table B-7
Corrosivity Test Results

Boring No.	Depth (feet)	pH	Water Soluble Sulfate (ppm)	Water Soluble Chloride (ppm)	Minimum Resistivity (ohm-cm)
PS-1	0-5	7.6	267	118	3,700



Sample Location	LL	PL	PI	U.S.C.S. Classification
PS-1 at 10 ft	**33**	**17**	**16**	**Sandy lean CLAY**
PS-2 at 10 ft	**33**	**22**	**11**	**Sandy lean CLAY**
PS-2 at 20 ft	**NP**	**NP**	**NP**	**Silty SAND**
PS-3 at 10 ft	**36**	**17**	**19**	**Lean CLAY with sand**
PS-3 at 20 ft	**NP**	**NP**	**NP**	**Sandy SILT**
PS-4 at 5 ft	**35**	**16**	**19**	**Sandy lean CLAY**

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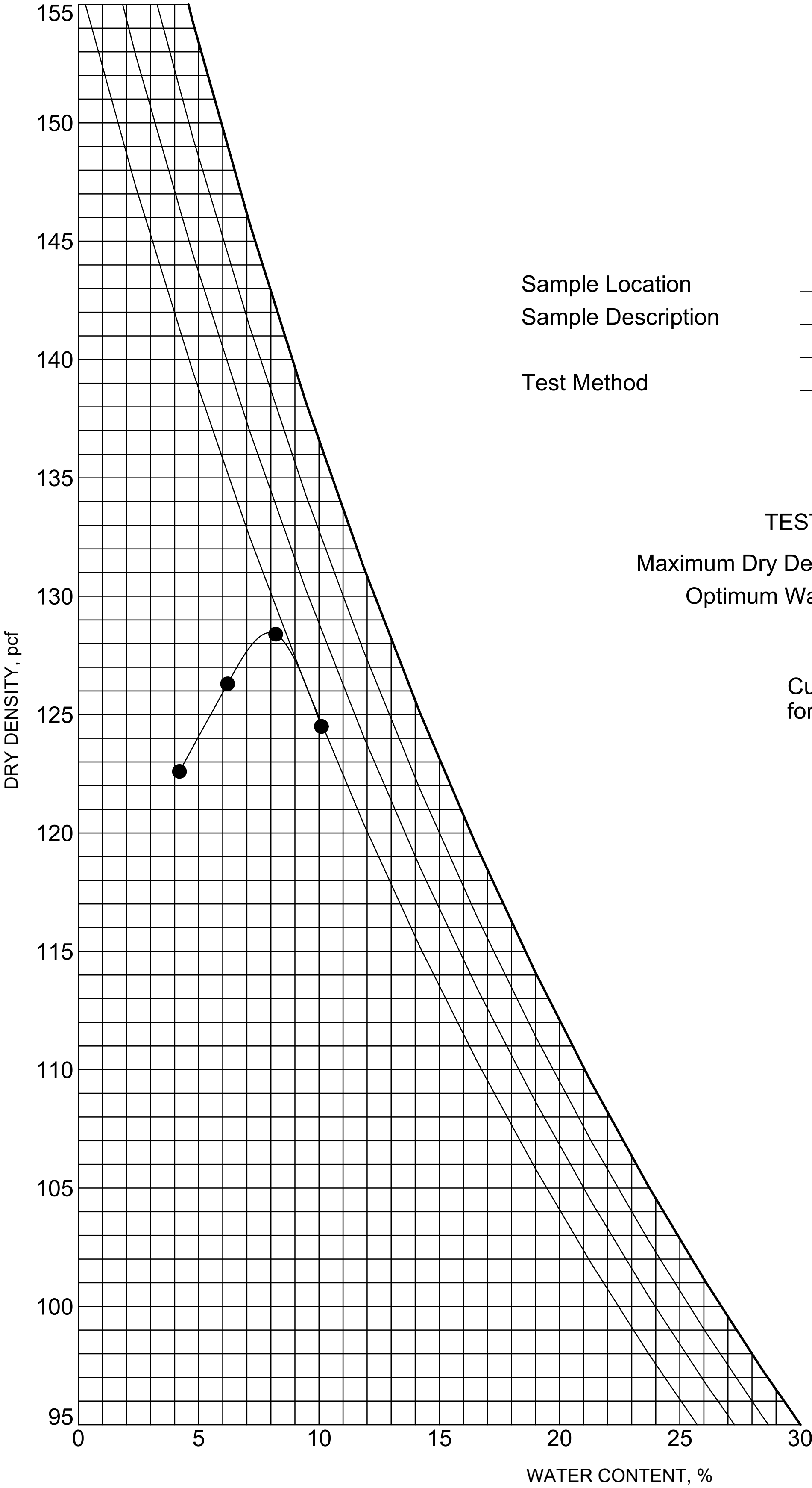
ATTERBERG LIMITS

Proposed Parking Structure
Kaiser Permanente Riverside Medical Center
Riverside, California

PROJECT NO.	REPORT DATE	FIGURE B-1
190919.3	March 2021	



Sample Location	**PS-4 at 0-5 ft**
Sample Description	**Sandy lean CLAY**
Test Method	**ASTM D1557 Method A**

TEST RESULTS

Maximum Dry Density **128.5** **pcf**

Optimum Water Content **8.0** **%**

Curves of 100% Saturation
for Specific Gravity Equal to:

2.80

2.70

2.60

2.50

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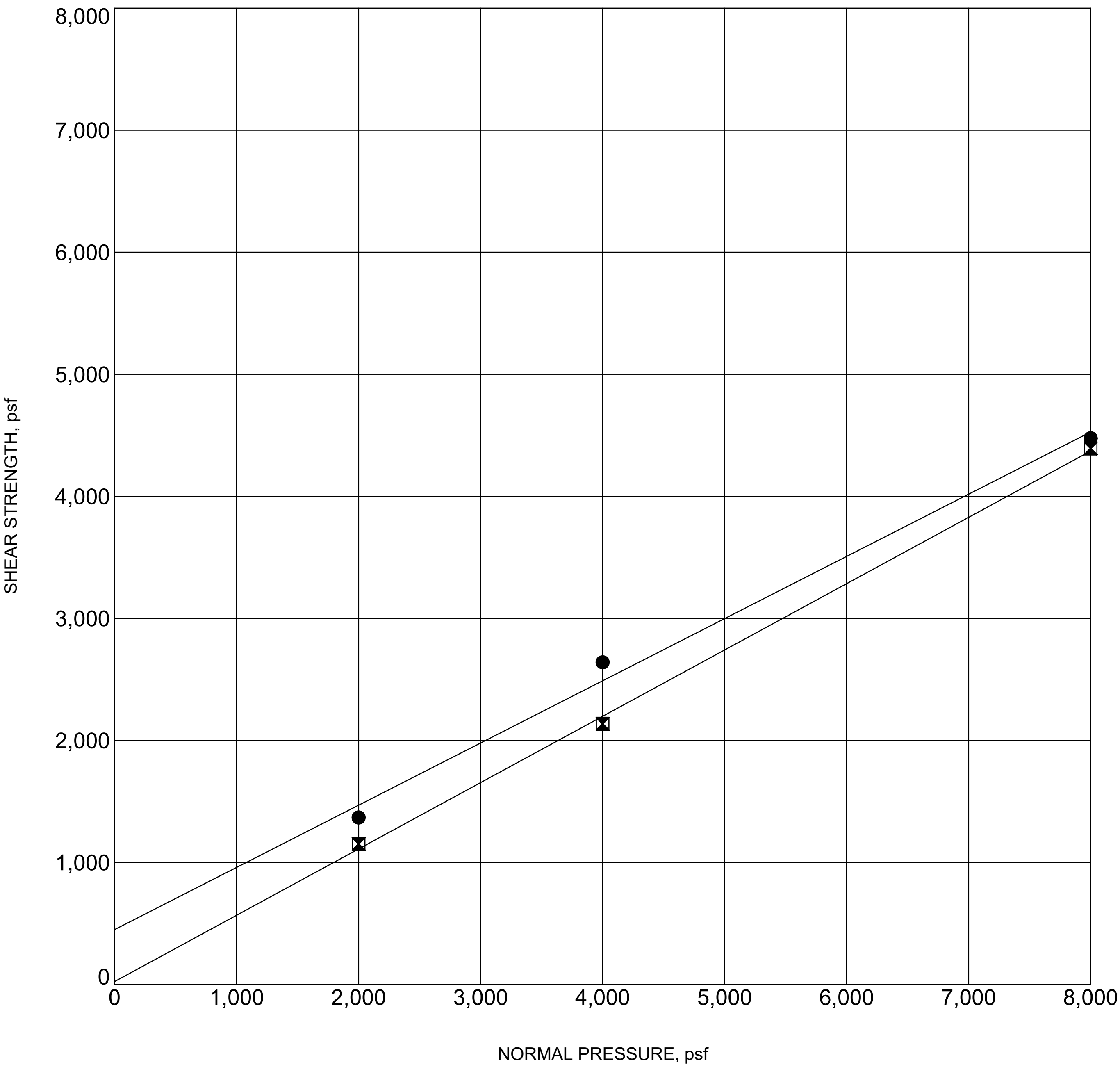
MOISTURE-DENSITY RELATIONSHIP

Proposed Parking Structure
Kaiser Permanente Riverside Medical Center
Riverside, California

PROJECT NO.	REPORT DATE	FIGURE B-2
190919.3	March 2021	



Boring No.: PS-1
Sample Depth (ft): 5
Sample Description: Sandy lean CLAY
Strain Rate (in./min): 0.005
Dry Density (pcf): 118.8

Shear Strength Parameters

	Peak	Ultimate
Cohesion, C (psf):	450	24
Friction Angle, Ø (deg):	27	29

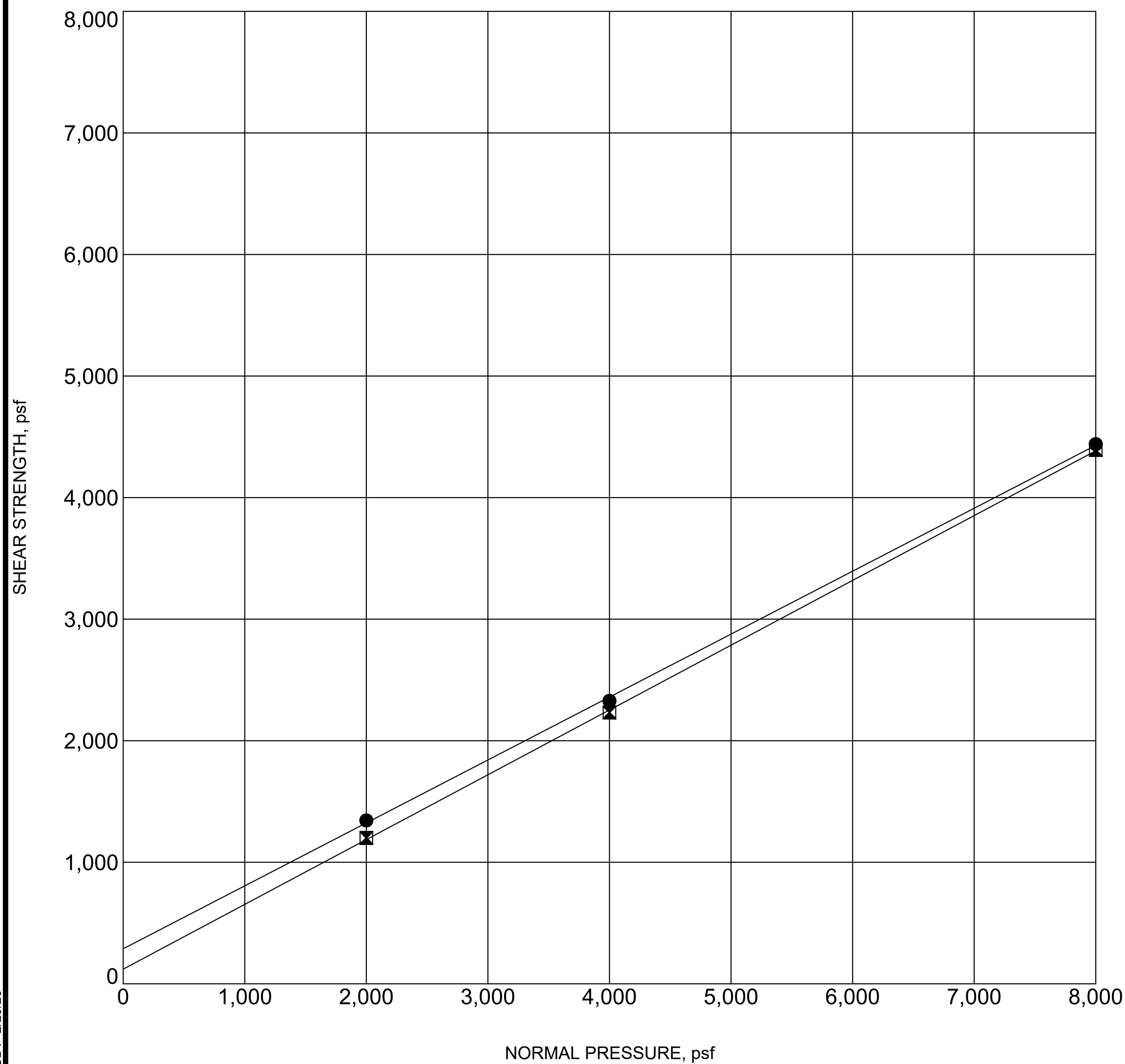
Initial Moisture (%): 13.5
Final Moisture (%): 22.6

TWINING

DIRECT SHEAR TEST

Proposed Parking Structure
Kaiser Permanente Riverside Medical Center
Riverside, California

PROJECT NO.	REPORT DATE	FIGURE B-3
190919.3	March 2021	



Shear Strength Parameters

Boring No.:	PS-4		**Peak**	**Ultimate**
Sample Depth (ft):	0-5 remolded to 90% relative RC	**Cohesion, C (psf):**	288	120
Sample Description:	Sandy lean CLAY	**Friction Angle, Ø (deg):**	27	28
Strain Rate (in./min):	0.005			
Dry Density (pcf):	115.7	**Initial Moisture (%):**		
		Final Moisture (%):	12.2	

DIRECT SHEAR 190919.3 - KAISER RIVERSIDE TOWER.GPJ TWINING LABS.GDT 2/20/20

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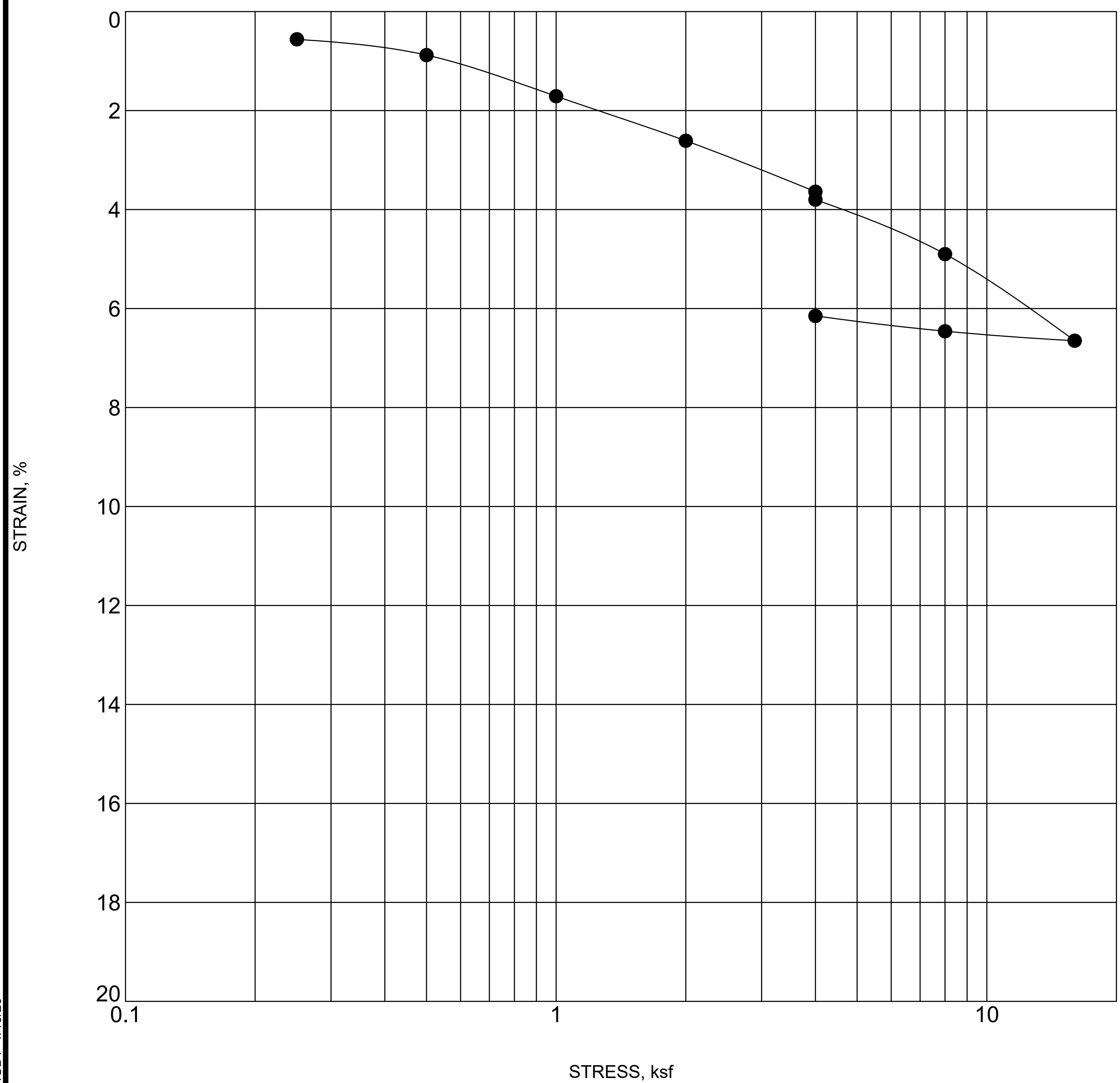
DIRECT SHEAR TEST

Proposed Parking Structure
Kaiser Permanente Riverside Medical Center
Riverside, California

PROJECT NO.	REPORT DATE	FIGURE B-4
190919.3	March 2021	



	Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
●	**PS-1 at 5 ft**	**Sandy lean CLAY**	**118.8**	**13.5**

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CONSOLIDATION TEST

Proposed Parking Structure
Kaiser Permanente Riverside Medical Center
Riverside, California

PROJECT NO.	REPORT DATE	
190919.3	March 2021	FIGURE B-5

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Hushmand Associates, Inc.
250 Goddard, Irvine,
CA 92618

p. (949) 777-1274
w. haieng.com
e. hai@haieng.com

December 27, 2019

Twining Inc.
3310 East Airport Way,
Long Beach, CA 90806

Attention: Mr. Brian Vollnogle

SUBJECT: Laboratory Test Result
Project Name: RMC
Project No.: 190919.3
HAI Project No.: TWI-19-013

Dear Mr. Vollnogle:

Enclosed is the result of the laboratory testing program conducted on samples from the above referenced project. The testing performed for this program was conducted in general accordance with the following test procedure:

Type of Test	Test Procedure
Moisture Content & Dry Density	ASTM D2216 & D2937
Consolidation	ASTM D2435
Unconfined Compression	ASTM D2166

Attached are: eight (8) Moisture Content & Dry Density test results; four (4) Consolidation test results; and eight (8) Unconfined Compression test results.

We appreciate the opportunity to provide our testing services to Twining Inc. If you have any questions regarding the test results, please contact us.

Sincerely,

HUSHMAND ASSOCIATES, INC.

Kang C. Lin, BS, EIT
Laboratory Manager

Ashkaan Hushmand, PhD, PE
Project Engineer

MOISTURE CONTENT AND DRY DENSITY OF RING SAMPLES

ASTM D2216 & ASTM D2937

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client: Twining Inc.
Project Name: RMC
Project No.: 190919.3

HAI Proj No.: TWI-19-013
Performed by: KL
Checked by: AH
Date: 12/13/2019

No.	Boring No.	Sample No.	Depth	Wt of Ring + Soil	Height of Sample	Dia. of Sample	Volume of Sample	Wt of Rings	Wt of Soil	Wet Density	Wt of Cont. + Wet Soil	Wt of Cont. + Dry Soil	Wt of Container	Moisture Content	Dry Density
			ft	gr	in	in	cu.ft	gr	gr	pcf	gr	gr	gr	%	pcf
4	PS-1	R	25	838.06	4.00	2.416	0.0106	181.44	656.62	136.4	172.05	150.98	11.81	15.1	118.5
5	PS-2	R	5	557.02	3.00	2.416	0.0080	136.08	420.94	116.6	168.55	160.09	10.98	5.7	110.3
6	PS-3	R	5	820.92	5.06	2.413	0.0134	0.00	820.92	135.2	835.71	742.28	16.42	12.9	119.8
7	PS-3	R	15	859.10	5.00	2.416	0.0133	226.80	632.30	105.1	158.41	144.35	23.06	11.6	94.2
8	PS-3	R	25	811.97	4.98	2.416	0.0132	0.00	811.97	135.5	827.03	736.58	16.05	12.6	120.4

CONSOLIDATION TEST

ASTM D2435

Client :	Twining Inc.	*HAI Project No.:*	TWI-19-013
Project Name:	RMC	*Tested by:*	KL
Project Number:	190919.3	*Checked by:*	MJ
Boring No.:	PS-1	*Date:*	12/13/19
Sample No.:	R		
Type of Sample:	Undisturbed Ring		
Depth (ft):	25		
Soil Description:	Brown, Clayey Sand (SC)		

Initial Total Weight	Final Total Weight	Final Dry Weight
(g)	(g)	(g)
164.65	162.95	143.23

			Initial Conditions	Final Conditions
Height	H	(in)	1.004	0.969
Height of Solids	H_s	(in)	0.706	0.706
Height of Water	H_w	(in)	0.285	0.262
Height of Air	H_a	(in)	0.013	0.000
Dry Density		**(pcf)**	**118.5**	**126.8**
Water Content		**(%)**	**15.0**	**13.8**
Saturation		**(%)**	**95.7**	**100.0**

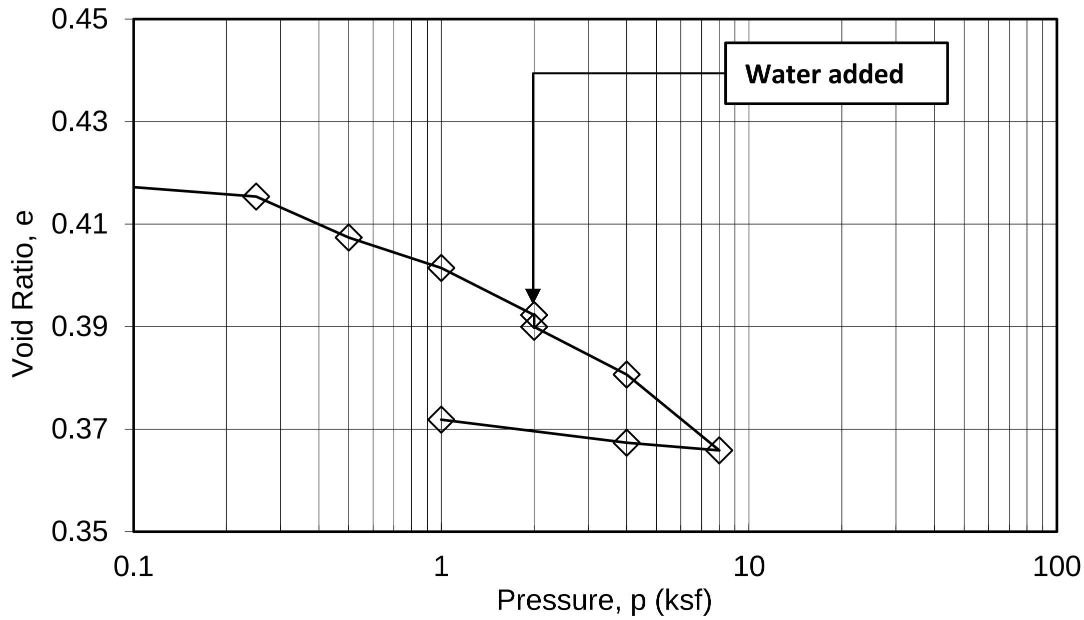
* Saturation is calcualted based on Gs= 2.70

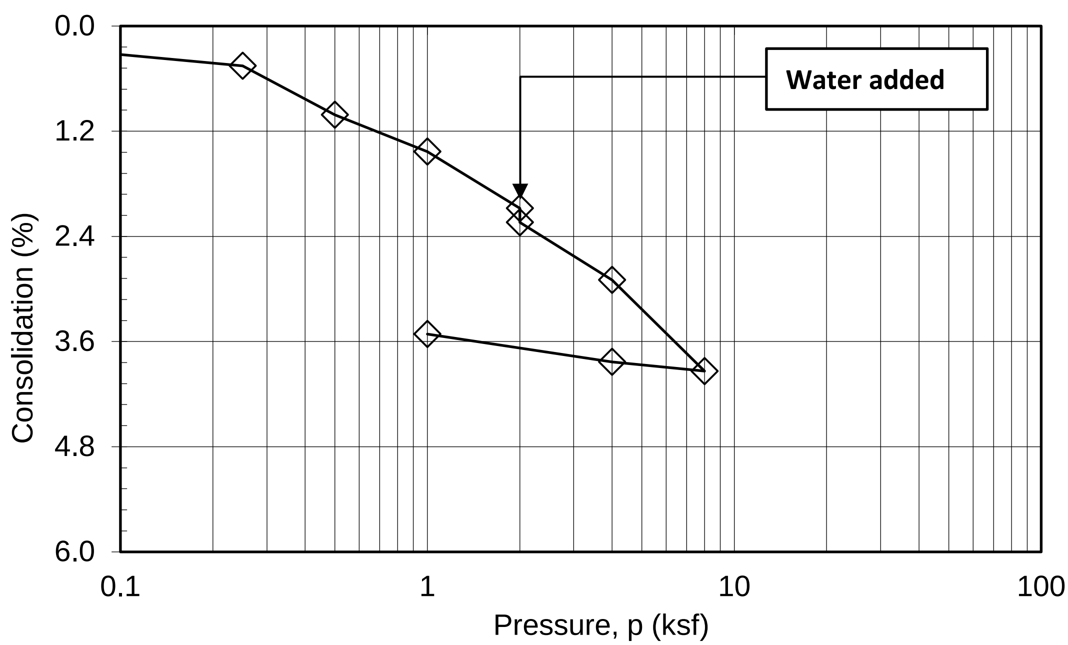
Load (ksf)	**δH (in)**	**H (in)**	**Voids (in)**	**e**	**Consol. (%)**	**a_v (ksf^{-1})**	**M_v (ksf^{-1})**	**Comment**
0.01	-------	1.0040	0.298	0.422	0			
0.25	0.0045	0.9995	0.293	0.415	0.5	2.7E-02	1.9E-02	
0.5	0.0102	0.9938	0.288	0.407	1.0	3.2E-02	2.3E-02	
1	0.0144	0.9896	0.283	0.401	1.4	1.2E-02	8.5E-03	
2	0.0209	0.9831	0.277	0.392	2.1	9.2E-03	6.6E-03	
2	0.0225	0.9815	0.275	0.390	2.2	Water Added		
4	0.0291	0.9749	0.269	0.381	2.9	4.7E-03	3.4E-03	
8	0.0395	0.9645	0.258	0.366	3.9	3.7E-03	2.7E-03	
4	0.0385	0.9655	0.259	0.367	3.8	Unloaded		
1	0.0353	0.9687	0.263	0.372	3.5			

CONSOLIDATION TEST

ASTM D2435

Client :	Twining Inc.	*HAI Project No.:*	TWI-19-013
Project Name:	RMC	*Tested by:*	KL
Project Number:	190919.3	*Checked by:*	MJ
Boring No.:	PS-1	*Date:*	12/13/19
Sample No.:	R		
Type of Sample:	Undisturbed Ring		
Depth (ft):	25		
Soil Description:	Brown, Clayey Sand (SC)		





CONSOLIDATION TEST

ASTM D2435

Client :	Twining Inc.	*HAI Project No.:*	TWI-19-013
Project Name:	RMC	*Tested by:*	KL
Project Number:	190919.3	*Checked by:*	MJ
Boring No.:	PS-2	*Date:*	12/13/19
Sample No.:	R		
Type of Sample:	Undisturbed Ring		
Depth (ft):	5		
Soil Description:	Reddish Brown, Silty Sand (SM)		

Initial Total Weight (g)	Final Total Weight (g)	Final Dry Weight (g)
143.28	156.57	135.60

			Initial Conditions	Final Conditions
Height	H	(in)	1.023	0.994
Height of Solids	H_s	(in)	0.676	0.676
Height of Water	H_w	(in)	0.102	0.279
Height of Air	H_a	(in)	0.245	0.039
Dry Density		**(pcf)**	**110.1**	**114.0**
Water Content		**(%)**	**5.7**	**15.5**
Saturation		**(%)**	**29.5**	**87.8**

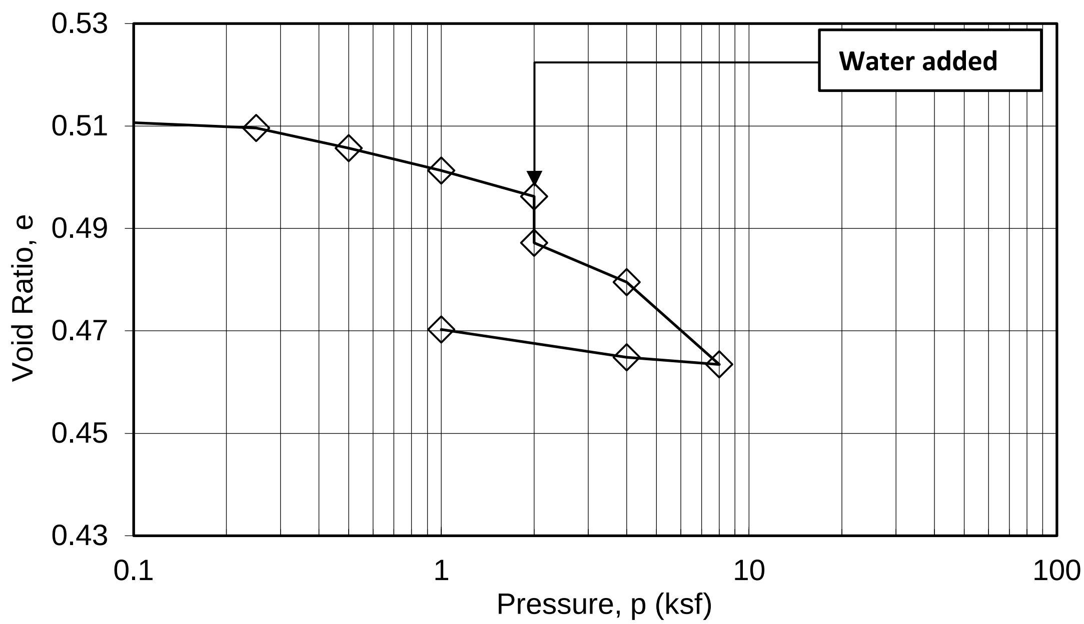
* Saturation is calcualted based on Gs= 2.67

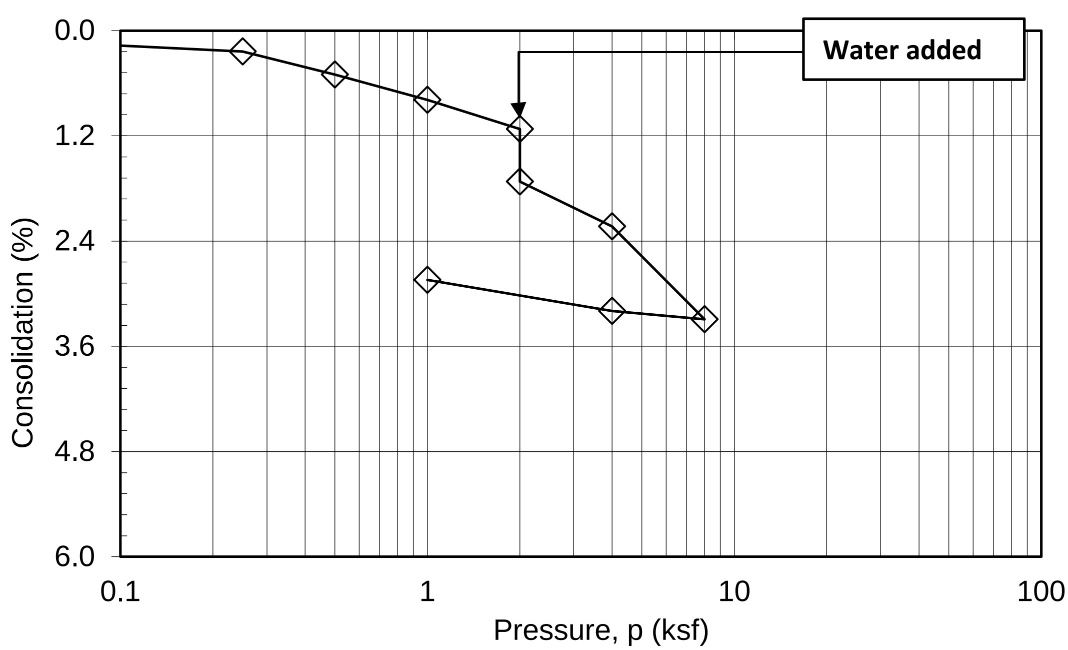
Load (ksf)	δH (in)	H (in)	Voids (in)	e	Consol. (%)	a_v (ksf^{-1})	M_v (ksf^{-1})	Comment
0.01	-------	1.0230	0.347	0.513	0			
0.25	0.0025	1.0206	0.345	0.510	0.2	1.5E-02	1.0E-02	
0.5	0.0051	1.0179	0.342	0.506	0.5	1.6E-02	1.1E-02	
1	0.0081	1.0149	0.339	0.501	0.8	8.7E-03	5.8E-03	
2	0.0115	1.0115	0.335	0.496	1.1	5.1E-03	3.4E-03	
2	0.0176	1.0054	0.329	0.487	1.7	Water Added		
4	0.0228	1.0002	0.324	0.479	2.2	3.9E-03	2.6E-03	
8	0.0337	0.9893	0.313	0.463	3.3	4.0E-03	2.7E-03	
4	0.0327	0.9903	0.314	0.465	3.2	Unloaded		
1	0.0291	0.9939	0.318	0.470	2.8			

CONSOLIDATION TEST

ASTM D2435

Client :	Twining Inc.	*HAI Project No.:*	TWI-19-013
Project Name:	RMC	*Tested by:*	KL
Project Number:	190919.3	*Checked by:*	MJ
Boring No.:	PS-2	*Date:*	12/13/19
Sample No.:	R		
Type of Sample:	Undisturbed Ring		
Depth (ft):	5		
Soil Description:	Reddish Brown, Silty Sand (SM)		





CONSOLIDATION TEST

ASTM D2435

Client :	Twining Inc.	*HAI Project No.:*	TWI-19-013
Project Name:	RMC	*Tested by:*	KL
Project Number:	190919.3	*Checked by:*	MJ
Boring No.:	PS-3	*Date:*	12/13/19
Sample No.:	R		
Type of Sample:	Undisturbed Ring		
Depth (ft):	15		
Soil Description:	Light Brown, Sandy Silt (ML)		

Initial Total Weight	Final Total Weight	Final Dry Weight
(g)	(g)	(g)
128.55	144.92	115.23

			Initial Conditions	Final Conditions
Height	H	(in)	1.016	0.959
Height of Solids	H_s	(in)	0.564	0.564
Height of Water	H_w	(in)	0.177	0.395
Height of Air	H_a	(in)	0.275	0.000
Dry Density		**(pcf)**	**94.2**	**104.0**
Water Content		**(%)**	**11.6**	**25.8**
Saturation		**(%)**	**39.2**	**99.9**

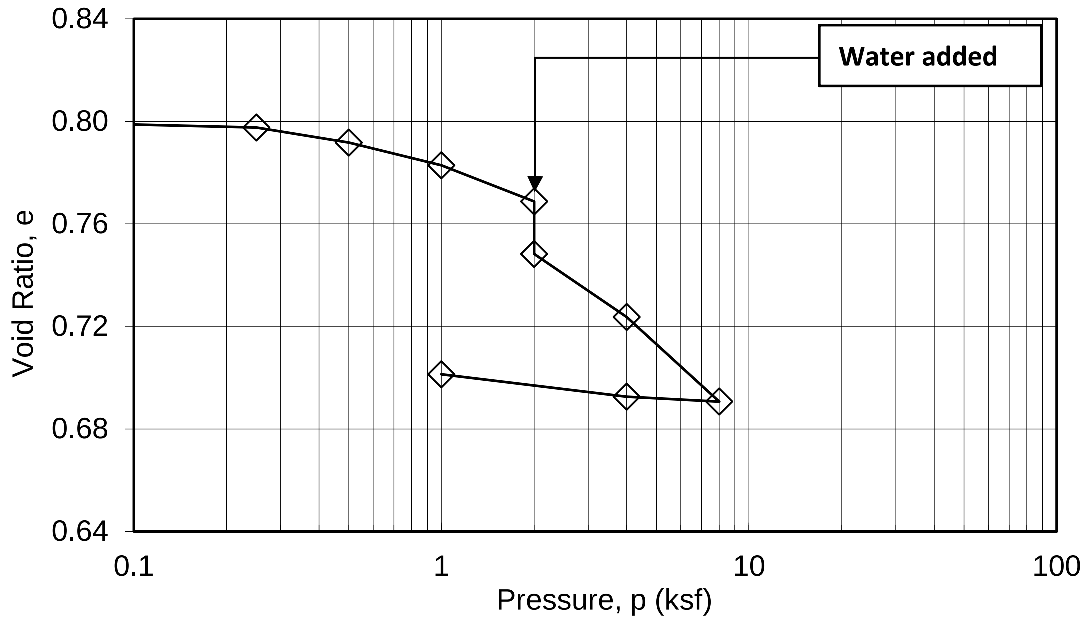
* Saturation is calcualted based on Gs= 2.72

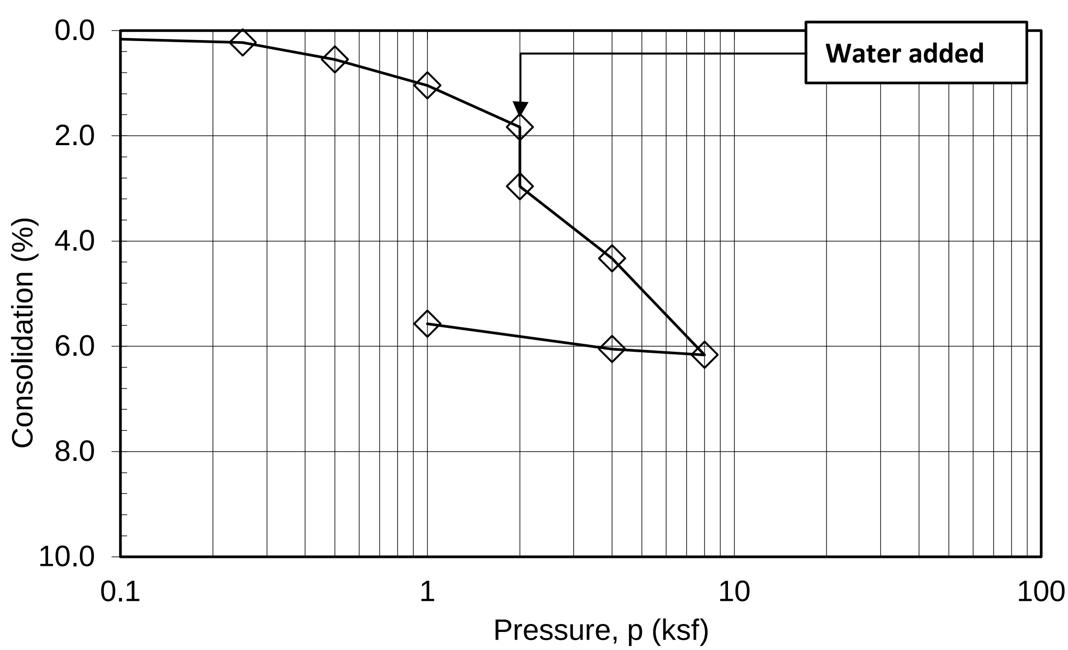
Load (ksf)	δH (in)	H (in)	Voids (in)	e	Consol. (%)	a_v (ksf^{-1})	M_v (ksf^{-1})	Comment
0.01	-------	1.0160	0.452	0.802	0			
0.25	0.0023	1.0137	0.450	0.798	0.2	1.7E-02	9.5E-03	
0.5	0.0056	1.0104	0.446	0.792	0.6	2.3E-02	1.3E-02	
1	0.0106	1.0054	0.441	0.783	1.0	1.8E-02	9.9E-03	
2	0.0186	0.9974	0.433	0.769	1.8	1.4E-02	8.0E-03	
2	0.0301	0.9859	0.422	0.748	3.0	Water Added		
4	0.0440	0.9720	0.408	0.724	4.3	1.2E-02	7.2E-03	
8	0.0626	0.9534	0.389	0.691	6.2	8.3E-03	4.9E-03	
4	0.0615	0.9545	0.391	0.693	6.1	Unloaded		
1	0.0566	0.9594	0.395	0.701	5.6			

CONSOLIDATION TEST

ASTM D2435

Client :	Twining Inc.	*HAI Project No.:*	TWI-19-013
Project Name:	RMC	*Tested by:*	KL
Project Number:	190919.3	*Checked by:*	MJ
Boring No.:	PS-3	*Date:*	12/13/19
Sample No.:	R		
Type of Sample:	Undisturbed Ring		
Depth (ft):	15		
Soil Description:	Light Brown, Sandy Silt (ML)		





UNCONFINED COMPRESSION STRENGTH TEST
ASTM D2166

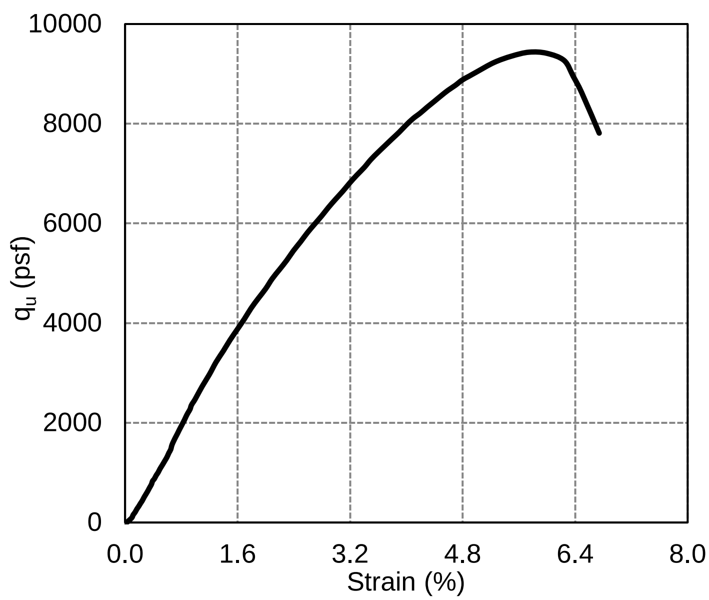
Client:	Twining Inc.	***HAI Project No.:***	TWI-19-013
Project:	RMC	***Tested by:***	KL
Project No.:	190919.3	***Checked by:***	AH
Boring No.:	PS-3	***Date:***	12/13/2019
Sample No.:	5		
Soil Description:	Borwn, Fat Clay with Sand (CH)		

1. Initial Specimen Information

Height:	(in)	5.06	Initial Wet Weight:	(g)	820.92
Diameter:	(in)	2.41	Wet Density:	(pcf)	135.2
Area:	(in^2)	4.57	**Moisture Content:**	**(%)**	**12.9**
Volume:	(in^3)	23.13	**Dry Density:**	**(pcf)**	**119.8**

2. Compression Test Data ***Curing Days:*** -

Vertical Displ. (in)	Load (lbs)	q_u (psf)	Strain (%)
0.0000	0.0	-3.1	0.00
0.0037	2.1	63.1	0.07
0.0098	10.4	322.9	0.19
0.0163	20.2	629.5	0.32
0.0211	28.1	877.4	0.42
0.0276	38.1	1190.4	0.55
0.0337	49.8	1553.5	0.67
0.0389	59.4	1853.6	0.77
0.0454	70.9	2209.6	0.90
0.0554	87.7	2729.3	1.10
0.0806	125.0	3871.2	1.59
0.1010	152.0	4686.9	2.00
0.1262	183.6	5633.5	2.50
0.1514	212.5	6488.3	3.00
0.1719	234.1	7119.3	3.40
0.1971	258.8	7829.3	3.90
0.2223	280.7	8446.9	4.40
0.2423	296.1	8873.9	4.79
0.2905	318.0	9435.3	5.75
0.3409	266.1	7811.3	6.74



Failure of the specimen

UNCONFINED COMPRESSION STRENGTH TEST
ASTM D2166

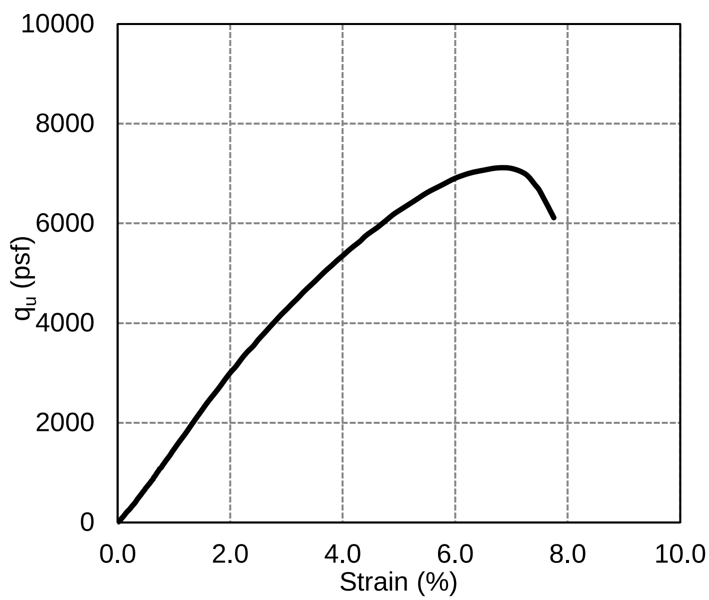
Client:	Twining Inc.	***HAI Project No.:***	TWI-19-013
Project:	RMC	***Tested by:***	KL
Project No.:	190919.3	***Checked by:***	AH
Boring No.:	PS-3	***Date:***	12/13/2019
Sample No.:	25		
Soil Description:	Borwn, Lean Clay with Sand (CL)		

1. Initial Specimen Information

Height:	(in)	4.98	Initial Wet Weight:	(g)	811.97
Diameter:	(in)	2.42	Wet Density:	(pcf)	135.5
Area:	(in^2)	4.58	**Moisture Content:**	**(%)**	**12.6**
Volume:	(in^3)	22.83	**Dry Density:**	**(pcf)**	**120.4**

2. Compression Test Data ***Curing Days:*** -

Vertical Displ. (in)	Load (lbs)	q_u (psf)	Strain (%)
0.0000	0.0	4.4	0.00
0.0034	2.4	79.9	0.07
0.0098	7.8	248.5	0.20
0.0163	13.4	425.2	0.33
0.0223	19.4	610.0	0.45
0.0287	25.3	793.5	0.58
0.0347	31.7	991.7	0.70
0.0411	37.9	1186.3	0.83
0.0475	44.5	1389.5	0.95
0.0646	62.3	1935.7	1.30
0.0899	87.6	2706.1	1.81
0.1096	106.7	3281.2	2.20
0.1348	128.3	3925.1	2.71
0.1596	148.0	4504.6	3.21
0.1844	167.0	5055.7	3.70
0.2093	184.2	5547.9	4.20
0.2341	200.2	5995.9	4.70
0.2739	222.8	6617.8	5.50
0.3364	242.7	7114.0	6.76
0.3860	210.9	6113.6	7.75



Failure of the specimen

ANAHEIM TEST LAB, INC

196 Technology Drive, Unit D
Irvine, CA 92618
Phone (949)336-6544

DATE: 12/16/2019

P.O. NO: Soils 121219

LAB NO: C-3447

SPECIFICATION: CTM-417/422/643

MATERIAL: Soil

TWINING LABS
3310 AIRPORT WAY
LONG BEACH, CA 90806

Project No.: 190919.3
Project: RMC
Date sampled: 12/08/2019

ANALYTICAL REPORT

CORROSION SERIES
SUMMARY OF DATA

pH	SOLUBLE SULFATES per CT. 417 ppm	SOLUBLE CHLORIDES per CT. 422 ppm	MIN. RESISTIVITY per CT. 643 ohm-cm
7.6	267	118	3,700

RESPECTFULLY SUBMITTED

WES BRIDGER LAB MANAGER

APPENDIX C
LIQUEFACTION ANALYSIS

2883 East Spring Street, Suite 300

TWINING Long Beach, CA 90806

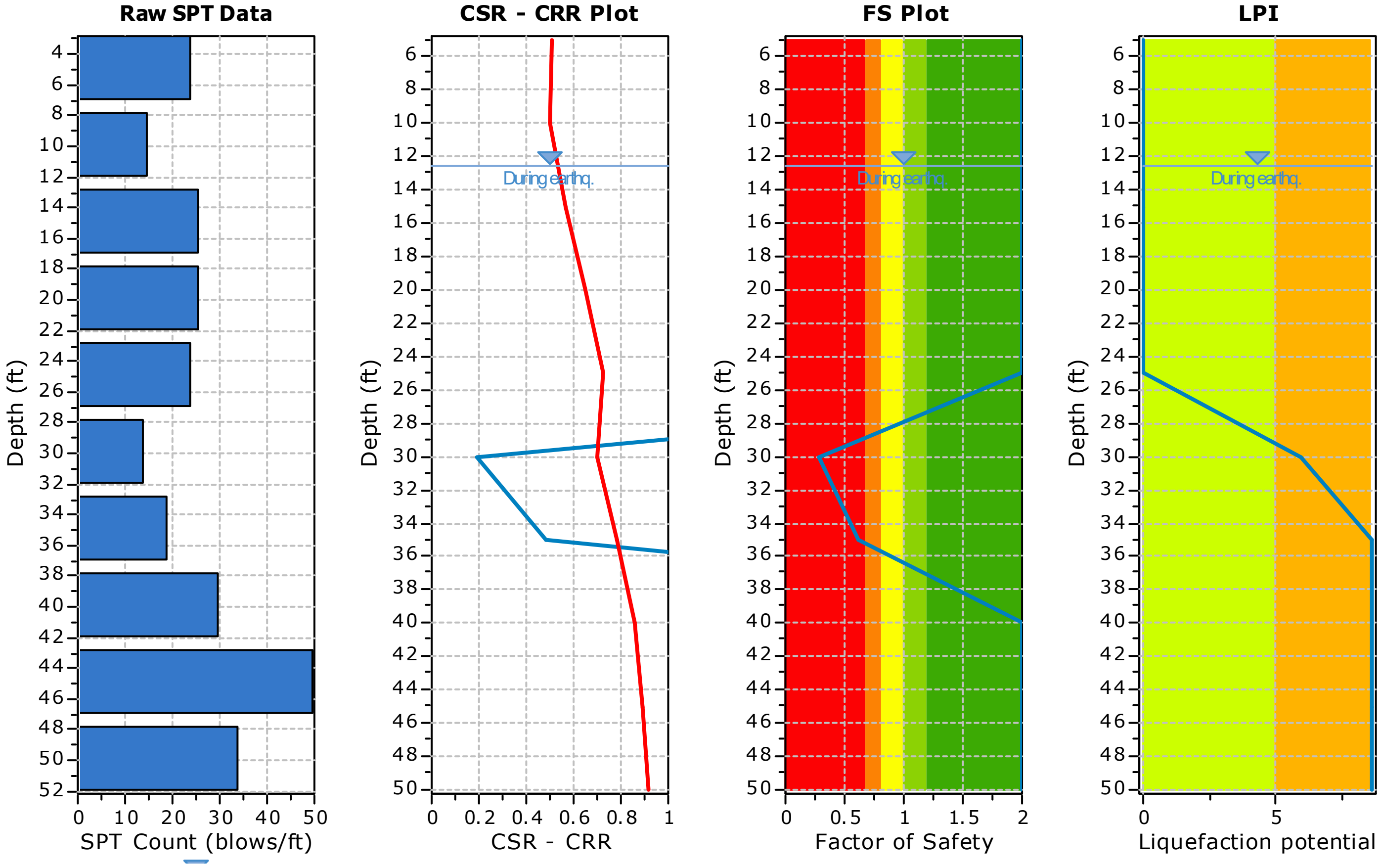
SPT BASED LIQUEFACTION ANALYSIS REPORT

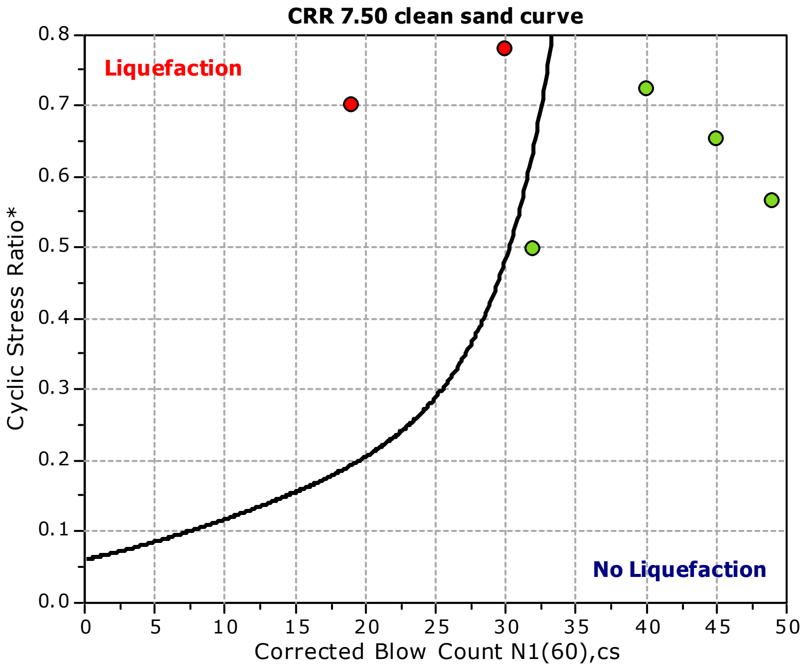
Project title : Proposed Parking Structure at Kaiser Riverside Medical Center **SPT Name: PS-1**

Location : 10800 Magnolia Avenue, Riverside, California

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	57.50 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	12.50 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	7.70
Borehole diameter:	200mm	Peak ground acceleration:	0.61 g
Rod length:	5.00 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.25		

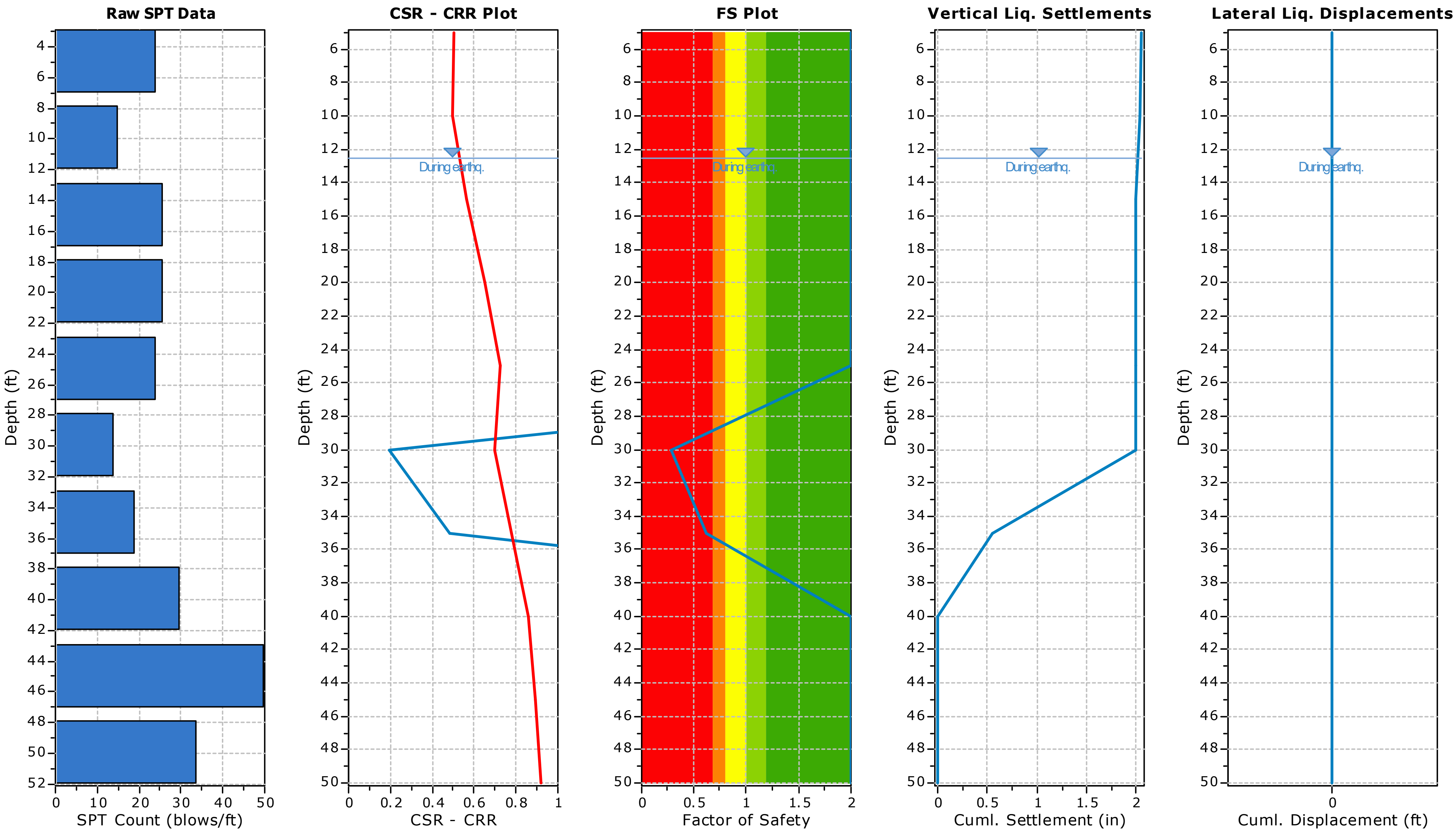




F.S. color scheme
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme
- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
5.00	24	65.00	134.80	5.00	No
10.00	15	65.00	134.80	5.00	No
15.00	26	85.00	128.70	5.00	Yes
20.00	26	28.50	128.70	5.00	Yes
25.00	24	65.00	136.00	5.00	No
30.00	14	10.00	136.00	5.00	Yes
35.00	19	85.00	124.00	5.00	Yes
40.00	30	15.00	112.00	5.00	Yes
45.00	50	5.20	112.00	5.00	Yes
50.00	34	5.20	112.00	5.00	Yes

Abbreviations

Depth:	Depth at which test was performed (ft)
SPT Field Value:	Number of blows per foot
Fines Content:	Fines content at test depth (%)
Unit Weight:	Unit weight at test depth (pcf)
Infl. Thickness:	Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy:	User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
5.00	24	134.80	0.34	0.00	0.34	0.26	1.35	1.25	1.15	0.80	1.20	45	65.00	5.59	51	4.000
10.00	15	134.80	0.67	0.00	0.67	0.35	1.17	1.25	1.15	0.85	1.20	26	65.00	5.59	32	4.000
15.00	26	128.70	1.00	0.00	1.00	0.26	1.02	1.25	1.15	0.95	1.20	43	85.00	5.53	49	4.000
20.00	26	128.70	1.32	0.00	1.32	0.27	0.94	1.25	1.15	0.95	1.20	40	28.50	5.30	45	4.000
25.00	24	136.00	1.66	0.00	1.66	0.30	0.87	1.25	1.15	0.95	1.20	34	65.00	5.59	40	4.000
30.00	14	136.00	2.00	0.00	2.00	0.45	0.75	1.25	1.15	1.00	1.20	18	10.00	1.15	19	0.194
35.00	19	124.00	2.31	0.00	2.31	0.37	0.75	1.25	1.15	1.00	1.20	24	85.00	5.53	30	0.485
40.00	30	112.00	2.59	0.00	2.59	0.28	0.78	1.25	1.15	1.00	1.20	40	15.00	3.26	43	4.000
45.00	50	112.00	2.87	0.00	2.87	0.26	0.77	1.25	1.15	1.00	1.20	66	5.20	0.00	66	4.000
50.00	34	112.00	3.15	0.00	3.15	0.28	0.74	1.25	1.15	1.00	1.20	43	5.20	0.00	43	4.000

Abbreviations

σ_v:	Total stress during SPT test (tsf)
u_o:	Water pore pressure during SPT test (tsf)
σ'_{vo}:	Effective overburden pressure during SPT test (tsf)
m:	Stress exponent normalization factor
C_N:	Overburden corretion factor
C_E:	Energy correction factor
C_B:	Borehole diameter correction factor
C_R:	Rod length correction factor
C_S:	Liner correction factor
$N_{1(60)}$:	Corrected N_{SPT} to a 60% energy ratio
$\Delta(N_1)_{60}$	Equivalent clean sand adjustment
$N_{1(60)cs}$:	Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$:	Cydic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
5.00	134.80	0.34	0.00	0.34	1.00	1.00	0.395	2.20	51	0.92	0.428	1.10	0.658	2.000	●

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
10.00	134.80	0.67	0.00	0.67	0.98	1.00	0.390	2.12	32	0.93	0.421	1.10	0.647	2.000
15.00	128.70	1.00	0.08	0.92	0.97	1.00	0.418	2.20	49	0.92	0.453	1.04	0.734	2.000
20.00	128.70	1.32	0.23	1.08	0.96	1.00	0.461	2.20	45	0.92	0.499	0.99	0.850	2.000
25.00	136.00	1.66	0.39	1.27	0.94	1.00	0.487	2.20	40	0.92	0.528	0.95	0.942	2.000
30.00	136.00	2.00	0.55	1.45	0.92	1.00	0.502	1.45	19	0.97	0.517	0.96	0.911	0.277
35.00	124.00	2.31	0.70	1.61	0.90	1.00	0.514	2.00	30	0.94	0.549	0.92	1.014	0.622
40.00	112.00	2.59	0.86	1.73	0.88	1.00	0.523	2.20	43	0.92	0.567	0.86	1.121	2.000
45.00	112.00	2.87	1.01	1.85	0.86	1.00	0.528	2.20	66	0.92	0.573	0.83	1.160	2.000
50.00	112.00	3.15	1.17	1.98	0.84	1.00	0.530	2.20	43	0.92	0.575	0.82	1.192	2.000

Abbreviations

$\sigma_{v,eq}$:	Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$:	Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$:	Effective overburden pressure, during earthquake (tsf)
r_d:	Nonlinear shear mass factor
α:	Improvement factor due to stone columns
CSR :	Cydic Stress Ratio
MSF :	Magnitude Scaling Factor
$CSR_{eq,M=7.5}$:	CSR adjusted for M=7.5
K_{sigma}:	Effective overburden stress factor
CSR^*:	CSR fully adjusted (user FS applied)***
FS:	Calculated factor of safety against soil liquefaction

*** User FS: 1.30

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
5.00	2.000	0.00	9.24	5.00	0.00
10.00	2.000	0.00	8.48	5.00	0.00
15.00	2.000	0.00	7.71	5.00	0.00
20.00	2.000	0.00	6.95	5.00	0.00
25.00	2.000	0.00	6.19	5.00	0.00
30.00	0.277	0.72	5.43	5.00	5.98
35.00	0.622	0.38	4.67	5.00	2.69
40.00	2.000	0.00	3.90	5.00	0.00
45.00	2.000	0.00	3.14	5.00	0.00
50.00	2.000	0.00	2.38	5.00	0.00

Overall potential I_L : 8.67

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	a	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
5.00	45	0.13	0.23	0.79	0.14	12291.42	0.00	0.00	17.10	0.01	5.00	0.013
10.00	26	0.26	0.45	0.95	0.15	8109.31	0.00	0.00	17.10	0.03	5.00	0.042

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)

Cumulative settlemetns: 0.055

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
15.00	49	0.06	-1.51	2.000	0.00	0.00	5.00	0.000	0.00
20.00	45	0.25	-1.19	2.000	0.00	0.00	5.00	0.000	0.00
25.00	40	0.00	0.00	2.000	0.00	0.00	5.00	0.000	0.00
30.00	19	17.78	0.57	0.277	17.78	2.40	5.00	1.441	0.00
35.00	30	4.65	-0.09	0.622	4.65	0.92	5.00	0.554	0.00
40.00	43	0.44	-1.03	2.000	0.00	0.00	5.00	0.000	0.00
45.00	66	0.00	-2.94	2.000	0.00	0.00	5.00	0.000	0.00
50.00	43	0.44	-1.03	2.000	0.00	0.00	5.00	0.000	0.00

Cumulative settlements: 1.996 0.00

Abbreviations

γ_{lim}: Limiting shear strain (%)
F_α/N: Maximun shear strain factor
γ_{max}: Maximum shear strain (%)
e_v:: Post liquefaction volumetric strain (%)
S_{v-1D}: Estimated vertical settlement (in)
LDI: Estimated lateral displacement (ft)

2883 East Spring Street, Suite 300

Long Beach, CA 90806

SPT BASED LIQUEFACTION ANALYSIS REPORT

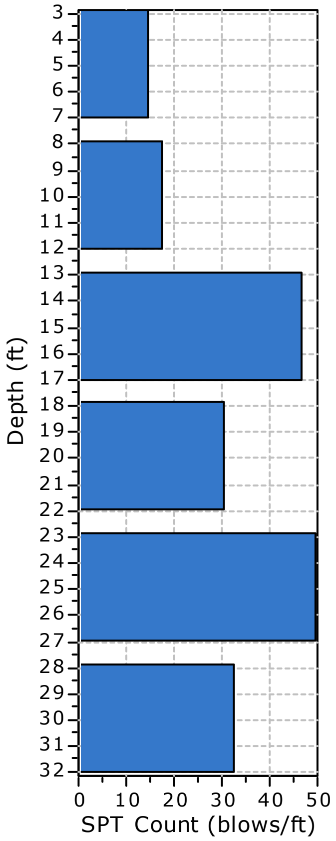
Project title : Proposed Parking Structure at Kaiser Riverside Medical Center **SPT Name: PS-2**

Location : 10800 Magnolia Avenue, Riverside, California

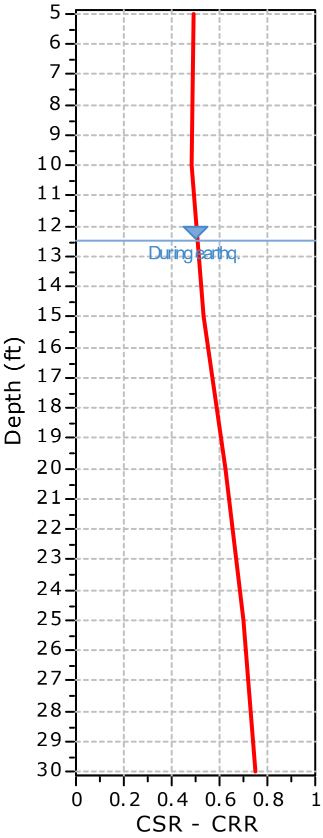
:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	57.50 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	12.50 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	7.70
Borehole diameter:	200mm	Peak ground acceleration:	0.59 g
Rod length:	5.00 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.25		

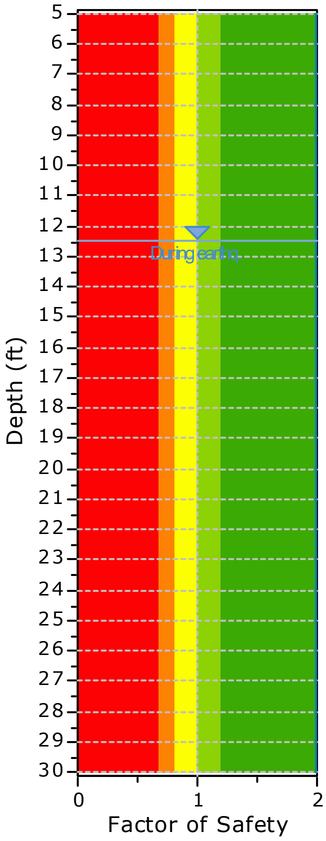
Raw SPT Data



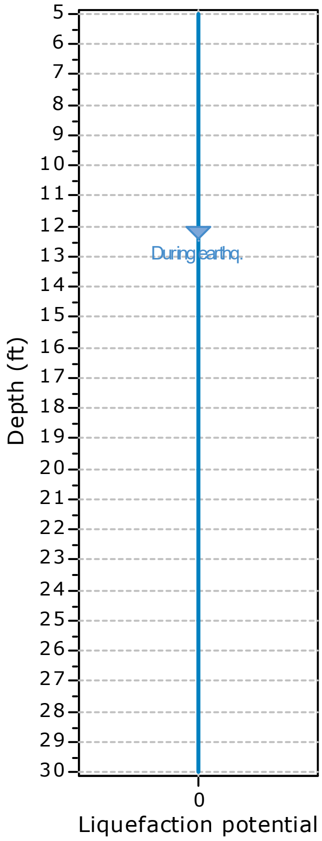
CSR - CRR Plot



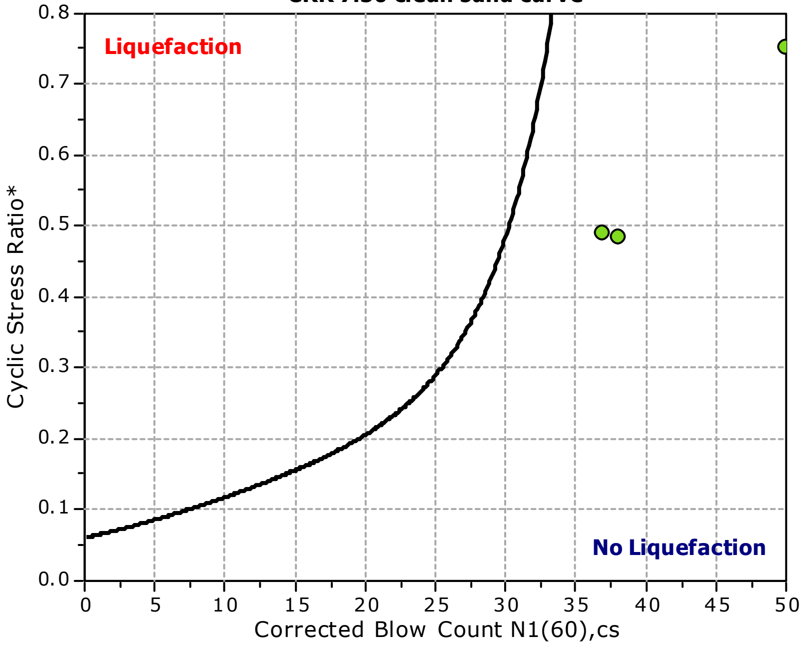
FS Plot



LPI



CRR 7.50 clean sand curve



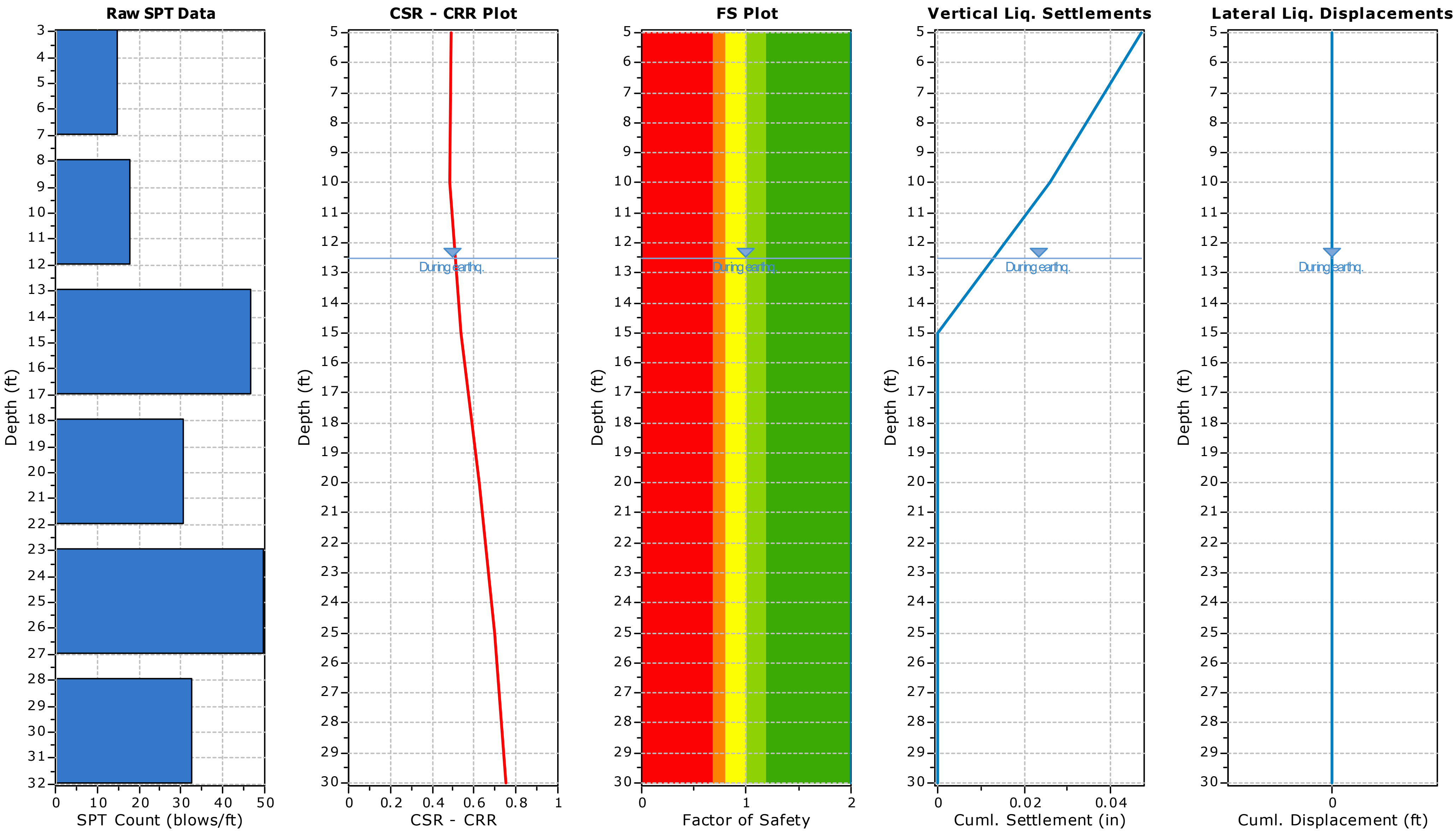
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
5.00	15	55.30	117.00	5.00	No
10.00	18	55.30	117.00	5.00	No
15.00	47	5.00	124.00	5.00	Yes
20.00	31	28.50	124.00	5.00	Yes
25.00	50	51.00	126.00	5.00	No
30.00	33	10.00	126.00	5.00	Yes

Abbreviations

Depth: Depth at which test was performed (ft)
SPT Field Value: Number of blows per foot
Fines Content: Fines content at test depth (%)
Unit Weight: Unit weight at test depth (pcf)
Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
5.00	15	117.00	0.29	0.00	0.29	0.30	1.48	1.25	1.15	0.80	1.20	31	55.30	5.61	37	4.000
10.00	18	117.00	0.59	0.00	0.59	0.31	1.20	1.25	1.15	0.85	1.20	32	55.30	5.61	38	4.000
15.00	47	124.00	0.90	0.00	0.90	0.26	1.05	1.25	1.15	0.95	1.20	80	5.00	0.00	80	4.000
20.00	31	124.00	1.21	0.00	1.21	0.26	0.97	1.25	1.15	0.95	1.20	49	28.50	5.30	54	4.000
25.00	50	126.00	1.52	0.00	1.52	0.26	0.91	1.25	1.15	0.95	1.20	74	51.00	5.61	80	4.000
30.00	33	126.00	1.84	0.00	1.84	0.26	0.87	1.25	1.15	1.00	1.20	49	10.00	1.15	50	4.000

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
m: Stress exponent normalization factor
C_N: Overburden corretion factor
C_E: Energy correction factor
C_B: Borehole diameter correction factor
C_R: Rod length correction factor
C_S: Liner correction factor
$N_{1(60)}$: Corrected N_{SPT} to a 60% energy ratio
$\Delta(N_1)_{60}$ Equivalent clean sand adjustment
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cydic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
5.00	117.00	0.29	0.00	0.29	1.00	1.00	0.382	2.20	37	0.92	0.414	1.10	0.636	2.000	●
10.00	117.00	0.59	0.00	0.59	0.98	1.00	0.378	2.20	38	0.92	0.409	1.10	0.629	2.000	●
15.00	124.00	0.90	0.08	0.82	0.97	1.00	0.408	2.20	80	0.92	0.442	1.08	0.694	2.000	●
20.00	124.00	1.21	0.23	0.97	0.96	1.00	0.455	2.20	54	0.92	0.493	1.03	0.812	2.000	●
25.00	126.00	1.52	0.39	1.13	0.94	1.00	0.484	2.20	80	0.92	0.525	0.98	0.905	2.000	●
30.00	126.00	1.84	0.55	1.29	0.92	1.00	0.503	2.20	50	0.92	0.545	0.94	0.978	2.000	●

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS

Abbreviations

$\sigma_{v,eq}$: Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$: Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$: Effective overburden pressure, during earthquake (tsf)
r_d: Nonlinear shear mass factor
α: Improvement factor due to stone columns
CSR : Cyclic Stress Ratio
MSF : Magnitude Scaling Factor
$CSR_{eq,M=7.5}$: CSR adjusted for M=7.5
K_{sigma}: Effective overburden stress factor
CSR^*: CSR fully adjusted (user FS applied)***
FS: Calculated factor of safety against soil liquefaction

*** User FS: 1.30

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
5.00	2.000	0.00	9.24	5.00	0.00
10.00	2.000	0.00	8.48	5.00	0.00
15.00	2.000	0.00	7.71	5.00	0.00
20.00	2.000	0.00	6.95	5.00	0.00
25.00	2.000	0.00	6.19	5.00	0.00
30.00	2.000	0.00	5.43	5.00	0.00

Overall potential I_L : 0.00

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
5.00	31	0.11	0.20	0.66	0.14	13381.49	0.00	0.00	17.10	0.02	5.00	0.021
10.00	32	0.22	0.39	0.94	0.15	8828.49	0.00	0.00	17.10	0.02	5.00	0.026

Cumulative settlemetns: 0.047

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
15.00	80	0.00	-4.20	2.000	0.00	0.00	5.00	0.000	0.00
20.00	54	0.00	-1.92	2.000	0.00	0.00	5.00	0.000	0.00
25.00	80	0.00	0.00	2.000	0.00	0.00	5.00	0.000	0.00
30.00	50	0.04	-1.59	2.000	0.00	0.00	5.00	0.000	0.00
							Cumulative settlements:	**0.000**	**0.00**

Abbreviations

γ_{lim}: Limiting shear strain (%)
F_α/N: Maximun shear strain factor
γ_{max}: Maximum shear strain (%)
e_v:: Post liquefaction volumetric strain (%)
S_{v-1D}: Estimated vertical settlement (in)
LDI: Estimated lateral displacement (ft)

2883 East Spring Street, Suite 300
TWINING
Long Beach, CA 90806

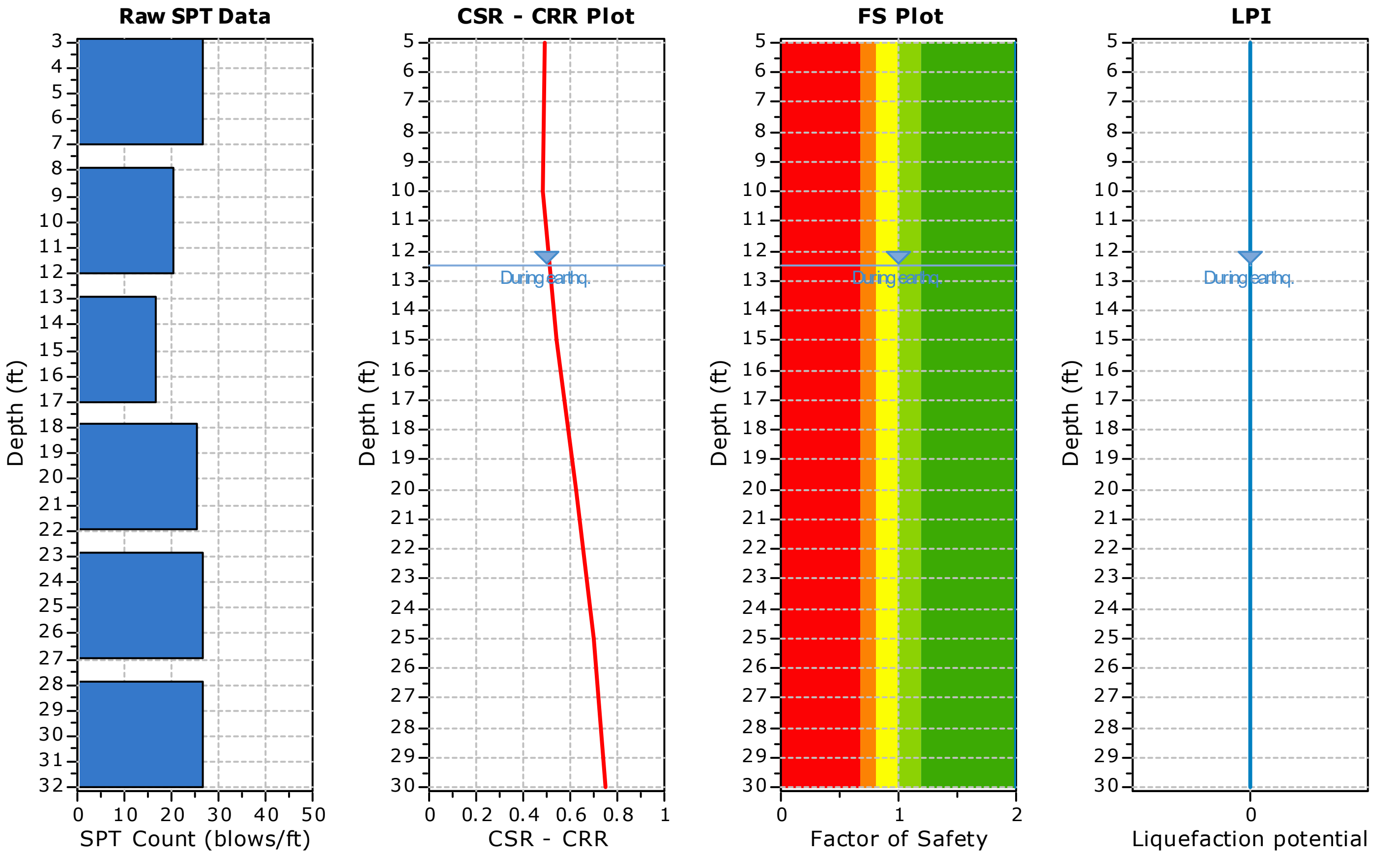
SPT BASED LIQUEFACTION ANALYSIS REPORT

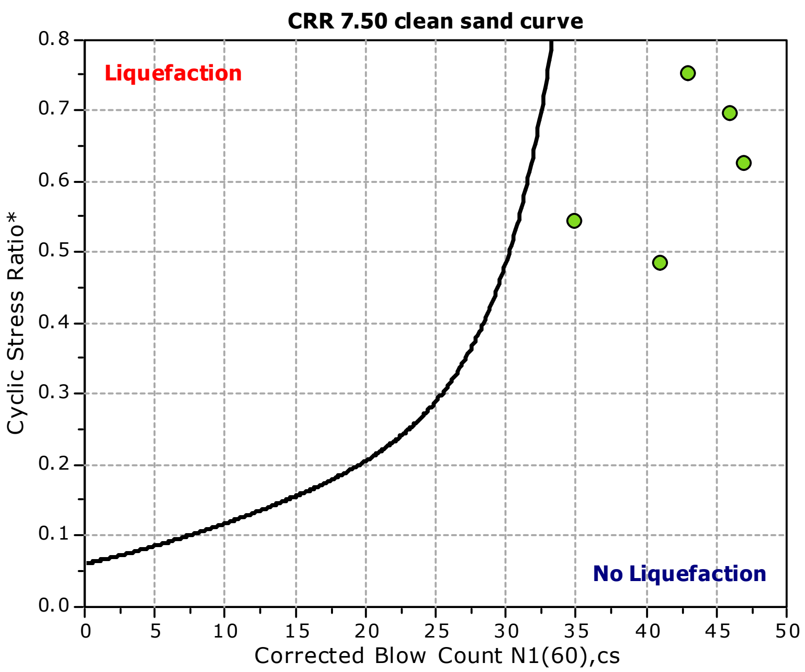
Project title : Proposed Parking Structure at Kaiser Riverside Medical Center **SPT Name: PS-3**

Location : 10800 Magnolia Avenue, Riverside, California

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	57.50 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	12.50 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	7.70
Borehole diameter:	200mm	Peak ground acceleration:	0.59 g
Rod length:	5.00 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.25		





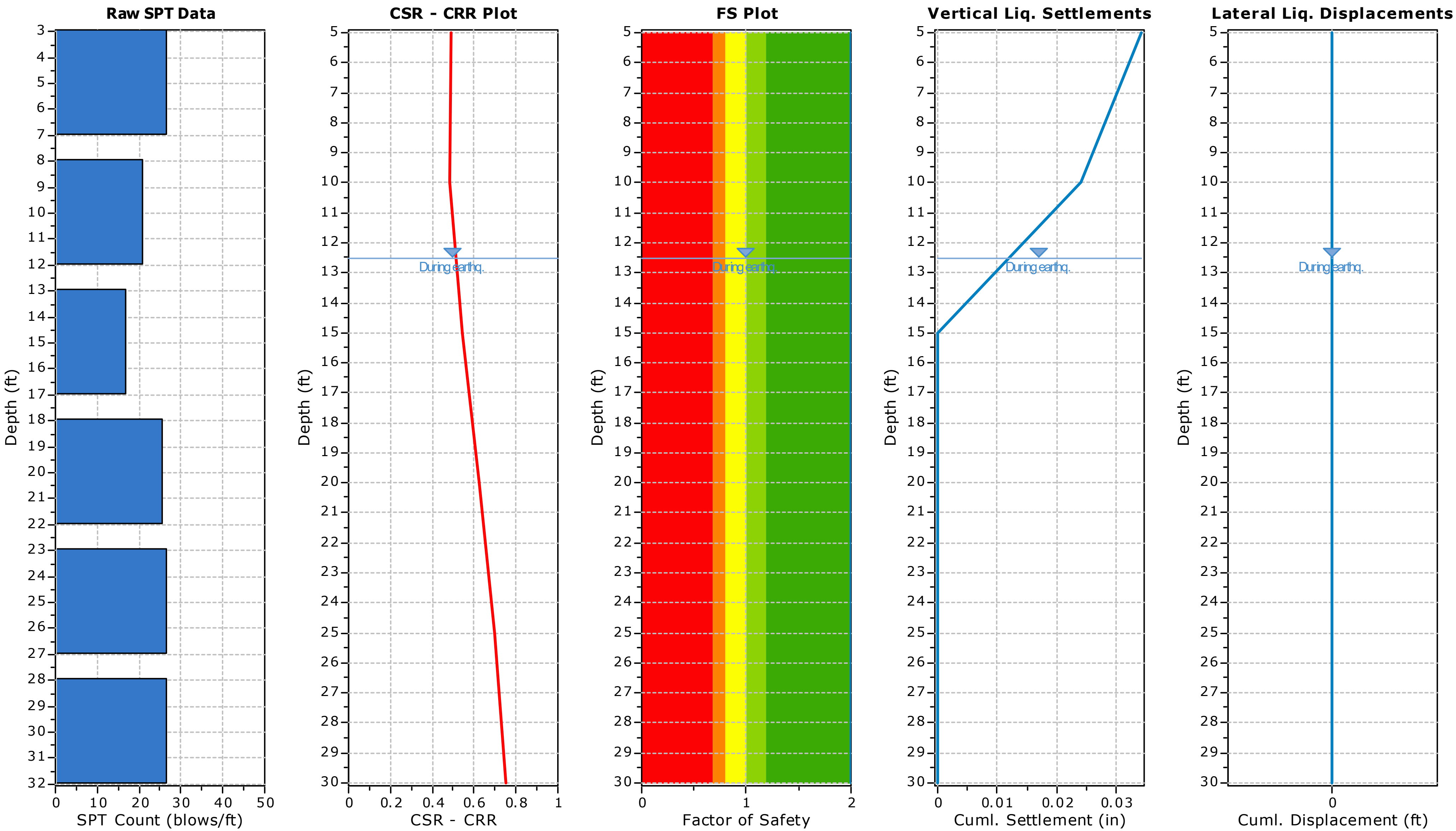
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
5.00	27	71.80	135.00	5.00	No
10.00	21	71.80	135.00	5.00	No
15.00	17	50.50	105.00	5.00	Yes
20.00	26	50.50	105.00	5.00	Yes
25.00	27	85.00	135.00	5.00	No
30.00	27	15.00	135.00	5.00	Yes

Abbreviations

Depth:	Depth at which test was performed (ft)
SPT Field Value:	Number of blows per foot
Fines Content:	Fines content at test depth (%)
Unit Weight:	Unit weight at test depth (pcf)
Infl. Thickness:	Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy:	User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
5.00	27	135.00	0.34	0.00	0.34	0.26	1.35	1.25	1.15	0.80	1.20	50	71.80	5.57	56	4.000
10.00	21	135.00	0.68	0.00	0.68	0.29	1.14	1.25	1.15	0.85	1.20	35	71.80	5.57	41	4.000
15.00	17	105.00	0.94	0.00	0.94	0.33	1.04	1.25	1.15	0.95	1.20	29	50.50	5.61	35	4.000
20.00	26	105.00	1.20	0.00	1.20	0.26	0.97	1.25	1.15	0.95	1.20	41	50.50	5.61	47	4.000
25.00	27	135.00	1.54	0.00	1.54	0.27	0.90	1.25	1.15	0.95	1.20	40	85.00	5.53	46	4.000
30.00	27	135.00	1.88	0.00	1.88	0.28	0.85	1.25	1.15	1.00	1.20	40	15.00	3.26	43	4.000

Abbreviations

σ_v:	Total stress during SPT test (tsf)
u_o:	Water pore pressure during SPT test (tsf)
σ'_{vo}:	Effective overburden pressure during SPT test (tsf)
m:	Stress exponent normalization factor
C_N:	Overburden corretion factor
C_E:	Energy correction factor
C_B:	Borehole diameter correction factor
C_R:	Rod length correction factor
C_S:	Liner correction factor
$N_{1(60)}$:	Corrected N_{SPT} to a 60% energy ratio
$\Delta(N_1)_{60}$	Equivalent clean sand adjustment
$N_{1(60)cs}$:	Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$:	Cydic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
5.00	135.00	0.34	0.00	0.34	1.00	1.00	0.382	2.20	56	0.92	0.414	1.10	0.636	2.000
10.00	135.00	0.68	0.00	0.68	0.98	1.00	0.378	2.20	41	0.92	0.409	1.10	0.629	2.000
15.00	105.00	0.94	0.08	0.86	0.97	1.00	0.406	2.20	35	0.92	0.440	1.05	0.706	2.000
20.00	105.00	1.20	0.23	0.97	0.96	1.00	0.455	2.20	47	0.92	0.493	1.03	0.812	2.000
25.00	135.00	1.54	0.39	1.15	0.94	1.00	0.482	2.20	46	0.92	0.523	0.98	0.905	2.000
30.00	135.00	1.88	0.55	1.33	0.92	1.00	0.498	2.20	43	0.92	0.540	0.93	0.978	2.000

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR*	FS

Abbreviations

$\sigma_{v,eq}$:	Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$:	Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$:	Effective overburden pressure, during earthquake (tsf)
r_d:	Nonlinear shear mass factor
α:	Improvement factor due to stone columns
CSR :	Cyclic Stress Ratio
MSF :	Magnitude Scaling Factor
$CSR_{eq,M=7.5}$:	CSR adjusted for M=7.5
K_{sigma}:	Effective overburden stress factor
CSR*:	CSR fully adjusted (user FS applied)***
FS:	Calculated factor of safety against soil liquefaction

*** User FS: 1.30

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
5.00	2.000	0.00	9.24	5.00	0.00
10.00	2.000	0.00	8.48	5.00	0.00
15.00	2.000	0.00	7.71	5.00	0.00
20.00	2.000	0.00	6.95	5.00	0.00
25.00	2.000	0.00	6.19	5.00	0.00
30.00	2.000	0.00	5.43	5.00	0.00

Overall potential I_L : 0.00

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
5.00	50	0.13	0.23	0.81	0.14	12280.49	0.00	0.00	17.10	0.01	5.00	0.010
10.00	35	0.25	0.45	1.04	0.15	8102.10	0.00	0.00	17.10	0.02	5.00	0.024

Cumulative settlemetns: 0.034

Abbreviations

τ_{av}:	Average cyclic shear stress
p:	Average stress
G_{max}:	Maximum shear modulus (tsf)
α, b:	Shear strain formula variables
γ:	Average shear strain
ε_{15}:	Volumetric strain after 15 cycles
N_c:	Number of cycles
ε_{Nc}:	Volumetric strain for number of cycles N_c (%)
Δh:	Thickness of soil layer (in)
ΔS:	Settlement of soil layer (in)

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
15.00	35	2.20	-0.44	2.000	0.00	0.00	5.00	0.000	0.00
20.00	47	0.13	-1.35	2.000	0.00	0.00	5.00	0.000	0.00
25.00	46	0.00	0.00	2.000	0.00	0.00	5.00	0.000	0.00
30.00	43	0.44	-1.03	2.000	0.00	0.00	5.00	0.000	0.00

Cumulative settlements: 0.000 0.00

Abbreviations

γ_{lim}: Limiting shear strain (%)
F_α/N: Maximun shear strain factor
γ_{max}: Maximum shear strain (%)
e_v:: Post liquefaction volumetric strain (%)
S_{v-1D}: Estimated vertical settlement (in)
LDI: Estimated lateral displacement (ft)

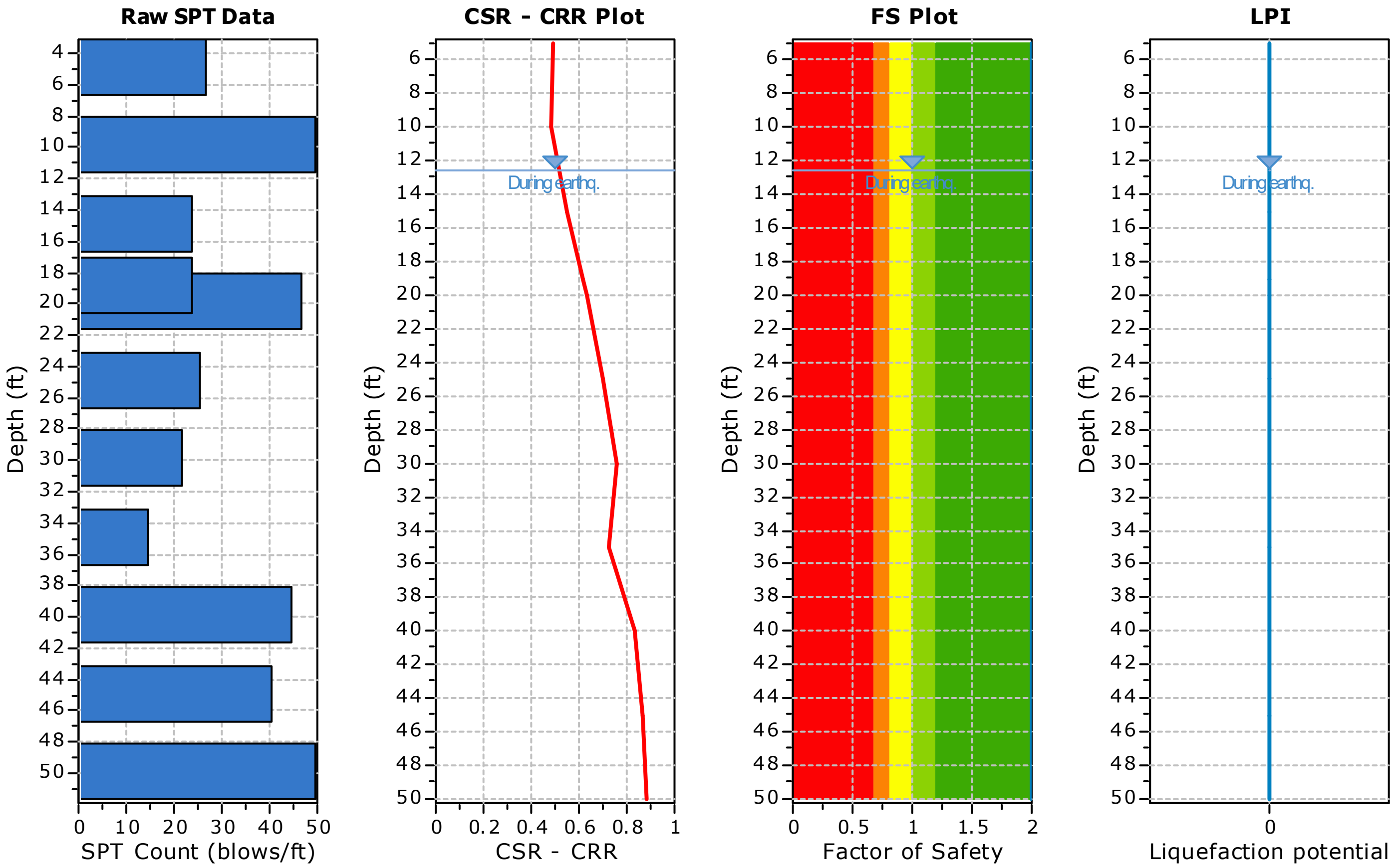
SPT BASED LIQUEFACTION ANALYSIS REPORT

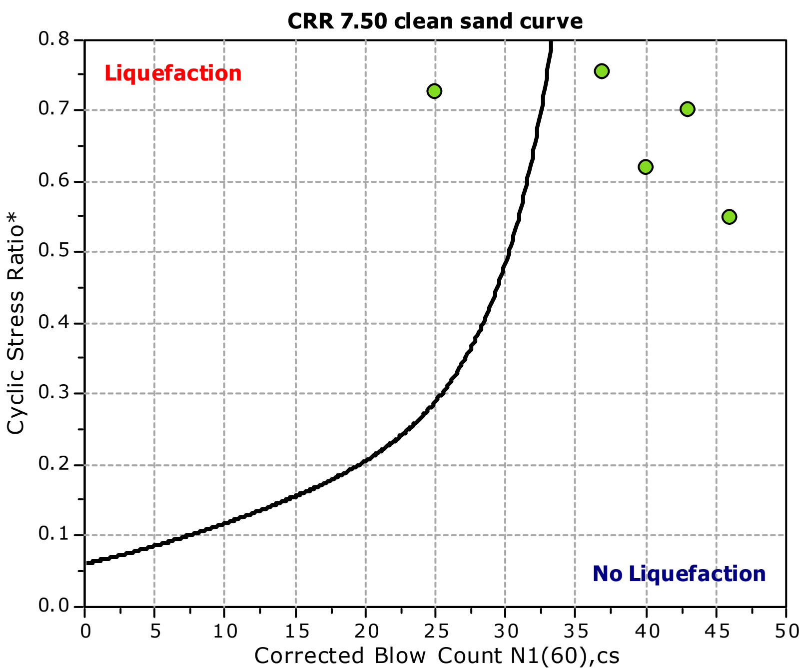
Project title : Proposed Parking Structure at Kaiser Riverside Medical Center **SPT Name: PS-4**

Location : 10800 Magnolia Avenue, Riverside, California

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	57.50 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	12.50 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	7.70
Borehole diameter:	200mm	Peak ground acceleration:	0.59 g
Rod length:	5.00 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.25		





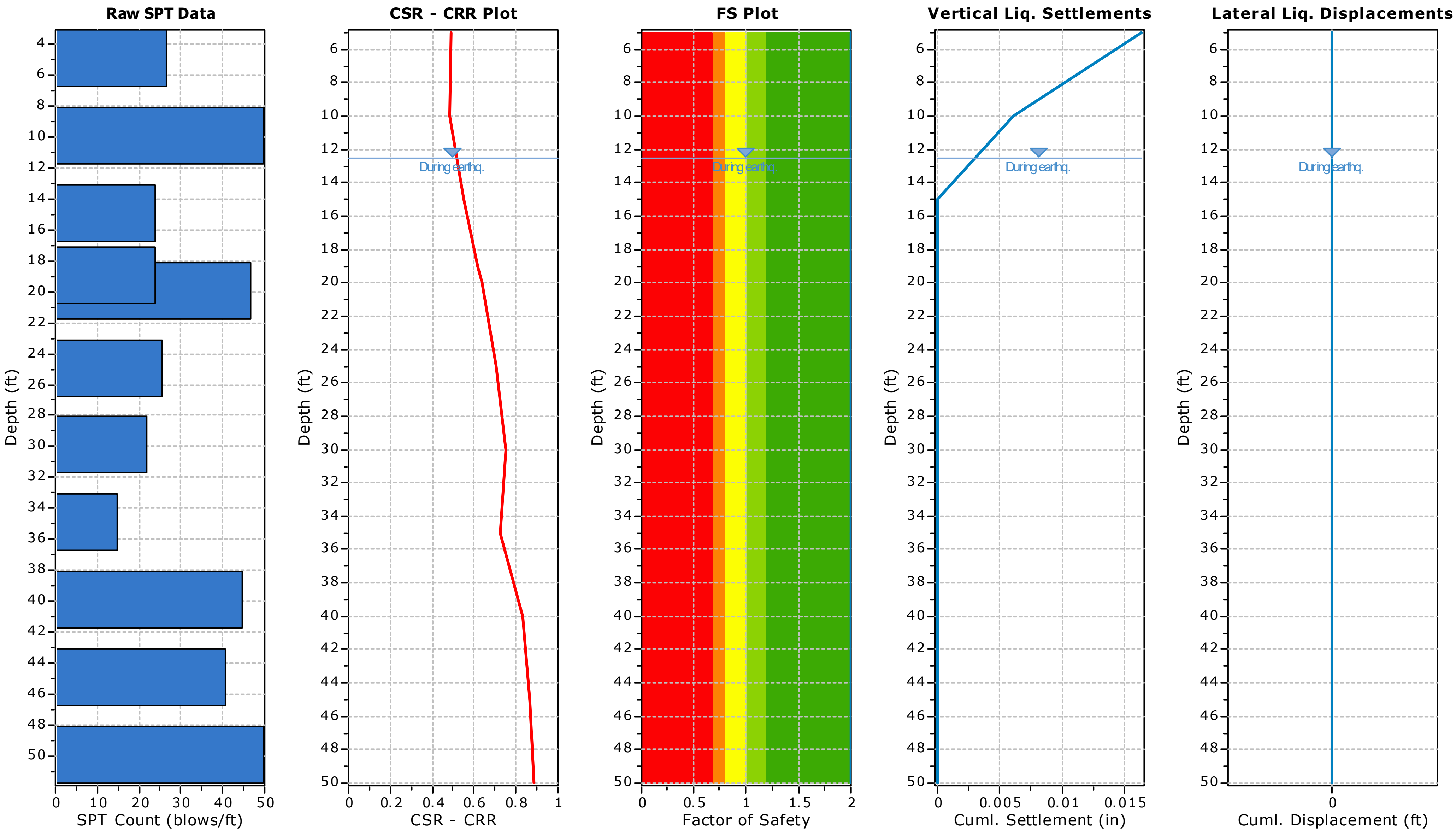
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
5.00	27	55.00	137.00	5.00	No
10.00	50	55.00	137.00	5.00	No
15.00	24	55.00	137.00	4.00	Yes
19.00	24	15.00	137.00	1.00	Yes
20.00	47	51.00	124.00	5.00	Yes
25.00	26	51.00	124.00	5.00	No
30.00	22	51.00	128.00	5.00	Yes
35.00	15	51.00	128.00	5.00	No
40.00	45	5.20	111.00	5.00	Yes
45.00	41	5.20	111.00	5.00	Yes
50.00	50	5.20	115.00	5.00	Yes

Abbreviations

Depth: Depth at which test was performed (ft)
SPT Field Value: Number of blows per foot
Fines Content: Fines content at test depth (%)
Unit Weight: Unit weight at test depth (pcf)
Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
5.00	27	137.00	0.34	0.00	0.34	0.26	1.35	1.25	1.15	0.80	1.20	50	55.00	5.61	56	4.000
10.00	50	137.00	0.69	0.00	0.69	0.26	1.12	1.25	1.15	0.85	1.20	82	55.00	5.61	88	4.000
15.00	24	137.00	1.03	0.00	1.03	0.27	1.01	1.25	1.15	0.95	1.20	40	55.00	5.61	46	4.000
19.00	24	137.00	1.30	0.00	1.30	0.30	0.94	1.25	1.15	0.95	1.20	37	15.00	3.26	40	4.000
20.00	47	124.00	1.36	0.00	1.36	0.26	0.94	1.25	1.15	0.95	1.20	72	51.00	5.61	78	4.000
25.00	26	124.00	1.67	0.00	1.67	0.28	0.88	1.25	1.15	0.95	1.20	37	51.00	5.61	43	4.000
30.00	22	128.00	1.99	0.00	1.99	0.33	0.81	1.25	1.15	1.00	1.20	31	51.00	5.61	37	4.000
35.00	15	128.00	2.31	0.00	2.31	0.42	0.72	1.25	1.15	1.00	1.20	19	51.00	5.61	25	4.000
40.00	45	111.00	2.59	0.00	2.59	0.26	0.79	1.25	1.15	1.00	1.20	61	5.20	0.00	61	4.000
45.00	41	111.00	2.87	0.00	2.87	0.26	0.77	1.25	1.15	1.00	1.20	54	5.20	0.00	54	4.000
50.00	50	115.00	3.16	0.00	3.16	0.26	0.75	1.25	1.15	1.00	1.20	65	5.20	0.00	65	4.000

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
m: Stress exponent normalization factor
C_N: Overburden corretion factor
C_E: Energy correction factor
C_B: Borehole diameter correction factor
C_R: Rod length correction factor
C_S: Liner correction factor
$N_{1(60)}$: Corrected N_{SPT} to a 60% energy ratio
$\Delta(N_1)_{60}$ Equivalent clean sand adjustment
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cydic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
5.00	137.00	0.34	0.00	0.34	1.00	1.00	0.382	2.20	56	0.92	0.414	1.10	0.636	2.000
10.00	137.00	0.69	0.00	0.69	0.98	1.00	0.378	2.20	88	0.92	0.409	1.10	0.629	2.000
15.00	137.00	1.03	0.08	0.95	0.97	1.00	0.403	2.20	46	0.92	0.437	1.03	0.715	2.000
19.00	137.00	1.30	0.20	1.10	0.96	1.00	0.435	2.20	40	0.92	0.472	0.99	0.807	2.000
20.00	124.00	1.36	0.23	1.13	0.96	1.00	0.442	2.20	78	0.92	0.480	0.98	0.826	2.000
25.00	124.00	1.67	0.39	1.28	0.94	1.00	0.469	2.20	43	0.92	0.509	0.94	0.912	2.000
30.00	128.00	1.99	0.55	1.45	0.92	1.00	0.486	2.20	37	0.92	0.527	0.91	0.981	2.000
35.00	128.00	2.31	0.70	1.61	0.90	1.00	0.496	1.72	25	0.95	0.520	0.93	0.944	2.000
40.00	111.00	2.59	0.86	1.73	0.88	1.00	0.505	2.20	61	0.92	0.548	0.85	1.084	2.000
45.00	111.00	2.87	1.01	1.85	0.86	1.00	0.511	2.20	54	0.92	0.554	0.83	1.122	2.000
50.00	115.00	3.16	1.17	1.99	0.84	1.00	0.512	2.20	65	0.92	0.555	0.81	1.153	2.000

Abbreviations

$\sigma_{v,eq}$:	Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$:	Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$:	Effective overburden pressure, during earthquake (tsf)
r_d:	Nonlinear shear mass factor
α:	Improvement factor due to stone columns
CSR :	Cyclic Stress Ratio
MSF :	Magnitude Scaling Factor
$CSR_{eq,M=7.5}$:	CSR adjusted for M=7.5
K_{sigma}:	Effective overburden stress factor
CSR^*:	CSR fully adjusted (user FS applied)***
FS:	Calculated factor of safety against soil liquefaction

*** User FS: 1.30

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
5.00	2.000	0.00	9.24	5.00	0.00
10.00	2.000	0.00	8.48	5.00	0.00
15.00	2.000	0.00	7.71	5.00	0.00
19.00	2.000	0.00	7.10	4.00	0.00
20.00	2.000	0.00	6.95	1.00	0.00
25.00	2.000	0.00	6.19	5.00	0.00
30.00	2.000	0.00	5.43	5.00	0.00
35.00	2.000	0.00	4.67	5.00	0.00
40.00	2.000	0.00	3.90	5.00	0.00
45.00	2.000	0.00	3.14	5.00	0.00
50.00	2.000	0.00	2.38	5.00	0.00

Overall potential I_L : 0.00

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
5.00	50	0.13	0.23	0.82	0.14	12172.61	0.00	0.00	17.10	0.01	5.00	0.010

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
10.00	82	0.26	0.46	1.35	0.15	8030.93	0.00	0.00	17.10	0.01	5.00	0.006

Cumulative settlemetns: 0.016

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
15.00	46	0.19	-1.27	2.000	0.00	0.00	4.00	0.000	0.00
19.00	40	0.87	-0.80	2.000	0.00	0.00	1.00	0.000	0.00
20.00	78	0.00	-4.01	2.000	0.00	0.00	5.00	0.000	0.00
25.00	43	0.00	0.00	2.000	0.00	0.00	5.00	0.000	0.00
30.00	37	1.56	-0.58	2.000	0.00	0.00	5.00	0.000	0.00
35.00	25	0.00	0.00	2.000	0.00	0.00	5.00	0.000	0.00
40.00	61	0.00	-2.51	2.000	0.00	0.00	5.00	0.000	0.00
45.00	54	0.00	-1.92	2.000	0.00	0.00	5.00	0.000	0.00
50.00	65	0.00	-2.86	2.000	0.00	0.00	5.00	0.000	0.00

Cumulative settlements: 0.000 0.00

Abbreviations

γ_{lim}: Limiting shear strain (%)
F_α/N: Maximun shear strain factor
γ_{max}: Maximum shear strain (%)
e_v:: Post liquefaction volumetric strain (%)
S_{v-1D}: Estimated vertical settlement (in)
LDI: Estimated lateral displacement (ft)

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TWINING

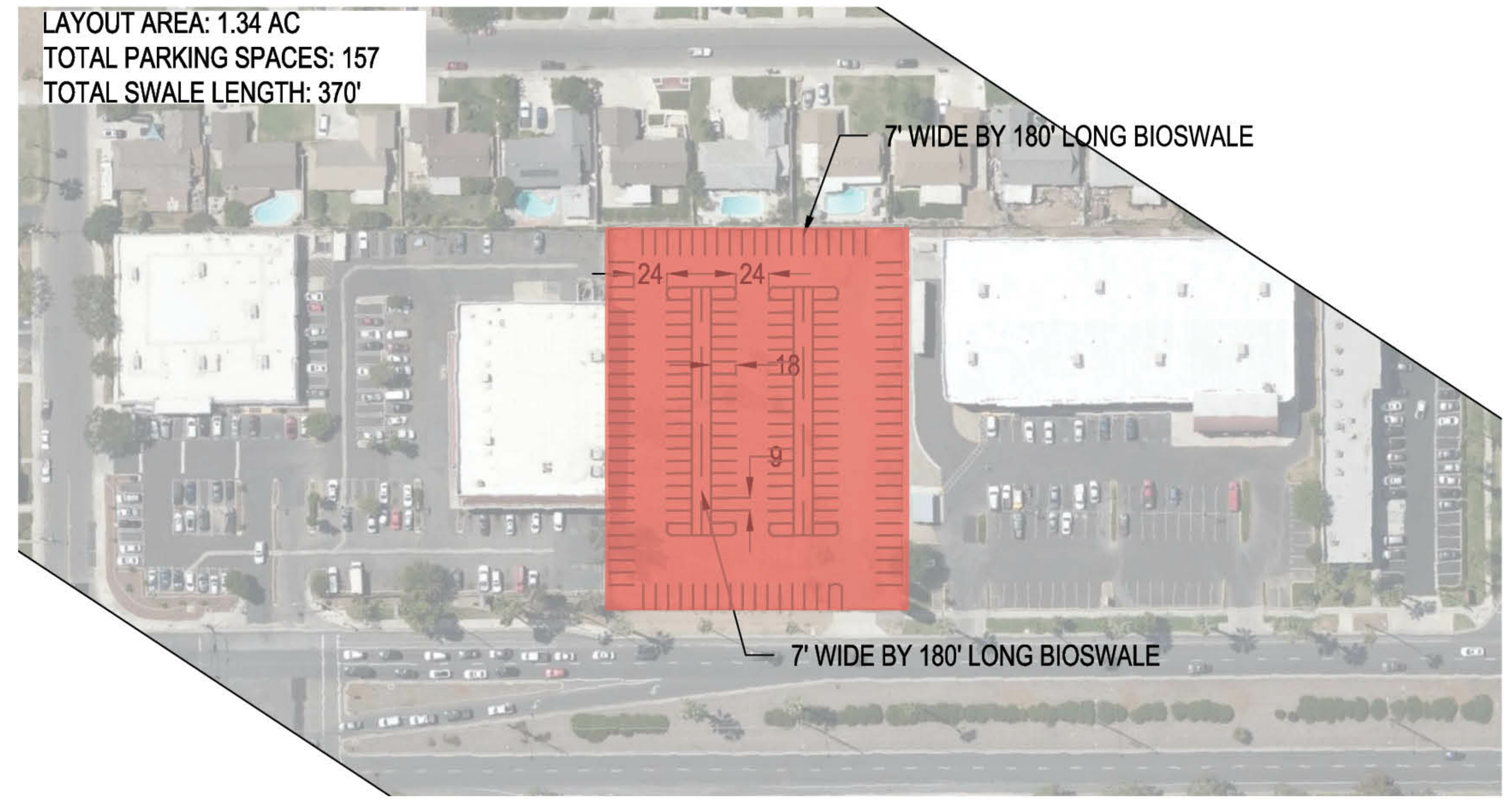
2883 East Spring Street
Suite 300
Long Beach CA 90806

Tel 562.426.3355
Fax 562.426.6424

APPENDIX D
SELECT PROJECT PLANS

PARKING DESIGNATION LEGEND

- STAFF & GENERAL CONTRACTOR PARKING
- GENERAL CONTRACTOR PARKING



PEP BOYS LOT
STALL COUNT: 157

FILLMORE LOT
STALL COUNT: 657

Phase 1 – Offsite Make Ready

Improvement of off-campus lots at 11510 Magnolia Ave (Fillmore) and 10861 Magnolia Ave (Pep Boys). The Fillmore parking lot will accomodate displacement of staff parking during the construction of the new parking structure and General Contractor parking during hospital construction. The Pep Boys lot will be improved for use by the General Contractor .

SITE PHASING - 1

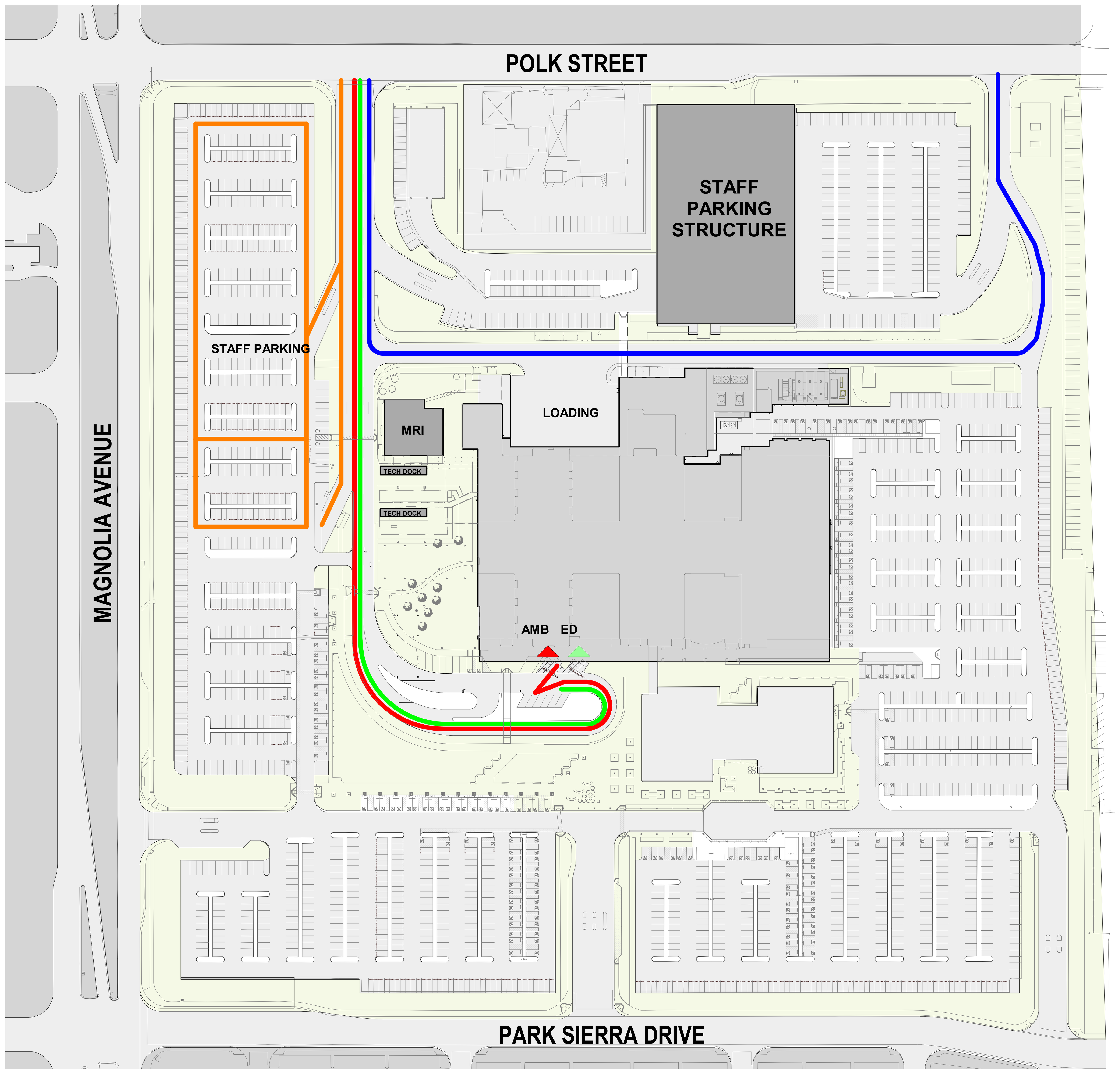
JULY 19, 2021

CO ARCHITECTS

KAISER PERMANENTE

RIVERSIDE MEDICAL CENTER

Sheet: **A 1.20**	CITY OF RIVERSIDE
Site Development Plan Number:	
OWNER: Kaiser Foundation Hospitals PHONE: 626.405.6333	
ADDRESS: 393 E. Walnut Street Pasadena, CA 91188	
ARCHITECT, ENGINEER, DESIGNER: CO Architects, Michael Baker International, Ridge Landscape Architects, Glumac	PHONE: 323.525.0500 (Architect)
ADDRESS: 5055 Wilshire Blvd. 9th Floor, Los Angeles CA 90036 (Architect)	LOCATION: 10800 Magnolia Ave. Riverside, CA 92505
TYPE OF DEVELOPMENT: XXXXX	
ZONE: XXXXX	ACCESSOR'S PARCEL NUMBER: 138-470-010



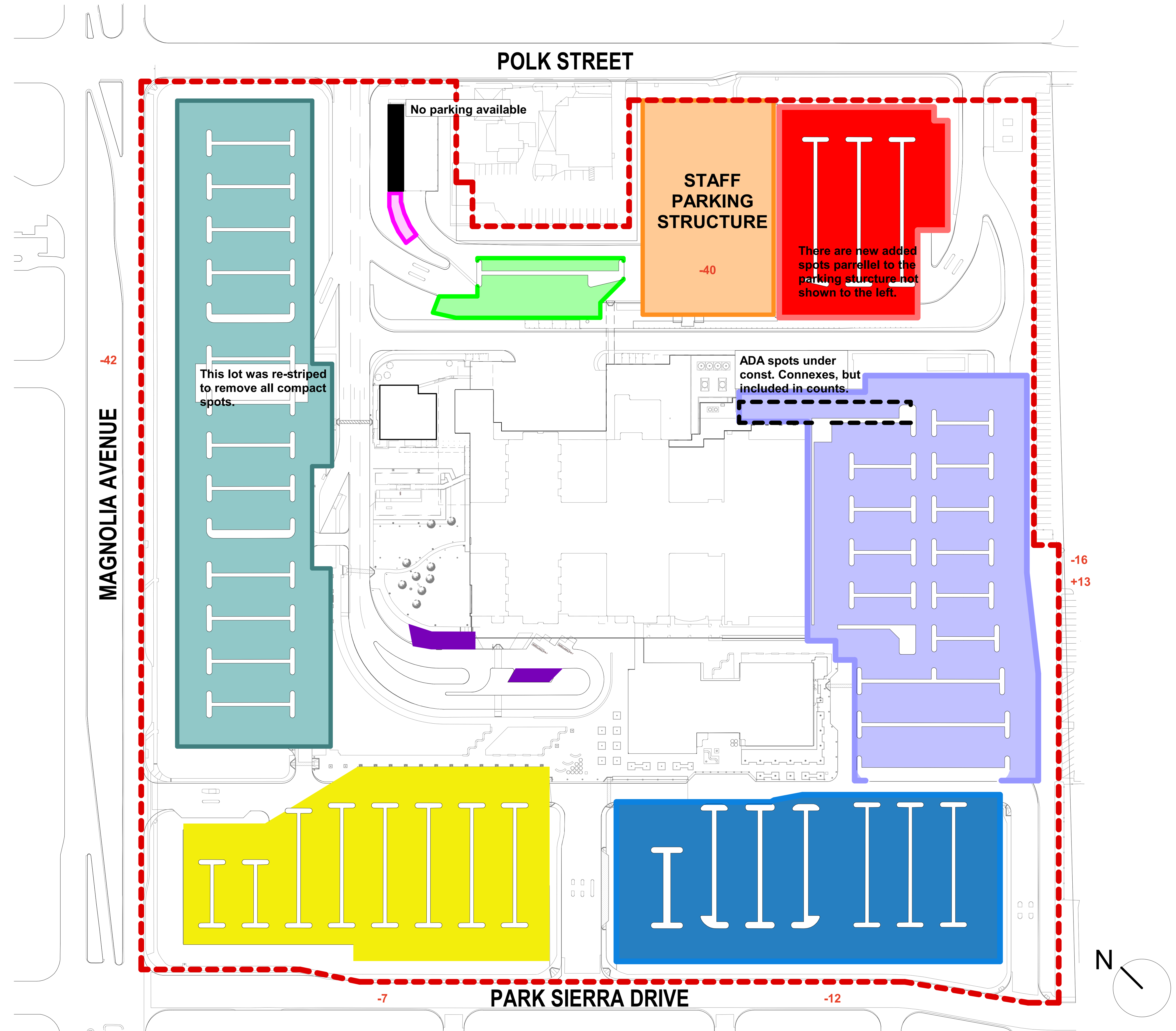
1 EXISTING SITE PLAN
1" = 100'-0"

CIRCULATION LEGEND
- AMBULANCE ROUTE
- MEMBER ROUTE
- STAFF ROUTE
- SERVICE ROUTE

EXISTING PARKING MATRIX

CODE REQ'D:	2,543
PROVIDED:	2,842

N



2 PH 2.0
1" = 100'-0"

PHASE 2 PARKING MATRIX

CODE REQ'D:	2,543
PROVIDED:	2,745

STANDARD: 454	STANDARD: 7	STANDARD: 65	STANDARD: 643	STANDARD: 190
ACCESSIBLE: 26	ACCESSIBLE: 0	ACCESSIBLE: 0	ACCESSIBLE: 15	ACCESSIBLE: 0
ACCESSIBLE VAN: 0	ACCESSIBLE VAN: 0	ACCESSIBLE VAN: 0	ACCESSIBLE VAN: 4	ACCESSIBLE VAN: 0
LOT TOTAL: 480	LOT TOTAL: 7	LOT TOTAL: 65	LOT TOTAL: 662	LOT TOTAL: 190

STANDARD: 359	STANDARD: 288	STANDARD: 249	STANDARD: 11
ACCESSIBLE: 36	ACCESSIBLE: 45	ACCESSIBLE: 42	ACCESSIBLE: 0
ACCESSIBLE VAN: 7	ACCESSIBLE VAN: 1	ACCESSIBLE VAN: 13	ACCESSIBLE VAN: 2
LOT TOTAL: 402	LOT TOTAL: 334	LOT TOTAL: 304	LOT TOTAL: 13

SHEET TOTAL:

STANDARD:	2268
COMPACT:	0
UNDER 8'-6":	0
ACCESSIBLE:	169
ACCESSIBLE VAN:	22
ON SITE TOTAL:	2459
OFFSITE STALLS:	286
TOTAL:	2745

Existing Site Plan

Phase 2: On-Site Parking Re-Stripe

Re-striping of onsite parking lots to convert all compact and sub-standard parking stalls to standard parking stalls. Convert parking lot south of Park Sierra's entry drive into accessible parking lot.

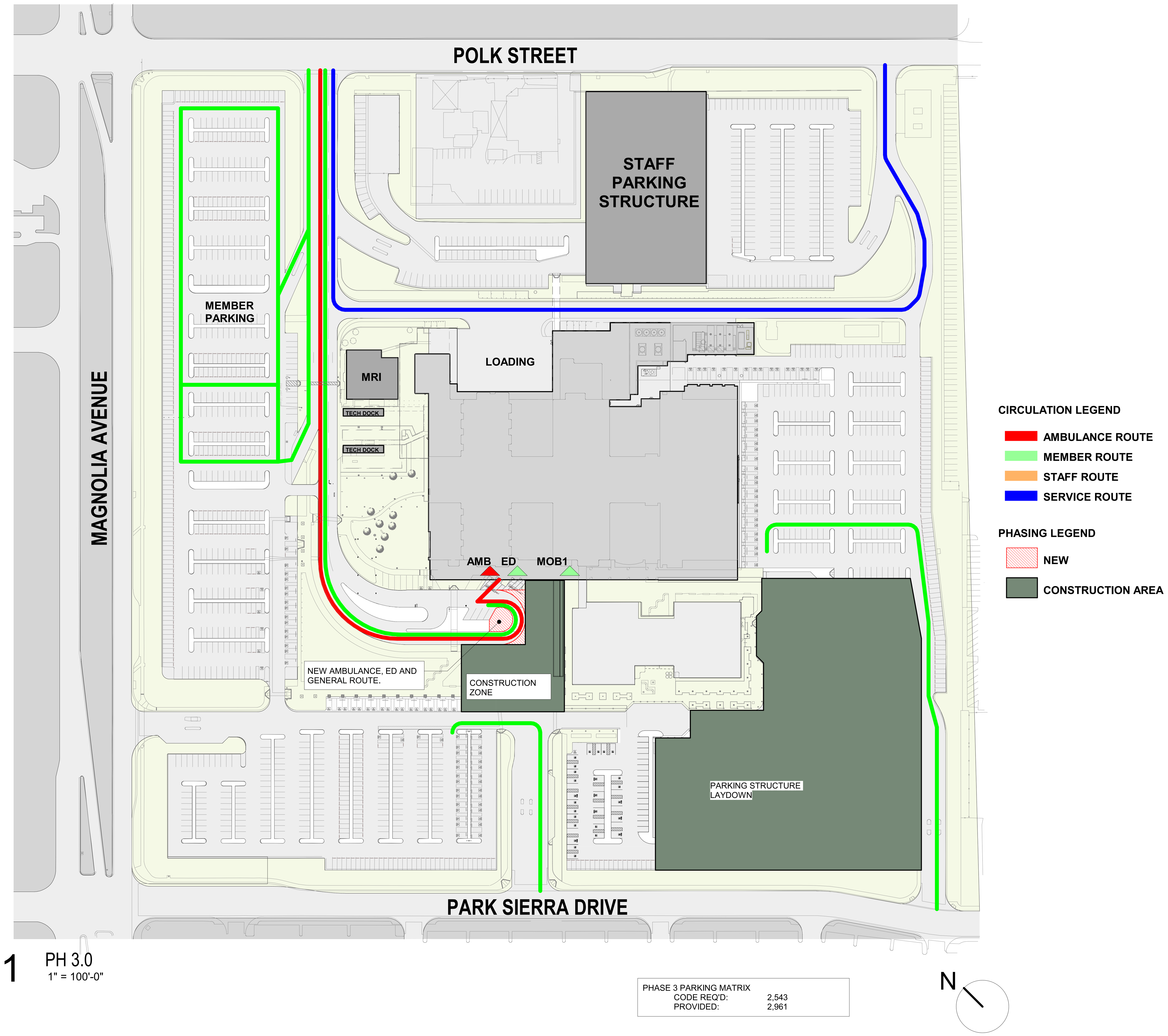
SITE PHASING - 2
1" = 100'-0"
JULY 19, 2021

CO ARCHITECTS

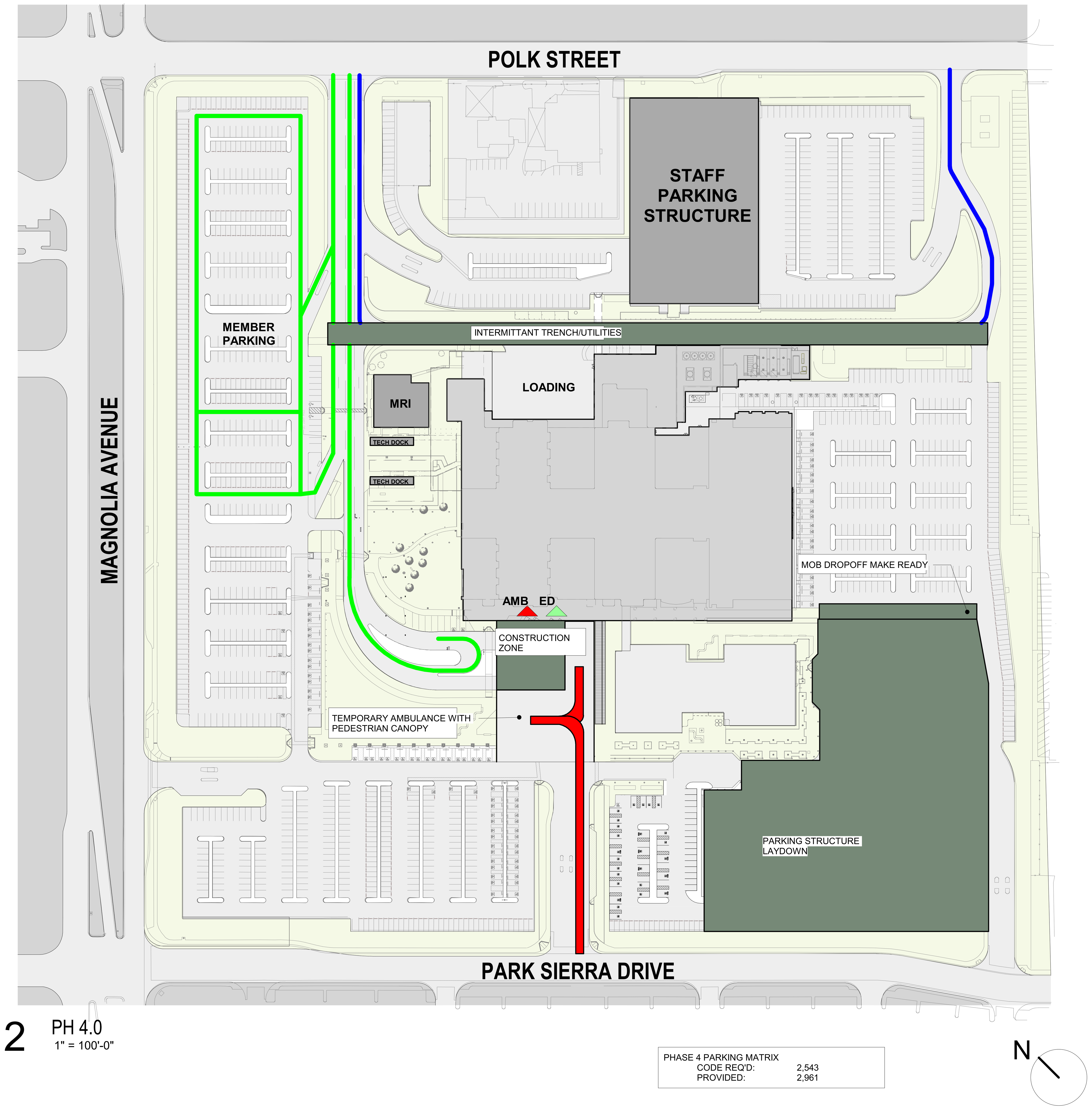
KAISER PERMANENTE

RIVERSIDE MEDICAL CENTER

Sheet: A 1.21
CITY OF RIVERSIDE
Site Development Plan Number:
OWNER: Kaiser Foundation Hospitals PHONE: 626.405.6333
ADDRESS: 393 E. Walnut Street Pasadena, CA 91188
ARCHITECT, ENGINEER, DESIGNER: CO Architects, Michael Baker International, Ridge Landscape Architects, Glumac
PHONE: 323.525.0500 (Architect)
ADDRESS: 5055 Wilshire Blvd. 9th Floor, Los Angeles CA 90036 (Architect)
LOCATION: 10800 Magnolia Ave. Riverside, CA 92505
TYPE OF DEVELOPMENT: XXXXX
ZONE: XXXXX
ACCESSOR'S PARCEL NUMBER: 138-470-010



1 PH 3.0
1" = 100'-0"



2 PH 4.0
1" = 100'-0"

Phase 3 - Shortened Ambulance Drive and Parking Structure Laydown

Shorten the ambulance and patient drop off loop, removing 2 parking stalls.

Phase 4 - Temporary Ambulance Drop-off and Upgrade CUP

Install new temporary ambulance drop-off area and canopy.

Upgrades to the Central Utility Plant (CUP) and utility connections from the CUP to the new Hospital Tower location.

SITE PHASING - 3,4

1" = 100'-0"

JULY 19, 2021

CO ARCHITECTS

KAISER PERMANENTE

RIVERSIDE MEDICAL CENTER

Sheet: **A 1.22**

CITY OF RIVERSIDE

Site Development Plan Number:

OWNER: Kaiser Foundation Hospitals PHONE: 626.405.6333

ADDRESS: 393 E. Walnut Street Pasadena, CA 91188

ARCHITECT, ENGINEER, DESIGNER: CO Architects, Michael Baker International, Ridge Landscape Architects, Glumac PHONE: 323.525.0500 (Architect)

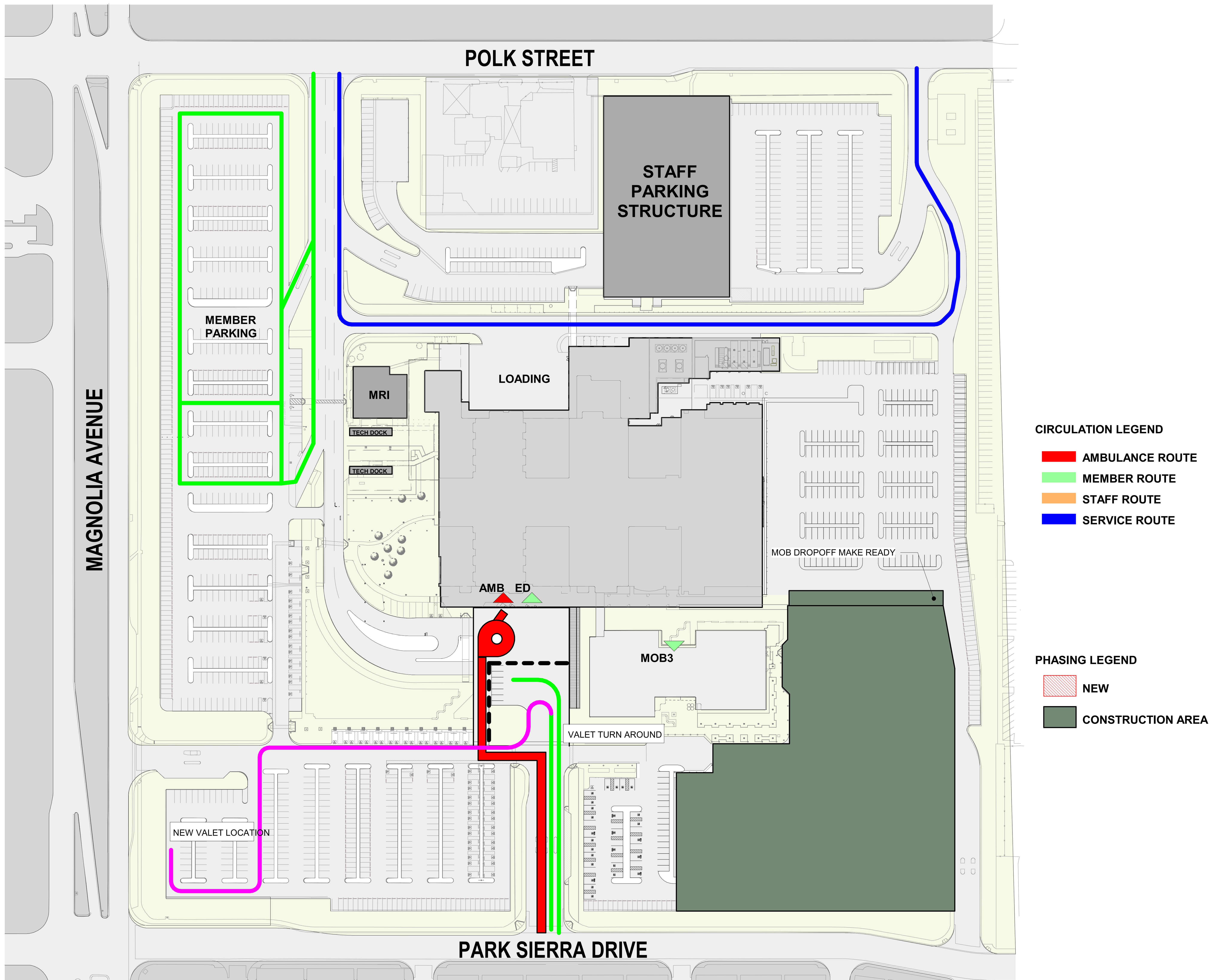
ADDRESS: 5055 Wilshire Blvd. 9th Floor, Los Angeles CA 90036 (Architect)

TYPE OF DEVELOPMENT: XXXXX

LOCATION: 10800 Magnolia Ave. Riverside, CA 92505

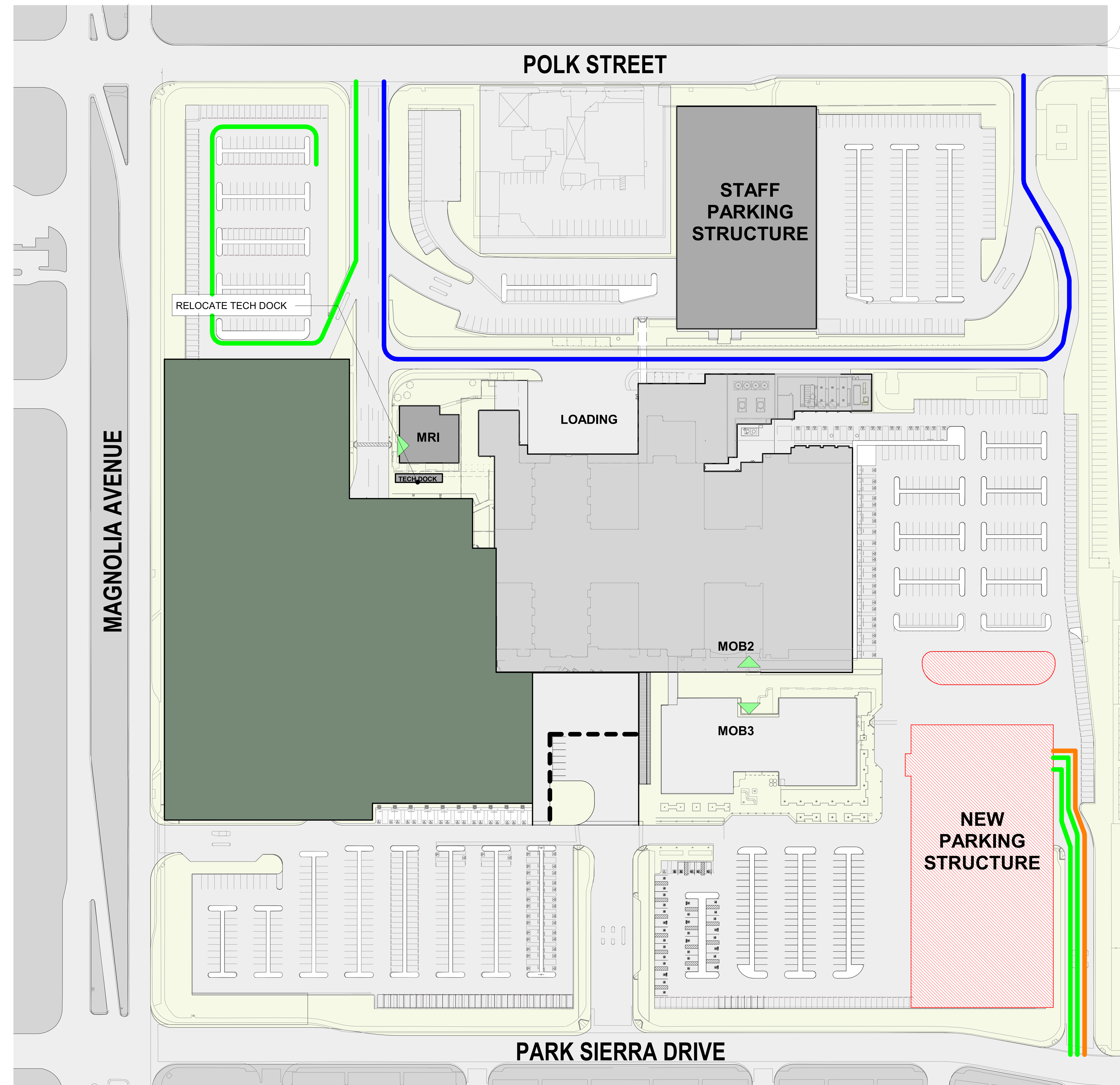
ZONE: XXXXX

ACCESSOR'S PARCEL NUMBER: 138-470-010



1 PH 5.0
1" = 100'-0"

PHASE 5 PARKING MATRIX
CODE REQ'D: 2,543
PROVIDED: 2,961



2 PH 6.0
1" = 100'-0"

PHASE 6 PARKING MATRIX
CODE REQ'D: 2,543
PROVIDED: 3,251

Phase 5 – Temporary Ambulance Drop off and Patient Drop off

New temporary Ambulance Drop off.
New Temporary Patient Drop off.

Phase 6 - Hospital New Tower (NT)

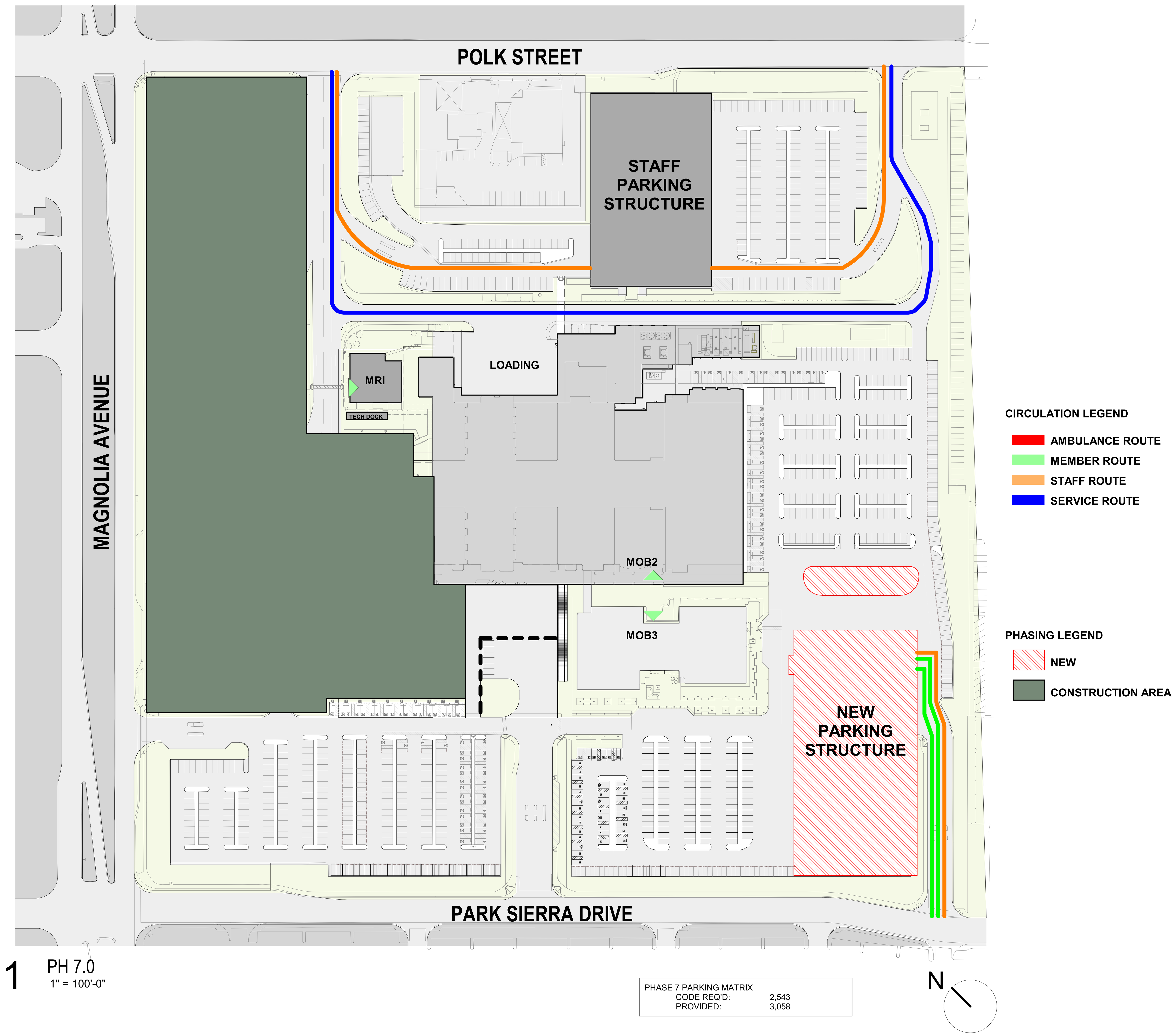
(NT), Hospital Tower construction, and correlating site work. The construction sequence and methods are as follows:

Demo and Grading

Sitework - Underground Utilities including relocating outside the building footprint, connections from the CUP to new Hospital Tower, and underground tanks.

Shoring and Mass Excavation

NT Underground – Cast-in-place reinforced concrete walls with spread footings, underground utilities, and waterproofing installation
NT Superstructure – Structural steel columns & beams including Sideplate moment frames and reinforced concrete slab on metal decks
NT Exterior façade – Glass and Aluminum Curtainwall system with select areas of stick built glass and aluminum storefront system and light gauge framed penthouses with metal panels
NT Building Interiors – Light gauge framing and drywall and mechanical, electrical, plumbing and fire protection systems, medical equipment, interior specialties & finishes
Landscaping - Planting and site concrete, exterior lighting, signage, site structures, and driveways and parking



1 PH 7.0
1" = 100'-0"

PHASE 7 PARKING MATRIX

CODE REQ'D:	2,543
PROVIDED:	3,058

2 SITE FINISHED - PLAN - FINAL SITE PLAN ENTITLEMENT PHASING
1" = 100'-0"

Phase 7 – Hospital Construction and New Parking Lot Configuration

Phase 7 extends scope of construction to the NE parking lot where a new parking configuration will be constructed to accomodate the New Tower.

Final Site Plan

SITE PHASING - 7

1" = 100'-0"

JULY 19, 2021

CO ARCHITECTS

KAISER PERMANENTE

RIVERSIDE MEDICAL CENTER

Sheet: **A 1.24**

CITY OF RIVERSIDE

Site Development Plan Number:

OWNER: Kaiser Foundation Hospitals PHONE: 626.405.6333

ADDRESS: 393 E. Walnut Street Pasadena, CA 91188

ARCHITECT, ENGINEER, DESIGNER: CO Architects, Michael Baker International, Ridge Landscape Architects, Glumac

PHONE: 323.525.0500 (Architect)

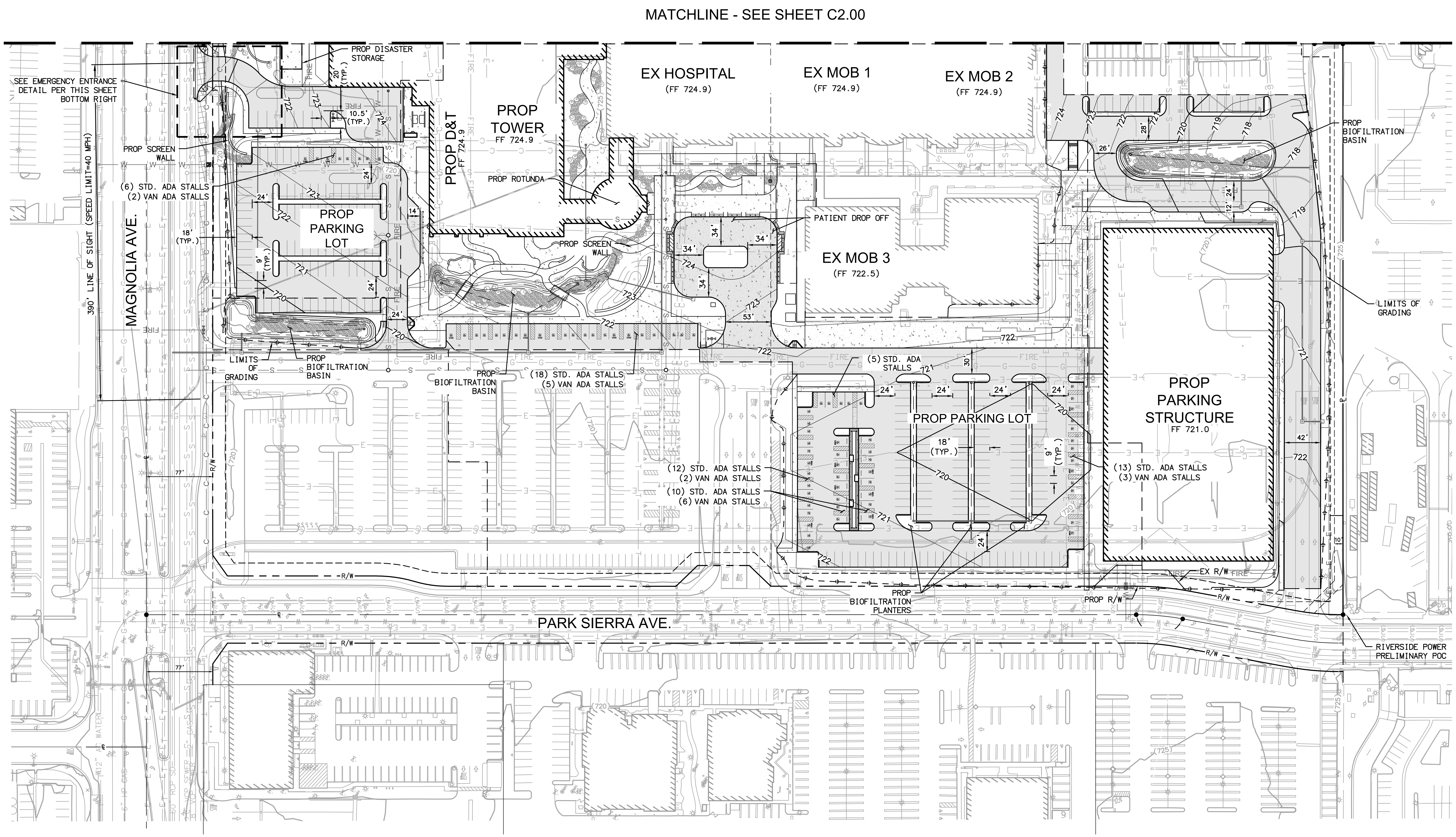
ADDRESS: 5055 Wilshire Blvd. 9th Floor, Los Angeles CA 90036 (Architect)

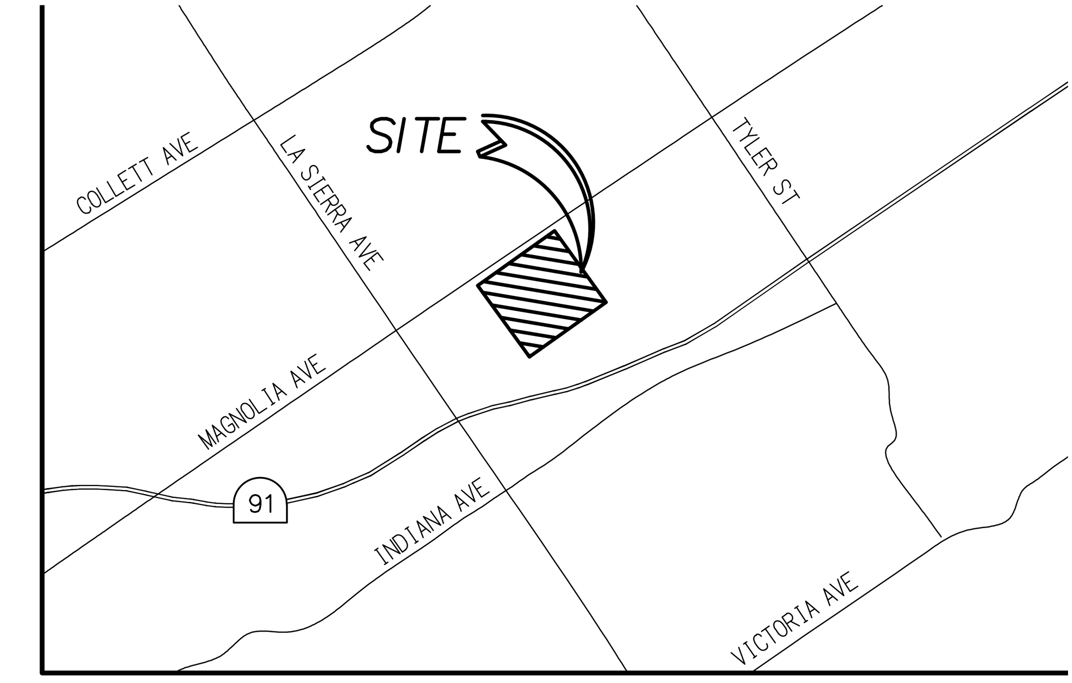
TYPE OF DEVELOPMENT: XXXXX

LOCATION: 10800 Magnolia Ave. Riverside, CA 92505

ZONE: XXXXX

ACCESSOR'S PARCEL NUMBER: 138-470-010

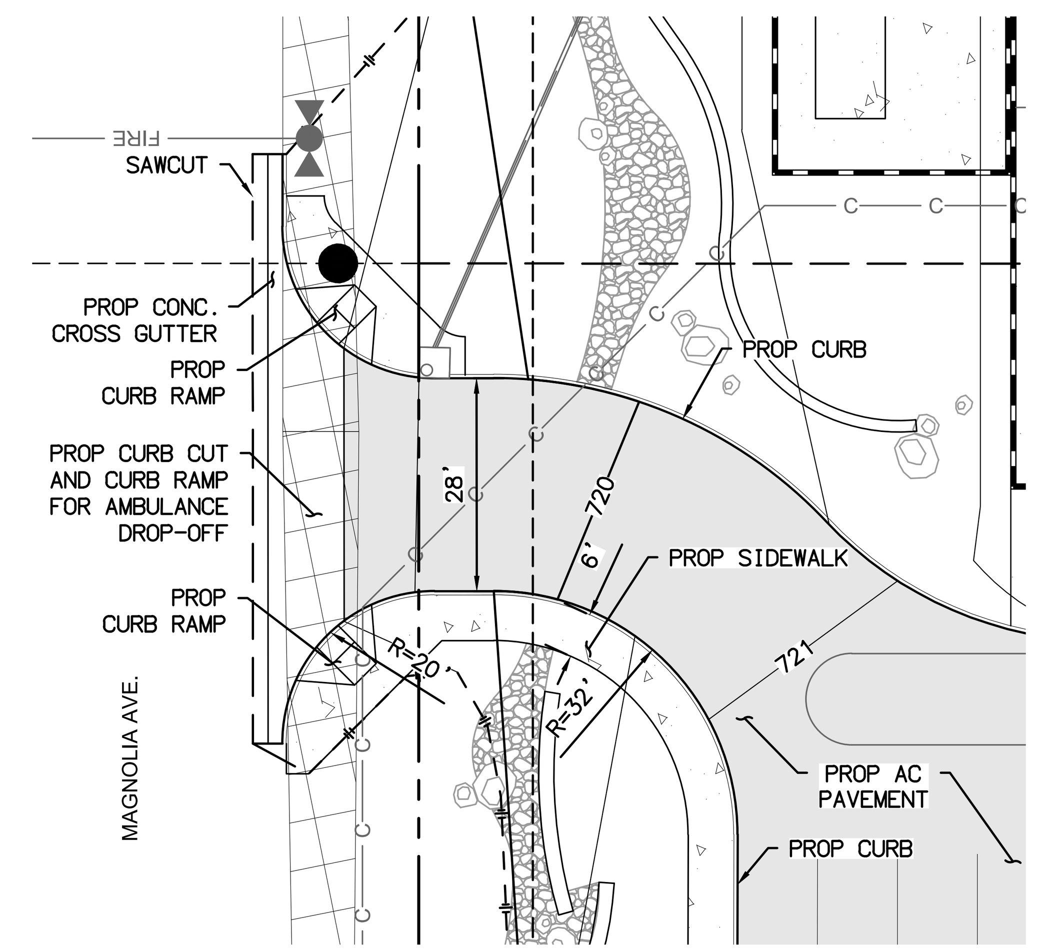




VICINITY MAP

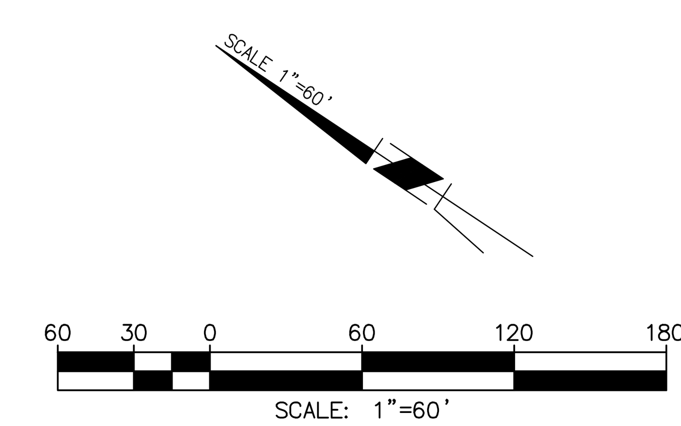
LEGEND

- RIGHT OF WAY
- PROPERTY LINE
- CURB AND GUTTER
- AC PAVEMENT
- CONCRETE PAVEMENT
- DECORATIVE CONCRETE PAVEMENT (PER LANDSCAPE PLANS)
- PROPOSED BUILDING
- ROOF OVERHANG



EMERGENCY ENTRANCE DETAIL

SCALE: 1" = 20'



SITE PLAN

MARCH 6, 2020

CO ARCHITECTS

KAISER PERMANENTE

RIVERSIDE MEDICAL CENTER

Sheet: C2.01	CITY OF RIVERSIDE
Site Development Plan Number:	
OWNER: Kaiser Foundation Hospitals PHONE: 626.405.5099	
ADDRESS: 393 E. Walnut Street Pasadena, CA 91188	
ARCHITECT, ENGINEER, DESIGNER: CO Architects, Michael Baker International, Ridge Landscape Architects, Glumac	PHONE: 323.525.0500 (Architect)
ADDRESS: 5055 Wilshire Blvd. 9th Floor, Los Angeles CA 90036 (Architect)	LOCATION: 10800 Magnolia Ave. Riverside, CA 92505
TYPE OF DEVELOPMENT: XXXXX	
ZONE: XXXXX	ACCESSOR'S PARCEL NUMBER: 138-470-010