June 16, 2016

Trammell Crow Company
3501 Jamboree Road, Suite 230
Newport Beach, California 92660

Attention: Mr. David Drake

Project No.: **16M123-1**

Subject: **Change of Engineer of Record, Response Report and Plan Review**
Building D
SEC of Oleander Avenue and Decker Road
Riverside County, California

References: 1) Geotechnical Investigation, Infiltration Study, and Rock Rippability Report for the Proposed Decker Assemblage Industrial Site, Located at the Southeast Corner of Oleander Avenue and Decker Road, Assessor's Parcel Numbers (APN's): 314-040-001, -002, -003, & -008, Western Perris Area, County of Riverside, California, prepared by Matrix Geotechnical Consulting, Inc., dated September 30, 2014.

2) Review Comments, Riverside County Planning Department, County Geologic Report No. 2491.

Gentlemen:

In accordance with your request, we have prepared this report to address the review sheet comments generated by the Riverside County Planning Department, following their review of the submitted geotechnical report, both of which are referenced above. A copy of the Riverside County review sheet is included herein. We are also providing additional grading, foundation, and pavement design information and recommendations.

Additionally, this letter shall document the change of geotechnical engineer of record responsibilities for the subject site. Southern California Geotechnical has reviewed the referenced report and will provide the remaining geotechnical services to completion of the project. These services will include preparation of response reports for the County of Riverside to achieve approval for grading and building permits, observation and testing services during construction, and preparation of compaction reports.

Current Site Conditions

The subject site is located southeast of the intersection of Oleander Avenue and Decker Road in an unincorporated area of Riverside County near Perris, California. The site is bordered to the north by the Oleander Avenue, to the east by vacant land, to the south by residential lots, and to the west by the Decker Road.

The site is a rectangular-shaped property, 34.5± acres in size, which is generally vacant and undeveloped. Ground surface cover consists of exposed soil with sparse amounts of native grass and weed growth. However, the southwest region of the property is developed with a single family residence. Remnants of a previous structure, including a concrete slab, are also located in the

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southwestern region of the site. Several irregular shaped outcrops of bedrock are visible throughout the site. The northeast quadrant of the overall site appears to be developed with a surficial layer of open-graded gravel.

Based on the site plan prepared by HPA, the site will be developed with one (1) new commercial/industrial building, identified as Building D, located in the central region of the overall site. Building D will be 702,645±ft^2 in size and will be constructed in a cross-dock configuration, with loading docks on the east and west sides of the building. We expect that the building will be surrounded by asphaltic concrete pavements in the automobile parking and drive areas and Portland cement concrete (PCC) pavements in the loading dock area.

Detailed structural information has not been provided. It is assumed that the building will be a single-story structure of tilt-up concrete construction, supported on conventional shallow foundations with a concrete slab-on-grade floor. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 80 kips and 3 to 5 kips per linear foot, respectively.

Previous Study

Matrix Geotechnical Consultants, Inc., (MGC) previously performed a geotechnical investigation for this site, the results of which were presented in the above referenced report. As part of this study, ten (10) borings were advanced to depths of 6½± to 31½± feet below existing site grades. In addition to the ten borings, sixteen (16) excavator pits were excavated to depths of 8½ to 25± feet below existing site grades. The previous study also included thirty-three (33) rotary percussion “air-track” borings to depths of 15 to 40 feet. Seven (7) seismic refraction survey lines were also performed at the site.

The soils encountered in the exploratory borings performed by MGC consisted of artificial fill soils, mainly located within the southwest and northeast portion of the site. The fill soils were generated from placement of crushed gravel throughout the northeast portion of the site and the creation of a gravel roadway. The fill materials that were encountered extended to depths of 2 to 6½± feet and consisted of light brown to brown medium dense silty sands, sands or silts, and crushed gravels. Very old alluvium was encountered beneath the fill materials and at the ground surface throughout the central and western portions of the property extending to the east. The alluvium consisted of medium dense silts, clayey sands, and silty sands. The Val Verde Tonalite was reported to underlie most of the site. The tonalite was described to be a similar chemical composition to gabbro, but includes a higher percentage of quartz. The tonalite was observed to be white-gray to gray and was found to be moderately hard to very hard.

Plan Review

At the time of the referenced geotechnical investigation, the proposed development consisted of one (1) new building, 714,000± ft^2 in size. The building was expected to be a single-story structure of concrete tilt-up construction, supported on a conventional shallow foundation with a concrete slab-on-grade floor. The remaining areas of the site were to be surrounded by asphaltic concrete pavements in the automobile parking and drive areas and Portland cement pavement (PCC) in the loading dock areas, with areas of concrete flatwork and some landscaping.

Based on our review, the referenced geotechnical investigation is considered applicable to the currently proposed development. No new subsurface exploration is considered warranted. This conclusion is based on a subsequent review of updated precise grading and foundation plans, as discussed in a subsequent section of this report.

Updated remedial grading and construction recommendations are presented with this report. This update report should be distributed to all consultants and contractors associated with this project, along with a copy of the original geotechnical investigations.

Further Plan Reviews

It is recommended that copies of the final grading plans, when they become available, also be provided to our office for review. We also recommend that our office review the foundation plans for the proposed development, as they become available.

Response to County Review Sheet

Each of the comments issued by the county of Riverside Planning Department (RPD) is presented below, followed by the Southern California Geotechnical, Inc. (SCG) response. A copy of the review sheet is enclosed with this correspondence for reference purposes.

RPD 1: *Please provide a discussion of the regional geologic setting including geologic province description, geomorphology of the project site, and geology of the vicinity.*

SCG: The subject site is located within the Peninsular Ranges province. The Peninsular Ranges province consists of several northwesterly-trending ranges in the southwestern California. The province is truncated to the north by the east-west trending Transverse Ranges. Prior to the mid-Mesozoic, the region was covered by seas and thick marine sedimentary and volcanic sequences were deposited. The bedrock geology that dominates the elevated areas of the Peninsular Ranges consists of high-grade metamorphic rocks intruded by Mesozoic plutons. During the Cretaceous, extensive mountain building occurred during the emplacement of the southern California batholith. The Peninsular Ranges have been significantly disrupted by Tertiary and Quaternary strike-slip faulting along the Elsinore and San Jacinto faults. This tectonic activity has resulted in the present terrain.

The primary available reference applicable to the subject site is the Geologic Map of the Steele Peak Quadrangle, Riverside County, California, published by Santa Barbara Museum of Natural History, Dibblee T. W., 2003. A portion of this map indicating the location of the subject site is included herein as Plate 3 of this report. This map indicates that the majority of the subject site is underlain by quartz diorite rocks of Cretaceous age (Map Symbol qdi). The quartz diorite rocks are described as gray to light gray, massive to more commonly gneissoid, composed mostly of plagioclase feldspar with the remainder of quartz, biotite, and hornblende. A portion of the northeastern corner of the site is mapped as older surficial sediments (Map Symbol Qoa). These soils are described as alluvial sand with gravel. Based on soils and bedrock that was encountered during the previous geotechnical report, the on-site materials appear to be consistent with the geologic mapping.

RPD 2: *Section 1.2 Location and Site Description (Page 4, 4th paragraph) contains the statement, "Confirmation of this condition is directly related to the upstream* *surface soil in excavator pit no. having highly saturated soil present at a depth of 20 feet." Please provide the missing excavator pit number.*

SCG: Based on our review of the Matrix report, we expect the missing excavator pit number is Test Pit No. 6.

RPD 3: *Section 2.4 Groundwater states that groundwater was encountered perched on top of the Tonalite. However, Section 2.6.2 Liquefaction & Seismically Induced Settlement states that "Groundwater was not identified below existing grade." Please clarify and revise.*

SCG: Based on the report by Matrix, the groundwater encountered is considered to be perched on the underlying impermeable bedrock and not representative of the regional groundwater levels.

The perched groundwater was considered during the liquefaction assessment. However, due to the dense to very dense alluvium and older alluvium underlain by very dense bedrock, liquefaction is not a design consideration for this project.

RPD 4: *Please provide a PGA_M and Seismic Design Category for the site.*

SCG: Seismic parameters and Site Design Category are included in a subsequent section of this report.

RPD 5: *Please provide a list of significant faults within 100 km of the project site, indicate name, magnitude and distance of each individually.*

SCG: The attached table was generated from the 2008 National Seismic Hazard Maps program from USGS website. The nearest active fault is the San Jacinto fault of the San Jacinto fault zone located 14.43 kilometers to the northeast.

RPD 6: *Please provide a listing of significant seismic events in the site vicinity (100 km).*

SCG: The attached table was generated from the earthquake archives from the USGS website. The listing includes significant seismic events within 100 km of the subject site. The largest event was a 6.4 magnitude located 7 km west-northwest of Newport Beach on March 11, 1933.

RPD 7: *Provide a reference for the aerial photography used for your analysis of geologic hazards on the site.*

SCG: Historical aerial photographs were provided by historicaerials.com and were reviewed to characterize any potential geologic hazard such as fault scarps, fault line scarps, landslides, etc. Aerial photographs from the following years were available for review: 1966, 1978, 1994, 2002, 2005, 2009 and 2012.

1966: Scale 1 inch = 500± feet

The subject site and the surrounding properties were vacant and undeveloped at the time of this photograph. A partial orchard consisting of approximately fifty (50) trees appears to be located in the west-central area of the site at the time this photograph was taken. The remaining areas of the site appear to consist of exposed soil with sparse to moderate native grass and weed growth. A natural drainage is visible trending roughly northeast-southwest across the subject site.

1978: Scale 1 inch = 500± feet

The partial orchard was no longer visible in this photograph. However, several large trees were present in the west-central area of the site at the time this photograph was taken. The surrounding area was generally unchanged from the previous photograph.

1994: Scale 1 inch = 500± feet

A building located in the southwest corner of the site is visible for this first time in this photograph. Several small canopy or shed structures are located east of the building. The ground surface surrounding the building and shed/canopy structures appear to consist of exposed soil with sparse native grass and weed growth.

2002: Scale 1 inch = 500± feet

Multiple small structures or canopies were present in the southwestern quadrant of the subject site at the time this photograph was taken. The remaining areas of the site are vacant and undeveloped.

2005: Scale 1 inch = 500± feet

The subject site is unchanged from the previous photograph.

2009: Scale 1 inch = 500± feet

The subject site is unchanged from the previous photograph.

2012: Scale 1 inch = 500± feet

The southwestern quadrant is still developed with one main building with several canopy/shed structures located east of the main building. The remaining areas of the subject property are still vacant and undeveloped.

No evidence of linear features (i.e. fault lines, fault line scarps) indicative of faulting was observed transecting the subject site or the surrounding area during our review of the historical aerial photographs

RPD 8: *Appendix C states that consolidation testing was performed and results are presented on Plate C-1, C-2, and C-3; however, these plates are not included in the report copies submitted to the County. Please provide.*

SCG: Plates C-1, C-2, and C-3 for the original MGC geotechnical investigation are included with this report.

RPD 9: *Please provide a north arrow and base map source for the Geotechnical Map.*

SCG: A north arrow and base map source has been included with the updated Geotechnical Map.

Geotechnical Report Update

This letter may serve as an update to the original geotechnical report. Provided that the updated recommendations contained within this report are implemented, the previous geotechnical report is considered valid for the currently proposed development. The following additional design information and recommendations shall supersede those contained in the previous geotechnical report.

Updated Site Stripping and Demolition Recommendations

Initial site stripping should include removal of any surficial vegetation. This should include any weeds, grasses, shrubs, and trees. Root masses and root balls associated with the shrubs and trees should be removed in their entirety, and the resultant excavations should be backfilled with compacted structural fill soils. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

Demolition of the existing residence present in the southwestern region of the site will be required. Demolition should include all foundations, floor-slabs, utilities and any other subsurface improvements that will not remain in place with the new development. Demolition resulting from demolition activities should be disposed of off-site

Updated Remedial Grading Recommendations

Treatment of Existing Soils: Building Pad Area

Remedial grading will be required in order to remove all of the undocumented fill soils and a portion of the near-surface alluvial soils. However, significant blasting will also be required at the site in order to achieve the new site grades. Based on the subsurface profile identified in the enclosed Geotechnical Map, Plate 2, blasting will generally be required in the cut areas located in the south and west regions of the site.

Within the cut areas of the site, it is recommended that blasting be performed to remove a portion of the bedrock and provide a uniform layer of compacted fill below the new floor-slab and new foundation elements. Removals are recommended to extend to a depth of 3 feet below pad grade. Within the influence zones of the new foundations, the overexcavation depth should also be sufficient to provide at least 2 feet of compacted fill below the proposed foundation bearing grades.

Within the fill portions of the site, the building pad should be overexcavated to a depth of at least 3 feet below existing grade. The overexcavation should also be sufficient to remove the undocumented fill soils in their entirety. Based on the subsurface exploration performed by Matrix, the fill soils extend to depths of 2 to 6½± feet below feet within isolated regions of the site. Where not encompassed within the general building pad overexcavations, additional overexcavation should be performed within the influence zones of the new foundations, extending to a depth of 2 feet below proposed bearing grade.

The overexcavation areas should extend at least 5 feet beyond the building and foundation perimeters, and to a horizontal extent equal to the depth of fill below the foundation bearing grades. If the proposed structures incorporate any exterior columns (such as for a canopy or overhang) the overexcavation should also encompass these areas.

Following completion of the overexcavation, the subgrade soils within the building areas should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade, as well as to support the foundation loads of the new structure. This evaluation should include proofrolling and probing to identify any soft, loose or otherwise unstable soils that must be removed. Some localized areas of deeper excavation may be required if loose, porous, or low density native soils are encountered at the base of the overexcavation. Soils suitable to serve as the structural fill subgrade within the building area should consist of either bedrock or dense alluvial soils that possess an in-situ dry density equal to at least 85 percent of the ASTM D-1557 maximum dry density.

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches, and moisture conditioned to a moisture content of 2 to 4 percent above optimum moisture content. The moisture conditioning of the overexcavation subgrade soils should be verified by the geotechnical engineer. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.

<u>Treatment of Existing Soils: Retaining Walls and Site Walls</u>

The existing soils within the areas of new retaining walls and non-retaining site walls should be overexcavated to a depth of 2 feet below foundation bearing grade and replaced as compacted structural fill, as discussed above for the proposed building pad. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

<u>Treatment of Existing Soils: Parking and Drive Areas</u>

Based on economic considerations, overexcavation of the existing soils in the new parking and drive areas is not considered warranted, with the exception of areas where lower strength, or unstable, soils are identified by the geotechnical engineer during grading. Subgrade preparation in the new parking and drive areas should initially consist of removal of all soils disturbed during stripping operations.

The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. Any such materials should be removed to a level of firm and unyielding soil. The exposed subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to at least 2 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength surficial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 2 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer. All fill should conform with the recommendations presented in the Grading Guide Specifications, included as Appendix D of this report. Specialized placement and sorting procedures will be required where existing granitic bedrock materials are used as fill. These considerations are discussed in a subsequent section of this report.
- All grading and fill placement activities should be completed in accordance with the requirements of the CBC and the grading code of Riverside County.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Fill soils should be well mixed.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

All imported structural fill should consist of low expansive (EI < 20), well graded soils possessing at least 10 percent fines (that portion of the sample passing the No. 200 sieve). Additional specifications for structural fill are presented in the Grading Guide Specifications, included as Appendix D.

Utility Trench Backfill

In general, all utility trench backfill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. As an alternative, a clean sand (minimum Sand Equivalent of 30) may be placed within trenches and compacted in place (jetting or flooding is not recommended). It is recommended that materials in excess of 3 inches in size not be used for utility trench backfill. Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by Riverside County. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

Utility trenches which parallel a footing, and extending below a 1h:1v plane projected from the outside edge of the footing should be backfilled with structural fill soils, compacted to at least 90 percent of the ASTM D-1557 standard. Pea gravel backfill should not be used for these trenches.

Updated Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pad will be underlain by structural fill soils used to replace undocumented fill soils, a portion of the near-surface native soils, and/or bedrock. The new structural fill soils are expected to extend to a depth of at least 2 feet below foundation bearing grade. Based on this subsurface profile, the proposed structure may be supported on a shallow foundation system.

Building Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,500 lbs/ft^2.
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: Two (2) No. 5 rebars (1 top and 1 bottom).
- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 24 inches below adjacent grade.
- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

The allowable bearing pressures presented above may be increased by 1/3 when considering short duration wind or seismic loads. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

The foundation subgrade soils should be evaluated at the time of overexcavation. It is further recommended that the foundation subgrade soils be evaluated by the geotechnical engineer immediately prior to steel or concrete placement. Within the new building area, soils suitable for direct foundation support should consist of newly placed structural fill, compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Any unsuitable materials should be removed to a depth of suitable bearing compacted structural fill or competent native alluvial soils, with the resulting excavations backfilled with compacted fill soils. As an alternative, lean concrete slurry (500 to 1,500 psi) may be used to backfill such isolated overexcavations.

The foundation subgrade soils should also be properly moisture conditioned to 2 to 4 percent of the Modified Proctor optimum, to a depth of at least 12 inches below bearing grade. **Since it is typically not feasible to increase the moisture content of the floor slab and foundation subgrade soils once rough grading has been completed, care should be taken to maintain the moisture content of the building pad subgrade soils throughout the construction process.**

Estimated Foundation Settlements

Post-construction total and differential settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.0 and 0.5 inches, respectively. Differential movements are expected to occur over a 50-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch.

Lateral Load Resistance

Lateral load resistance will be developed by a combination of friction acting at the base of foundations and slabs and the passive earth pressure developed by footings below grade. The following friction and passive pressure may be used to resist lateral forces:

- Passive Earth Pressure: 300 lbs/ft^3
- Friction Coefficient: 0.30

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against compacted structural fill. The maximum allowable passive pressure is 2500 lbs/ft^2.

Updated Floor Slab Design and Construction

Subgrades which will support the new floor slab should be prepared in accordance with the recommendations contained in the ***Updated Remedial Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, the floor of the new structure may be constructed as a conventional slab-on-grade supported on newly placed structural fill, extending to a depth of at least 3 feet below finished pad grade. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 6 inches.

- Modulus of Subgrade Reaction: k = 150 psi/in.

- Minimum slab reinforcement: Not required for geotechnical considerations. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading.

- Slab underlayment: If moisture sensitive floor coverings will be used then minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire area of the proposed slab where such moisture sensitive floor coverings are expected. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. A polyolefin material such as Stego® Wrap Vapor Barrier or equivalent will meet these specifications. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. Given that a rock free subgrade is anticipated and that a capillary break is not required, sand below the barrier is not required. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview. Where moisture sensitive floor coverings are not anticipated, the vapor barrier may be eliminated.

- Moisture condition the floor slab subgrade soils to 2 to 4 percent above the Modified Proctor optimum moisture content, to a depth of 12 inches. The moisture content of the floor slab subgrade soils should be verified by the geotechnical engineer within 24 hours prior to concrete placement.

- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.

The actual design of the floor slab should be completed by the structural engineer to verify adequate thickness and reinforcement.

Updated Seismic Design Parameters

The 2013 California Building Code (CBC) was adopted by all municipalities on January 1, 2014. The seismic design parameters provided in the referenced report were based on the 2010 CBC, which was in effect at the time of its issuance. Therefore, it is necessary to update the seismic design parameters to comply with the current code.

The 2013 CBC Seismic Design Parameters have been generated using U.S. Seismic Design Maps, a web-based software application developed by the United States Geological Survey. This software application, available at the USGS web site calculates seismic design parameters in accordance with the 2013 CBC, utilizing a database of deterministic site accelerations at 0.01 degree intervals. A copy of the output generated from this program is enclosed as Plate E-1 of this report. Based on this output, the following parameters may be utilized for the subject site:

2013 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.500
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.600
Site Class	---	D
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.500
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	0.900
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.000
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.600

Ground Motion Parameters

The peak ground acceleration (PGA_M) for this site was determined in accordance with Section 11.8.3 of ASCE 7-10. The parameter PGA_M is the maximum considered earthquake geometric mean (MCE_G) PGA, multiplied by the appropriate site coefficient from Table 11.8-1 of ASCE 7-10. The web-based software application U.S. Seismic Design Maps (described in the previous section) was used to determine PGA_M, using ASCE 7-10 as the building code reference document. A portion of the program output is included as Plate E-2 in Appendix E of this report. As indicated on Plate E-2, the PGA_M for this site is 0.50g.

Updated Retaining Wall Design and Construction

Retaining Wall Design Parameters

Based on the conditions encountered at the boring and test pit locations, the following parameters may be used in the design of new retaining walls for this site. We have provided parameters assuming the use of on-site soils for retaining wall backfill. The near surface soils generally consist of silty and clayey sands. Based on their classifications, these materials are expected to possess a friction angle of at least 30 degrees when compacted to 90 percent of the ASTM-1557 maximum dry density.

If desired, SCG could provide design parameters for an alternative select backfill material behind the retaining walls. The use of select backfill material could result in lower lateral earth pressures. In order to use the design parameters for the imported select fill, this material must be placed within the entire active failure wedge. This wedge is defined as extending from the heel of the retaining wall upwards at an angle of approximately 60° from horizontal. If select backfill material behind the retaining wall is desired, SCG should be contacted for supplementary recommendations.

RETAINING WALL DESIGN PARAMETERS

Design Parameter		**Soil Type**
		On-Site Silty Sands and Clayey Sands
Internal Friction Angle (ϕ)		30°
Unit Weight		130 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	43 lbs/ft^3
	Active Condition (2h:1v backfill)	70 lbs/ft^3
	At-Rest Condition (level backfill)	65 lbs/ft^3

Regardless of the backfill type, the walls should be designed using a soil-footing coefficient of friction of 0.30 and an equivalent passive pressure of 300 lbs/ft^3. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

The active earth pressure may be used for the design of retaining walls that do not directly support structures or support soils that in turn support structures and which will be allowed to deflect. The at-rest earth pressure should be used for walls that will not be allowed to deflect such as those which will support foundation bearing soils, or which will support foundation loads directly.

Where the soils on the toe side of the retaining wall are not covered by a "hard" surface such as a structure or pavement, the upper 1 foot of soil should be neglected when calculating passive resistance due to the potential for the material to become disturbed or degraded during the life of the structure.

Seismic Lateral Earth Pressures

In accordance with the 2013 CBC, any retaining walls more than 6 feet in height must be designed for seismic lateral earth pressures. The recommended seismic pressure distribution is triangular in shape, assumed to occur at the top of the wall, decreasing to 0 at the base of the wall. For a level backfill condition behind the top of the wall, the seismic lateral earth pressure is 18H lbs/ft^2, where H is the overall height of the wall. Where the ground surface above the wall consists of a 2h:1v (horizontal to vertical) sloping condition, the seismic lateral earth pressure is 52H lbs/ft^2. The seismic pressure distribution is based on the Mononobe-Okabe equation, utilizing a design acceleration of 0.33g. The 2013 CBC does not provide definitive guidance on determination of the design acceleration to be used in generating the seismic lateral earth pressure. In accordance with standard geotechnical practice, we have calculated the design acceleration as $^2/_3$ of the PGA_M. However, for combinations of high ground motion and steep slopes above the wall, the Mononobe-Okabe equation gives unrealistic high estimates of active earth pressures. Therefore, the seismic earth pressure for the sloping condition presented above was derived using a design acceleration equal to 50% of the PGA_m. The PGA_M for the subject site was obtained from the U.S. Seismic Design Maps application which is described in a previous section of this report.

Retaining Wall Foundation Design

The retaining wall foundations should be supported within newly placed compacted structural fill, extending to a depth of at least 2 feet below the proposed bearing grade. Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

Backfill Material

On-site soils may be used to backfill the retaining walls. However, all backfill material placed within 3 feet of the back wall face should have a particle size no greater than 3 inches. The retaining wall backfill materials should be well graded.

It is recommended that a properly installed prefabricated drainage composite such as the MiraDRAIN 6000XL (or approved equivalent), which is specifically designed for use behind retaining walls be used. If the drainage composite material is not covered by an impermeable surface, such as a structure or pavement, a 12-inch thick layer of a low permeability soil should be placed over the backfill to reduce surface water migration to the underlying soils. The drainage composite should be separated from the backfill soils by a suitable geotextile, approved by the geotechnical engineer.

All retaining wall backfill should be placed and compacted under engineering controlled conditions in the necessary layer thicknesses to ensure an in-place density between 90 and 93 percent of the maximum dry density as determined by the Modified Proctor test (ASTM D1557). Care should be taken to avoid over-compaction of the soils behind the retaining walls, and the use of heavy compaction equipment should be avoided.

Subsurface Drainage

As previously indicated, the retaining wall design parameters are based upon drained backfill conditions. Consequently, some form of permanent drainage system will be necessary in conjunction with the appropriate backfill material. Subsurface drainage may consist of either:

- A weep hole drainage system typically consisting of a series of 4-inch diameter holes in the wall situated slightly above the ground surface elevation on the exposed side of the wall and at an approximate 8-foot on-center spacing. The weep holes should include a 2 cubic foot pocket of open graded gravel, surrounded by an approved geotextile fabric, at each weep hole location.

- A 4-inch diameter perforated pipe surrounded by 2 cubic feet of gravel per linear foot of drain placed behind the wall, above the retaining wall footing. The gravel layer should be wrapped in a suitable geotextile fabric to reduce the potential for migration of fines. The footing drain should be extended to daylight or tied into a storm drainage system.

Updated Pavement Design Parameters

Site preparation in the pavement area should be completed as previously recommended in the ***Updated Remedial Grading Recommendations*** section of this report. The subsequent pavement recommendations assume proper drainage and construction monitoring, and are based on either PCA or CALTRANS design parameters for a twenty (20) year design period. However, these designs also assume a routine pavement maintenance program to obtain the anticipated 20-year pavement service life.

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The on-site soils generally consist of silty sands and clayey sands. Based on their classification, these materials are expected to possess good pavement support characteristics. Based on our previous work in the vicinity of the subject site, the on-site soils are expected to possess R-values ranging from 60 to 70. Therefore, the subsequent pavement design is based upon a conservative R-value of 60. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering controlled conditions. It may be desirable to perform R-value testing after the completion of rough grading to verify the R-value of the as-graded parking subgrade.

Asphaltic Concrete

Presented below are the recommended thicknesses for new flexible pavement structures consisting of asphaltic concrete over a granular base. The pavement designs are based on the traffic indices (TI's) indicated. The client and/or civil engineer should verify that these TI's are representative of the anticipated traffic volumes. If the client and/or civil engineer determine that the expected traffic volume will exceed the applicable traffic index, we should be contacted for supplementary recommendations. The design traffic indices equate to the following approximate daily traffic volumes over a 20-year design life, assuming six operational traffic days per week.

Traffic Index	No. of Heavy Trucks per Day
4.0	0
5.0	1
6.0	3
7.0	11
8.0	35

For the purpose of the traffic volumes indicated above, a truck is defined as a 5-axle tractor trailer unit with one 8-kip axle and two 32-kip tandem axles. All of the traffic indices allow for 1,000 automobiles per day.

ASPHALT PAVEMENTS (R = 60)					
Materials	**Thickness (inches)**				
	Auto Parking (TI = 4.0)	Auto Drive Lanes (TI = 5.0)	Truck Traffic		
			(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
Asphalt Concrete	3	3	3½	4	5
Aggregate Base	3	3	3	3	3
Compacted Subgrade	12	12	12	12	12

The aggregate base course should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density. The asphaltic concrete should be compacted to at least 95 percent of the Marshall maximum density, as determined by ASTM D-2726. The aggregate base course may consist of crushed aggregate base (CAB) or crushed miscellaneous base (CMB), which is a recycled gravel, asphalt and concrete material. The gradation, R-Value, Sand Equivalent, and Percentage Wear of the CAB or CMB should comply with appropriate specifications contained in the current edition of the "Greenbook" Standard Specifications for Public Works Construction.

Portland Cement Concrete

The preparation of the subgrade soils within concrete pavement areas should be performed as previously described for proposed asphalt pavement areas. The minimum recommended thicknesses for the Portland Cement Concrete pavement sections are as follows:

PORTLAND CEMENT CONCRETE PAVEMENTS				
Materials	**Thickness (inches)**			
	Auto Parking & Drives (TI = 5.0)	Truck Traffic		
		(TI =6.0)	(TI = 7.0)	(TI = 8.0)
PCC	5	5½	6	7
Compacted Subgrade	12	12	12	12

The concrete should have a 28-day compressive strength of at least 3,000 psi. The maximum joint spacing within all of the PCC pavements is recommended to be equal to or less than 30 times the pavement thickness.

Closure

We sincerely appreciate the opportunity to be of continued service on this project. We look forward to providing additional consulting services during the course of the project. If we may be of further assistance in any manner, please contact our office.

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

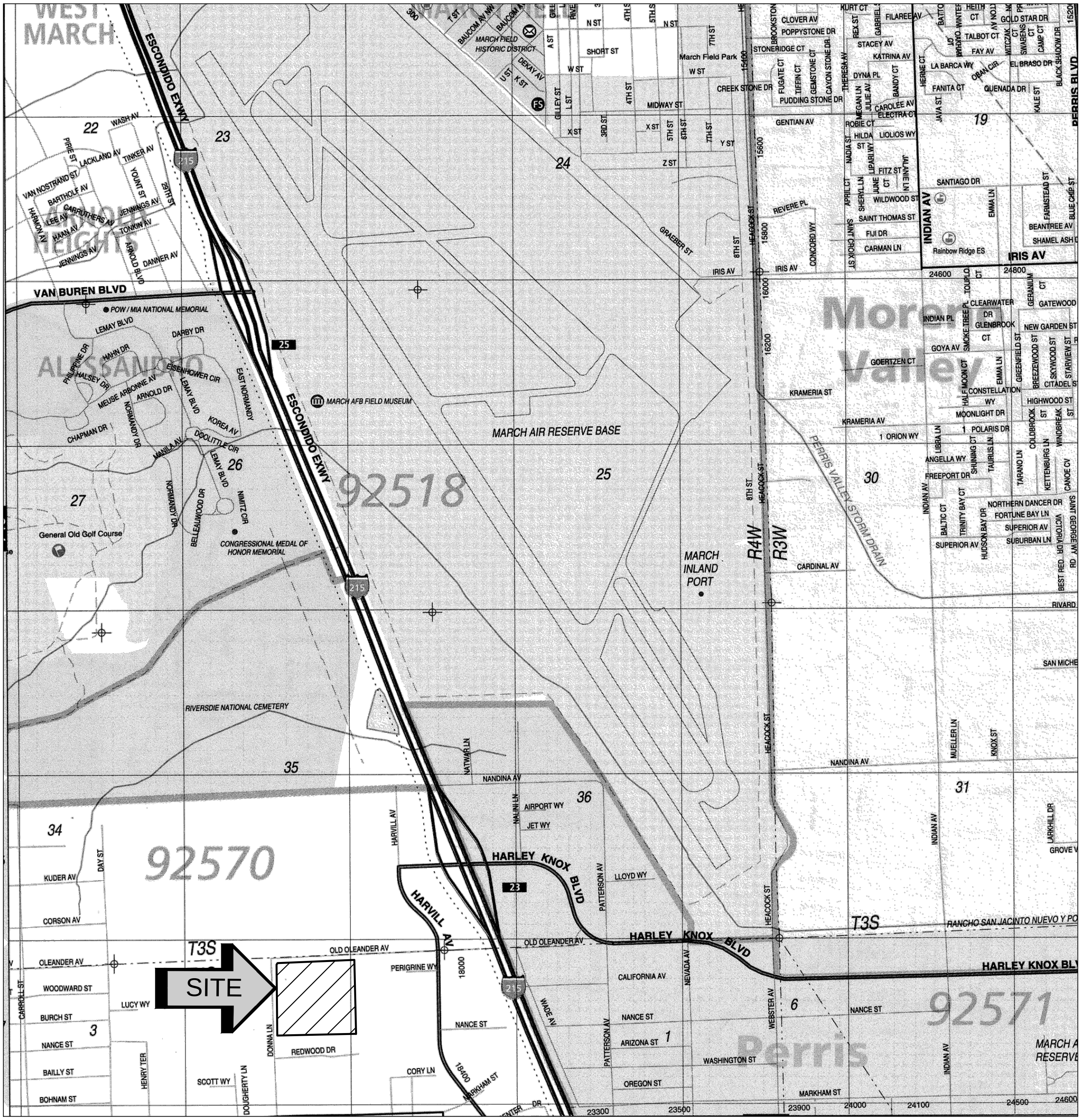
Pablo Montes Jr.
Staff Engineer

Daryl R. Kas, CEG 2467
Project Geologist

John A. Seminara, GE 2294
Principal Engineer

Enclosures:
- Plate 1 Site Location Map
- Plate 2 Updated Geotechnical Map
- Plate 3 Geologic Map
- Plates E-1 and E-2: Seismic Design Parameters
- MGD Consolidation Test Results (3 pages)
- Distance of Subject Site to Known Faults (3 pages)
- Recent Significant Seismic Events (4 pages)
- Riverside County Planning Department Review Sheet (3 pages)

Distribution: (2) Addressee



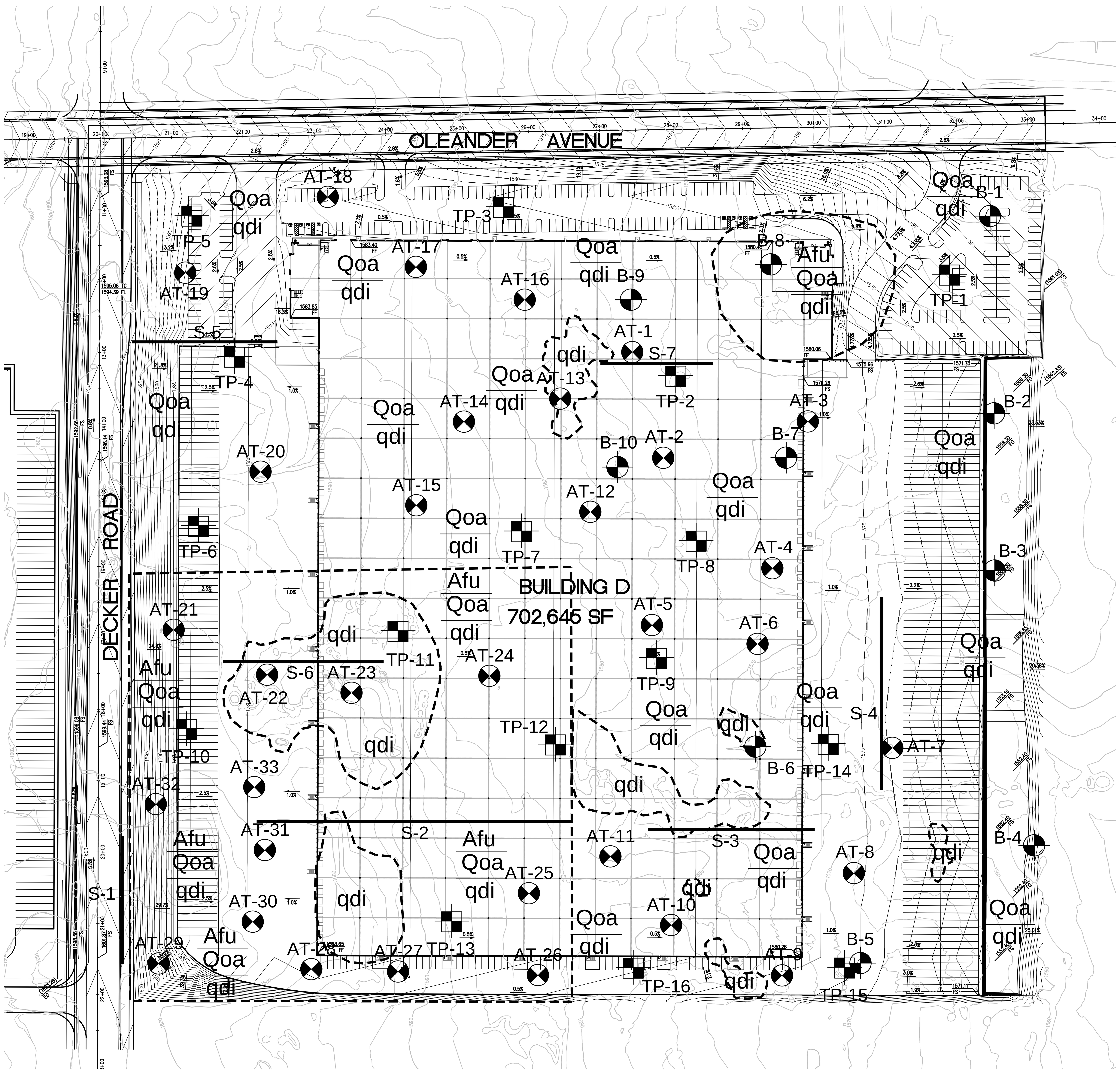
SOURCE: RIVERSIDE COUNTY
THOMAS GUIDE, 2013

SITE LOCATION MAP

BUILDING D

RIVERSIDE COUNTY, CALIFORNIA

SCALE: 1" = 2400'	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: PM CHKD: JAS	
SCG PROJECT 16M123-1	
PLATE 1	



GEOTECHNICAL LEGEND

Afu	Undocumented Fill
Qoa	OLDER ALLUVIUM
qdi	QUARTZ DIORITE (VAL VERDE TONALITE)
	APPROXIMATE GEOLOGIC CONTACT
B-1	PREVIOUS HOLLOW STEM BORING LOCATION MATRIX GEOTECHNICAL CONSULTING (MGD) PROJECT NO. M113-004
AT-1	PREVIOUS AIR-TRACK BORING LOCATION MGD PROJECT NO. M113-004
TP-1	PREVIOUS TEST PIT LOCATION MGD PROJECT NO. M113-004
SL-1	PREVIOUS SEISMIC LINE LOCATION MGD PROJECT NO. M113-004

NORTH
SoCalGeo

NOTE: BASE MAP PROVIDED BY DAVID EVANS AND ASSOCIATES, INC.

22885 Savi Ranch Parkway
Suite E
Yorba Linda, CA 92887
Phone: (714) 685-1115
Fax: (714) 685-1118
www.socalgeo.com

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation
SoCalGeo

SCALE: 1" = 80'
DRAWN: PM
CHKD: JAS
SCG PROJECT
16M123-1

UPDATED GEOTECHNICAL MAP
BUILDING D
RIVERSIDE COUNTY, CALIFORNIA

PLATE
2

LEGEND

af
Qa

SURFICIAL SEDIMENTS
Alluvial sediments, unconsolidated, undissected

af *Artificial fill at construction site of March Airforce base in northeast area and at 2 sites over buried Colorado River aqueduct near Cajalco Road in north area*
Qa *Alluvial sand and clay of valley areas, covered with gray clay soil*

Qoa

OLDER SURFICIAL SEDIMENTS
Slightly indurated, much dissected alluvial sediments

Qoa *Alluvial sand, commonly pebbly, light reddish brown, arkosic, includes alluvial fan gravel at base of hill terranes*

— *UNCONFORMITY* —

Tcg
Tcrr
Ts

TERRESTRIAL SEDIMENTARY DEPOSITS
Indurated, massively bedded, form erosional remnants of once more extensive alluvial stream-laid deposits at or near west border of quadrangle; age, Tertiary, probably Pliocene or late Miocene

Tcg *Conglomerate of Lake Mathews area (Morton, 2001), of granitic cobbles in light gray coarse grained sandstone matrix*
Tcrr *Conglomerate of red rhyolitic cobbles in coarse grained sandstone matrix, exposed at west border of quadrangle and beyond into Lake Mathews area*
Ts *Mudstone and minor conglomerate and sandstone of Lake Mathews Formation (Morton, 2001)*

— *UNCONFORMITY* —

p gr
grd qdi qdu hdg qdc grb

PLUTONIC ROCKS
Mostly medium grained holocrystalline plutonic rocks of Peninsular Range batholith of Cretaceous age; includes post – plutonic granitic dike rocks of variable grain size, also of Cretaceous age

p *Pegmatite dike rocks, leucocratic, very coarse grained; of quartz, alkali feldspar biotite and mica; dikes as wide as 2 m, forms dike swarm in quartz diorite in central west area, and scattered dikes elsewhere*
gr *Granitic dike rocks, leucocratic, tan – white, fine to medium grained, massive; of quartz, alkali feldspars, and minor biotite; form scattered dikes, some large, intrusive into quartz diorite (**qdi**)*
grd *Granodiorite, ranging to quartz monzonite in north area; includes Woodson Mountain Granodiorite of Larsen 1948; Steele Valley Pluton of Dudley 1935, and Arroyo del Toro Pluton of Morton, 2001,) in southwest area; leucocratic, tan-white, massive, homogeneous, of quartz, alkali feldspars and minor biotite*
qdi *Quartz diorite (includes Perris quartz diorite of Dudley, 1935, renamed Vat Verde Tonalite by Osborn, 1939, included in Bonsal Tonalite of Larsen, 1948, and Val Verde Tonalite by Morton and Cox, 2001, in east area: gray to light gray, massive to more commonly gneissoid, composed mostly of sodic plagioclase feldspar and the remainder of quartz, biotite and hornblende, and very minor potassic feldspar, contains few to abundant dark gray discoid inclusions; (xenoliths) oriented parallel to gneissoid structure of rock; radiometric age 105.7 MA, Ar 40/Ar 39 age of hornblende, 100 MA, biotite 95 MA, and potassic feldspar 85.5 MA (Morton 2001)*
qdu *Quartz diorite, similar to qdi but massive, contains hypersthene, few equant (non-discoid inclusions; radiometric age of zircon 112.9 and 113.6 MA (Morton, 2001)*
hdg *Hornblende gabbro, included in San Marcos Gabbro by Larsen, 1948, gray-black, weathers brown, medium to coarse grained, massive, of hornblende and calcic plagioclase feldspar, some hornblende anhedra, very large, poikilitic*
qdc *Quartz diorite, cataclastic, gray, homogeneous but very gneissoid due to parallel orientation of biotite, composition about same as **qdi** but richer in biotite and hornblende; gneissoid structure due in part to thin cataclastic or mylonitic laminae; rock sheared at depth in a metamorphic environment, part of Santa Rosa shear zone; contacts with adjacent rocks (**qdi** and **ms**) gradational, difficult to map*
grb *"Black granite", hyperthene monzogranite of Morton 2001, nearly black, weathers dark brown, massive, composed of quartz, alkali feldspar, biotite, hornblende, hyperthene and clinopyroxene, exposed in one area north of Steele Valley, formerly quarried as "black granite" building stone; radiometric age of zircon 109 and 106 MA, Ar 40/Ar39 of biotite 104.5 MA and potassic feldspar 99.3 MA (Morton 2001)*

SOURCE: "GEOLOGIC MAP OF THE STEELE PEAK 7.5' QUADRANGLE, RIVERSIDE COUNTIY, CALIFORNIA" DIBBLEE, 2003

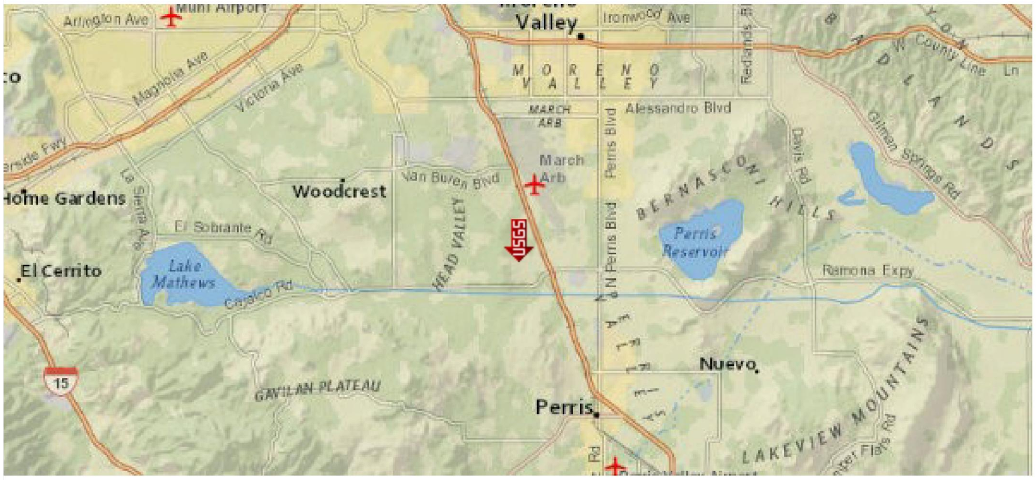
GEOLOGIC MAP
BUILDING D
RIVERSIDE COUNTY, CALIFORNIA

SCALE: 1" = 2000'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: DRK CHKD: JAS	
SCG PROJECT 16M123-1	
PLATE 3	

USGS Design Maps Summary Report

User–Specified Input

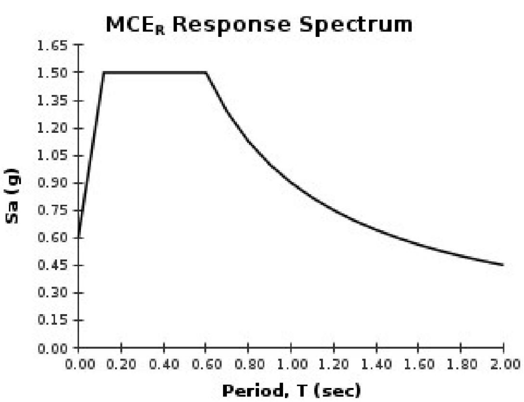
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	33.85719°N, 117.26796°W
Site Soil Classification	Site Class D – “Stiff Soil”
Risk Category	I/II/III

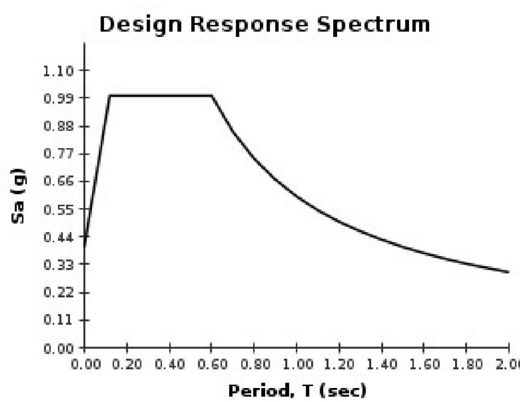


USGS–Provided Output

S_S =	1.500 g	S_{MS} =	1.500 g	S_{DS} =	1.000 g
S_1 =	0.600 g	S_{M1} =	0.900 g	S_{D1} =	0.600 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the “2009 NEHRP” building code reference document.





SOURCE: U.S. GEOLOGICAL SURVEY (USGS)
<http://geohazards.usgs.gov/designmaps/us/application.php>

SEISMIC DESIGN PARAMETERS

BUILDING D

MORENO VALLEY, CALIFORNIA

DRAWN: AL
CHKD: JAS

SCG PROJECT
16M123-1

PLATE E-1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.500

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.500 = 0.5$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.500 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 1.077

From Figure 22-18 [6] C_{R1} = 1.046

SOURCE: U.S. GEOLOGICAL SURVEY (USGS)
<http://geohazards.usgs.gov/designmaps/us/application.php>

MCE PEAK GROUND ACCELERATION
BUILDING D
RIVERSIDE COUNTY, CALIFORNIA

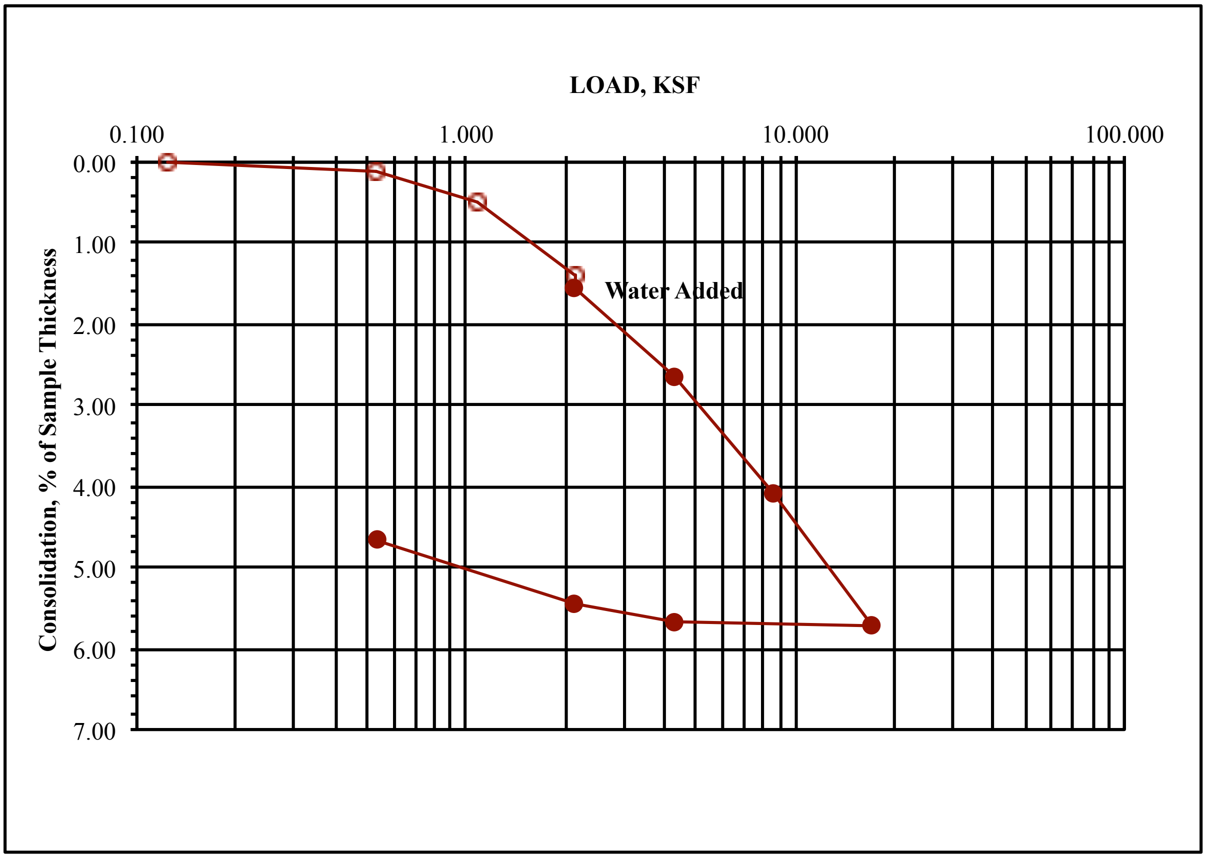
DRAWN: PM
CHKD: RGT

SCG PROJECT
16M123-1

PLATE E-2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

CONSOLIDATION TEST



Note: Solid circle denotes sample innudation

	In-place	Remolded:	
Dry Density, (pcf):		116.55	90% of Max Dry Density
Water Content (%):		9.6	

Bore/Sample/Depth(ft): B-2, Bulk 0-5'

SOIL DESCRIPTION: Silty Sand, dark yellow brown (10YR 4/6)

U.S.C.S. SM-SC

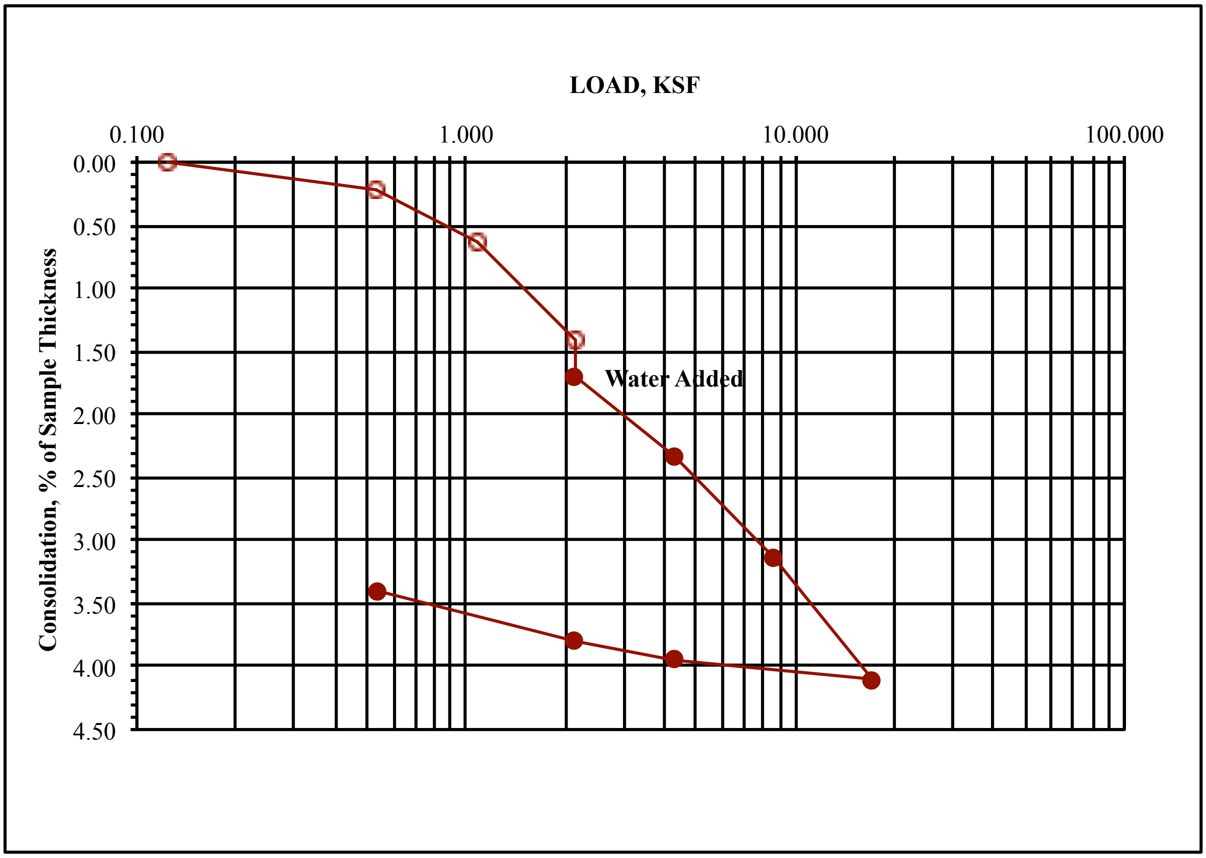
P.N. M1103-004 **LOCATION:** West Perris

CLIENT: Trammell-Crow Company

MATRIX

Plate: C-1

CONSOLIDATION TEST



Note: Solid circle denotes sample innudation

	In-place	Remolded:	
Dry Density, (pcf):		114.3	90% of Max Dry Density
Water Content (%):		10.7	

Bore/Sample/Depth(ft): B-5, Bulk 5-7'

SOIL DESCRIPTION: Silty Sand, dark yellow brown (10YR 4/6)

U.S.C.S. SM-SC

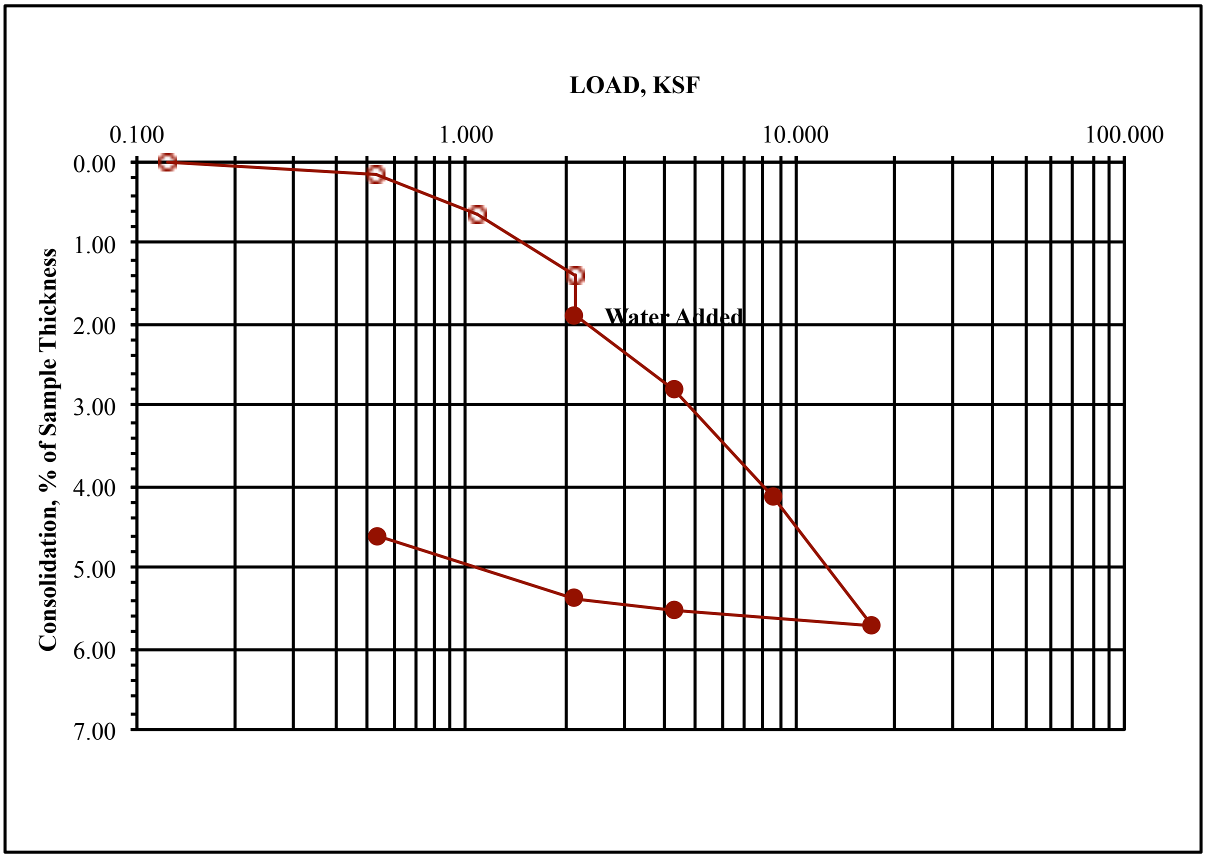
P.N. M1103-004 **LOCATION:** West Perris

CLIENT: Trammell-Crow Company

MATRIX

Plate: C-2

CONSOLIDATION TEST



Note: Solid circle denotes sample innudation

	In-place	Remolded:
Dry Density, (pcf):	113.5	
Water Content (%):	5.6	

Bore/Sample/Depth(ft): B-4, Ring: 7.5'

SOIL DESCRIPTION: Silty Sand, dark yellow brown (10YR 4/6)

U.S.C.S. SM-SC

P.N. M1103-004 **LOCATION:** West Perris

CLIENT: Trammell-Crow Company

MATRIX

Plate: C-3

DISTANCE OF SUBJECT SITE TO KNOWN FAULTS

Fault Name	Distance to Site (km)	Maximum Earthquake Magnitude
San Jacinto;SJV	14.43	7.04
San Jacinto;SBV+SJV+A+T	14.43	7.73
San Jacinto;SBV+SJV+A	14.43	7.62
San Jacinto;SBV+SJV	14.43	7.35
San Jacinto;SBV+SJV+A+CC+B	14.43	7.77
San Jacinto;SBV+SJV+A+CC+B+SM	14.43	7.80
San Jacinto;SJV+A	14.43	7.47
San Jacinto;SJV+A+C	14.43	7.63
San Jacinto;SJV+A+CC	14.43	7.61
San Jacinto;SJV+A+CC+B	14.43	7.67
San Jacinto;SJV+A+CC+B+SM	14.43	7.72
San Jacinto;SBV+SJV+A+CC	14.43	7.72
San Jacinto;A+CC	14.99	7.47
San Jacinto;A+CC+B	14.99	7.56
San Jacinto;A+CC+B+SM	14.99	7.62
San Jacinto;A	14.99	7.28
San Jacinto;A+C	14.99	7.50
San Jacinto;SBV	17.17	7.06
Elsinore;W+GI+T	21.69	7.48
Elsinore;GI	21.69	6.89
Elsinore;GI+T+J+CM	21.69	7.70
Elsinore;GI+T+J	21.69	7.62
Elsinore;GI+T	21.69	7.29
Elsinore;W+GI+T+J+CM	21.69	7.79
Elsinore;W+GI+T+J	21.69	7.72
Elsinore;W+GI	21.69	7.27
Elsinore;T+J+CM	24.55	7.63
Elsinore;T	24.55	7.07
Elsinore;T+J	24.55	7.54
Chino, alt 2	27.98	6.80
Elsinore;W	30.14	7.03
Chino, alt 1	30.56	6.70
S. San Andreas;SSB+BG+CO	30.66	7.52
S. San Andreas;BB+NM+SM+NSB+SSB	30.66	7.76
S. San Andreas;BB+NM+SM+NSB+SSB+BG	30.66	7.85
S. San Andreas;NM+SM+NSB+SSB+BG	30.66	7.77
S. San Andreas;BB+NM+SM+NSB+SSB+BG+CO	30.66	7.91
S. San Andreas;SSB	30.66	6.95
S. San Andreas;SM+NSB+SSB+BG+CO	30.66	7.78
S. San Andreas;NM+SM+NSB+SSB+BG+CO	30.66	7.84
S. San Andreas;NSB+SSB	30.66	7.20
S. San Andreas;NSB+SSB+BG	30.66	7.47
S. San Andreas;PK+CH+CC+BB+NM+SM+NSB+SSB	30.66	7.93
S. San Andreas;PK+CH+CC+BB+NM+SM+NSB+SSB+BG	30.66	7.99
S. San Andreas;CH+CC+BB+NM+SM+NSB+SSB+BG+CO	30.66	8.03
S. San Andreas;PK+CH+CC+BB+NM+SM+NSB+SSB+BG+CO	30.66	8.04
S. San Andreas;SSB+BG	30.66	7.35
S. San Andreas;NSB+SSB+BG+CO	30.66	7.61
S. San Andreas;SM+NSB+SSB	30.66	7.56

Fault Name	Distance to Site (km)	Maximum Earthquake Magnitude
S. San Andreas;CC+BB+NM+SM+NSB+SSB+BG+CO	30.66	7.98
S. San Andreas;CH+CC+BB+NM+SM+NSB+SSB	30.66	7.92
S. San Andreas;CH+CC+BB+NM+SM+NSB+SSB+BG	30.66	7.98
S. San Andreas;NM+SM+NSB+SSB	30.66	7.65
S. San Andreas;CC+BB+NM+SM+NSB+SSB+BG	30.66	7.93
S. San Andreas;CC+BB+NM+SM+NSB+SSB	30.66	7.85
S. San Andreas;SM+NSB+SSB+BG	30.66	7.70
S. San Andreas;PK+CH+CC+BB+NM+SM+NSB	31.97	7.88
S. San Andreas;CC+BB+NM+SM+NSB	31.97	7.79
S. San Andreas;CH+CC+BB+NM+SM+NSB	31.97	7.87
S. San Andreas;NM+SM+NSB	31.97	7.56
S. San Andreas;NSB	31.97	6.86
S. San Andreas;BB+NM+SM+NSB	31.97	7.68
S. San Andreas;SM+NSB	31.97	7.44
Cucamonga	38.39	6.70
S. San Andreas;BG+CO	44.19	7.39
S. San Andreas;BG	44.19	7.13
Cleghorn	45.5	6.80
San Jose	47.9	6.70
San Joaquin Hills	47.96	7.10
North Frontal (West)	50.22	7.20
Sierra Madre	52.27	7.20
Sierra Madre Connected	52.27	7.30
Pinto Mtn	54.78	7.30
Puente Hills (Coyote Hills)	55.65	6.90
S. San Andreas;NM+SM	56.48	7.46
S. San Andreas;PK+CH+CC+BB+NM+SM	56.48	7.83
S. San Andreas;SM	56.48	7.31
S. San Andreas;CH+CC+BB+NM+SM	56.48	7.83
S. San Andreas;CC+BB+NM+SM	56.48	7.74
S. San Andreas;BB+NM+SM	56.48	7.61
Elsinore;J	62.84	7.35
Elsinore;J+CM	62.84	7.49
Newport Inglewood Connected alt 1	63.12	7.50
Newport Inglewood Connected alt 2	63.12	7.50
Newport-Inglewood (Offshore)	63.12	7.00
Helendale-So Lockhart	65.96	7.40
Clamshell-Sawpit	67.77	6.70
Newport-Inglewood, alt 1	67.8	7.20
North Frontal (East)	67.88	7.00
Puente Hills (Santa Fe Springs)	69.83	6.70
Raymond	74.54	6.80
Lenwood-Lockhart-Old Woman Springs	78.85	7.50
San Jacinto;CC	80.21	7.03
San Jacinto;CC+B+SM	80.21	7.35
San Jacinto;CC+B	80.21	7.24
Elysian Park (Upper)	80.22	6.70
Puente Hills (LA)	80.27	7.00
San Jacinto;C	81.32	7.10
Burnt Mtn	81.46	6.80

Fault Name	Distance to Site (km)	Maximum Earthquake Magnitude
Rose Canyon	83.53	6.90
Landers	85.07	7.40
Eureka Peak	85.55	6.70
Palos Verdes Connected	86.44	7.70
Palos Verdes	86.44	7.30
Verdugo	87.08	6.90
Coronado Bank	88.56	7.40
Johnson Valley (No)	88.91	6.90
Hollywood	93.36	6.70
S. San Andreas;CO	94.88	7.04
Santa Monica Connected alt 2	97.81	7.40
Earthquake Valley	98.78	6.80
So Emerson-Copper Mtn	99.08	7.10

RECENT SIGNIFICANT SEISMIC EVENTS

Magnitude	Location of Event	Date/Time of Event
4.6	11km NE of Running Springs	2014-07-05 16:59:34 UTC
5.1	2km NW of Brea, CA	2014-03-29 04:09:42 UTC
4.7	21km ESE of Anza, CA	2013-03-11 16:56:06 UTC
5.4	20km NNW of Borrego Springs, CA	2010-07-07 23:53:33 UTC
4.9	4.9 15km NNW of Borrego Springs, CA	2010-06-13 03:08:57 UTC
4.7	2km E of Lennox, CA	2009-05-18 03:39:36 UTC
5.4	5km S of Chino Hills, CA	2008-07-29 18:42:15 UTC
4.7	13km NE of Trabuco Canyon, CA	2007-09-02 17:29:14 UTC
4.9	4km NE of Yucaipa, CA	2005-06-16 20:53:26 UTC
5.2	10km ESE of Anza, CA	2005-06-12 15:41:46 UTC
4.6	6km N of Big Bear City, CA	2003-02-25 04:03:04 UTC
4.5	5km N of Big Bear City, CA	2003-02-22 19:33:45 UTC
5.0	6km N of Big Bear City, CA	2003-02-22 12:19:10 UTC
4.8	4km NE of Yorba Linda, CA	2002-09-03 07:08:51 UTC
5.0	16km ESE of Anza, CA	2001-10-31 07:56:16 UTC
4.7	6km NNW of Big Bear Lake, CA	2001-02-10 21:05:05 UTC
5.6	7km ENE of Running Springs, CA	1999-10-16 09:59:38 UTC
4.9	8km SE of Yucca Valley, California	1999-05-14 07:54:03 UTC
4.8	7km N of Big Bear City, California	1998-10-27 01:08:40 UTC
4.6	14km NE of Yucaipa, California	1998-10-01 18:18:15 UTC
4.8	14km S of Big Bear Lake, California	1998-08-16 13:34:40 UTC
4.9	16km N of Borrego Springs, California	1997-07-26 03:14:55 UTC
4.8	13km NE of Thousand Palms, California	1995-05-07 11:03:33 UTC
5.0	17km NNW of Joshua Tree, California	1994-06-16 16:24:27 UTC
4.8	2km SE of Running Springs, California	1994-04-06 19:01:04 UTC
5.0	12km S of Joshua Tree, California	1993-08-21 01:46:38 UTC
4.7	9km SSE of Lucerne Valley, California	1992-12-04 05:25:11 UTC
4.6	9km SSE of Lucerne Valley, California	1992-12-04 05:25:07 UTC
5.3	10km SE of Lucerne Valley, California	1992-12-04 02:08:57 UTC
5.3	10km NNW of Big Bear City, California	1992-11-27 16:00:57 UTC
5.3	9km SE of Yucca Valley, California	1992-09-15 08:47:11 UTC
5.2	7km SE of Big Bear Lake, California	1992-08-17 20:41:52 UTC
4.7	4km SE of Yucca Valley, California	1992-08-15 08:24:14 UTC
4.6	29km N of Yucca Valley, California	1992-08-08 15:37:43 UTC
4.6	2km E of Yucca Valley, California	1992-07-28 18:27:03 UTC
4.8	15km NNE of Thousand Palms, California	1992-07-25 04:31:59 UTC
5.0	13km NE of Thousand Palms, California	1992-07-24 18:14:36 UTC
4.9	3km SSE of Big Bear City, California	1992-07-09 01:43:57 UTC
5.3	24km N of Yucca Valley, California	1992-07-01 07:40:29 UTC
4.8	16km NW of Morongo Valley, CA	1992-06-30 21:22:54 UTC
5.0	14km ENE of Desert Hot Springs, CA	1992-06-30 14:38:11 UTC
4.6	12km E of Big Bear City, California	1992-06-29 16:41:41 UTC
4.8	13km ENE of Thousand Palms, CA	1992-06-29 16:01:42 UTC

Magnitude	Location of Event	Date/Time of Event
4.6	10km NNE of Yucaipa, CA	1992-06-29 14:41:26 UTC
4.6	6km SE of Yucca Valley, California	1992-06-29 14:31:30 UTC
5.1	4km ESE of Yucca Valley, CA	1992-06-29 14:13:38 UTC
5.7	3km ESE of Yucca Valley, California	1992-06-29 14:08:37 UTC
4.7	2km SSE of Yucca Valley, California	1992-06-28 21:13:16 UTC
5.3	1km N of Big Bear Lake, California	1992-06-28 17:05:57 UTC
5.0	7km S of Big Bear Lake, CA	1992-06-28 17:01:32 UTC
4.8	11km SE of Big Bear City, CA	1992-06-28 15:24:29 UTC
4.6	10km SE of Big Bear City, CA	1992-06-28 15:18:33 UTC
4.7	15km SSE of Big Bear Lake, CA	1992-06-28 15:17:14 UTC
6.3	7km SSE of Big Bear City, CA	1992-06-28 15:05:30 UTC
4.5	12km S of Big Bear City, CA	1992-06-28 15:04:51 UTC
5.5	11km SSE of Big Bear Lake, California	1992-06-28 14:43:21 UTC
5.0	3km ESE of Yucca Valley, California	1992-06-28 13:50:46 UTC
4.9	6km NNE of Yucca Valley, California	1992-06-28 13:26:05 UTC
4.5	4km ESE of Yucca Valley, California	1992-06-28 13:18:15 UTC
4.9	34km NNW of Joshua Tree, CA	1992-06-28 13:10:50 UTC
5.4	26km NNW of Yucca Valley, CA	1992-06-28 12:40:53 UTC
5.5	5 0km E of Yucca Valley, CA	1992-06-28 12:36:40 UTC
4.5	4km WNW of Joshua Tree, California	1992-06-28 12:02:58 UTC
5.0	1km SSE of Yucca Valley, California	1992-06-28 12:02:31 UTC
5.0	7km SSW of Yucca Valley, California	1992-06-28 12:02:16 UTC
5.7	2km SSW of Joshua Tree, California	1992-06-28 12:01:16 UTC
5.8	3km NE of Yucca Valley, California	1992-06-28 12:00:45 UTC
7.3	10km N of Yucca Valley, CA	1992-06-28 11:57:34 UTC
5.0	15km E of Desert Hot Springs, California	1992-05-18 15:44:17 UTC
4.8	15km NNE of Thousand Palms, California	1992-05-06 02:38:43 UTC
5.0	16km NNE of Thousand Palms, California	1992-05-04 16:19:49 UTC
6.1	17km NNE of Thousand Palms, California	1992-04-23 04:50:23 UTC
4.5	17km NNE of Thousand Palms, California	1992-04-23 02:25:29 UTC
5.8	13km NNE of Sierra Madre, CA	1991-06-28 14:43:54 UTC
4.5	1km N of Claremont, CA	1990-04-17 22:32:27 UTC
4.7	6km N of Claremont, CA	1990-03-01 03:23:03 UTC
5.5	6km NNE of Claremont, CA	1990-02-28 23:43:36 UTC
4.8	1km WSW of East Los Angeles, CA	1989-06-12 16:57:18 UTC
4.8	2km E of Newport Beach, CA	1989-04-07 20:07:30 UTC
5.0	12km SW of Morongo Valley, CA	1988-12-16 05:53:04 UTC
5.0	1km SSE of Pasadena, CA	1988-12-03 11:38:26 UTC
4.8	18km SW of Newport Beach, CA	1988-11-20 05:39:28 UTC
4.7	4km NNE of Claremont, CA	1988-06-26 15:04:58 UTC
4.7	2km WNW of El Monte, CA	1988-02-11 15:25:55 UTC
5.3	2km WSW of Rosemead, CA	1987-10-04 10:59:38 UTC
4.6	3km ESE of Monterey Park, CA	1987-10-01 15:12:31 UTC
4.8	2km E of Monterey Park, CA	1987-10-01 14:49:05 UTC

Magnitude	Location of Event	Date/Time of Event
4.7	3km SE of Monterey Park, CA	1987-10-01 14:45:41 UTC
5.9	2km SSW of Rosemead, CA	1987-10-01 14:42:20 UTC
6.0	6km SSW of Morongo Valley, CA	1986-07-08 09:20:44 UTC
4.6	3km SSE of Loma Linda, CA	1985-10-02 23:44:12 UTC
4.8	1km SSW of Anza, CA	1982-06-15 23:49:21 UTC
5.3	18km ESE of Anza, CA	1980-02-25 10:47:38 UTC
4.8	3km E of Big Bear Lake, CA	1979-06-30 00:34:11 UTC
4.8	25km NNW of Joshua Tree, CA	1979-03-15 23:07:57 UTC
5.2	23km NNW of Joshua Tree, CA	1979-03-15 21:07:16 UTC
4.8	21km NNW of Joshua Tree, CA	1979-03-15 20:17:50 UTC
4.9	12km ESE of Anza, CA	1975-08-02 00:14:07 UTC
5.2	3km W of Lytle Creek, CA	1970-09-12 14:30:53 UTC
4.7	4km E of Anza, CA	1967-05-21 14:42:34 UTC
5.0	8km NNE of Cabazon, CA	1965-10-17 09:45:18 UTC
4.6	5km NNE of Fontana, CA	1965-04-15 20:08:33 UTC
4.5	29km N of Yucca Valley, CA	1964-01-06 23:47:13 UTC
5.3	6km SSE of Hemet, CA	1963-09-23 14:41:52 UTC
5.0	10km NNW of Big Bear City, CA	1962-10-29 02:42:53 UTC
4.6	3km SSE of Huntington Beach, CA	1961-10-20 19:49:50 UTC
4.5	16km E of Desert Hot Springs, CA	1957-02-01 07:52:14 UTC
4.7	7km ESE of Big Bear City, CA	1956-05-11 16:30:50 UTC
4.7	7km ENE of Big Bear City, CA	1956-03-16 20:29:33 UTC
4.7	14km NE of Trabuco Canyon, CA	1956-01-03 00:25:49 UTC
4.5	10km NW of Borrego Springs, CA	1954-02-12 09:44:27 UTC
4.6	21km SE of Anza, CA	1951-02-15 10:49:57 UTC
4.7	11km SE of Anza, CA	1951-02-15 10:48:00 UTC
4.8	2km SE of Idyllwild, CA	1950-09-05 19:19:56 UTC
4.7	12km NNE of Thousand Palms, CA	1948-12-05 00:07:20 UTC
6.0	16km E of Desert Hot Springs, CA	1948-12-04 23:43:16 UTC
4.6	2km ENE of Lytle Creek, CA	1948-03-01 08:12:13 UTC
4.5	3km N of Desert Hot Springs, CA	1947-07-26 02:49:42 UTC
5.2	10km SSE of Yucca Valley, CA	1947-07-25 06:19:49 UTC
4.8	6km SSE of Yucca Valley, CA	1947-07-25 00:46:30 UTC
4.7	3km ENE of Garnet, CA	1947-07-24 22:54:28 UTC
5.3	4km NNE of Desert Hot Springs, CA	1947-07-24 22:10:46 UTC
4.9	1km NE of Banning, CA	1946-09-28 07:19:10 UTC
5.2	12km WSW of Morongo Valley, CA	1944-06-12 11:16:35 UTC
5.1	10km NNE of Cabazon, CA	1944-06-12 10:45:34 UTC
5.3	6km WNW of Big Bear Lake, CA	1943-08-29 03:45:14 UTC
4.6	14km NNW of Joshua Tree, CA	1942-08-07 01:15:34 UTC
4.6	14km E of Lucerne Valley, CA	1942-02-01 15:18:28 UTC
5.7	5km E of Lomita, CA	1941-11-14 08:41:37 UTC
4.7	2km NNE of Lomita, CA	1941-10-22 06:57:18 UTC
4.6	5km SE of Joshua Tree, CA	1940-06-01 05:27:01 UTC

Magnitude	Location of Event	Date/Time of Event
4.5	11km SSE of Joshua Tree, CA	1940-05-18 06:04:31 UTC
5.2	11km S of Joshua Tree, CA	1940-05-18 05:51:21 UTC
5.3	6km SSE of Joshua Tree, CA	1940-05-18 05:03:59 UTC
4.5	3km ESE of Rolling Hills, CA	1939-12-27 19:28:48 UTC
5.2	8km ENE of Trabuco Canyon, CA	1938-05-31 08:34:55 UTC
4.7	1km WSW of Grand Terrace, CA	1936-02-23 22:20:42 UTC
4.8	5km S of Anza, CA	1935-11-04 03:55:54 UTC
4.6	17km WNW of Morongo Valley, CA	1935-10-24 14:52:22 UTC
4.9	13km WNW of Morongo Valley, CA	1935-10-24 14:48:07 UTC
4.6	0km S of Citrus, CA	1935-07-13 10:54:18 UTC
4.8	2km N of Signal Hill, CA	1933-10-02 09:10:18 UTC
4.7	2km N of Huntington Beach, CA	1933-03-15 11:13:19 UTC
4.8	2km WSW of Huntington Beach, CA	1933-03-14 19:02:36 UTC
4.8	2km SSE of Westminster, CA	1933-03-13 13:18:14 UTC
4.5	4km S of Westminster, CA	1933-03-11 16:54:29 UTC
4.8	3km SSE of Garden Grove, CA	1933-03-11 09:10:58 UTC
4.9	1km NNW of Huntington Beach, CA	1933-03-11 08:55:39 UTC
5.3	7km W of Newport Beach, CA	1933-03-11 06:58:44 UTC
5.0	2km ENE of Westminster, CA	1933-03-11 05:18:47 UTC
4.9	1km WSW of Westminster, CA	1933-03-11 05:11:02 UTC
4.7	2km ENE of Seal Beach, CA	1933-03-11 04:40:00 UTC
4.9	2km ENE of Seal Beach, CA	1933-03-11 04:39:00 UTC
4.8	5km SE of Seal Beach, CA	1933-03-11 03:24:01 UTC
4.7	2km ENE of Seal Beach, CA	1933-03-11 02:29:59 UTC
4.6	2km SW of Santa Ana, CA	1933-03-11 02:17:23 UTC
4.5	2km ENE of Seal Beach, CA	1933-03-11 02:16:00 UTC
4.5	4km WNW of Newport Beach, CA	1933-03-11 02:09:00 UTC
4.7	4km WNW of Newport Beach, CA	1933-03-11 02:04:00 UTC
6.4	7km WNW of Newport Beach, CA	1933-03-11 01:54:09 UTC

Steven Weiss
Planning Director

RIVERSIDE COUNTY
PLANNING DEPARTMENT

May 18, 2016 Pages 3 (including this cover)

Matrix Geotechnical Consulting, Inc.
Email: Chris@matrix-geo.com
Attn: Chris Josef

RE: Review Comments
County Geologic Report No. 2491
"Geotechnical Investigation, Infiltration Study, and Rock Rippability Report for the Proposed Decker Assemblage Industrial Site, Located at the Southwest Corner of Oleander Avenue and Decker Road, Assessor's Parcel Numbers (APN's): 314-040-001, -002, -003, & -008, Western Perris Area, County of Riverside, California," Dated September 30, 2014.

Please see the attached review comments pertaining to the subject report. Prior to approval of this report, all comments must be adequately addressed.

Please call me at (951) 955-6187 if you have any questions.

Sincerely,

RIVERSIDE COUNTY PLANNING DEPARTMENT
Steven Weiss, Planning Director

Daniel P. Walsh

Daniel P. Walsh, CEG No. 2413
Associate Engineering Geologist, TLMA-Planning

Attachments: Review Comments

cc: Planner: Desiree Bowie, Riverside Office Hand Deliver
Eng./Rep.: Glenn Chung (Gchung@deainc.com) Applicant: Trammel Crow. So. Cal Devel Inc. (nholdridge@trammelcrow.com)

File: GEO02491, PM36950

B:\Geology\CGR\GEO02300-2499\geo2491_comments.doc

PARCEL MAP Parcel Map #: PM36950 Parcel: 314-040-008

5. DRT CORRECTIONS REQUIRED

PLANNING DEPARTMENT

5.PLANNING. 1 DRT - GEO02491 REQUIRED

County Geologic Report GEO No. 2491, submitted for the project APNs 314-040-001, -002, -003, and -008, was prepared by Matrix Geotechnical Consulting, Inc. The report is titled; "Geotechnical Investigation, Infiltration Study, and Rock Rippability Report for the Proposed Decker Assemblage Industrial Site, Located at the Southwest Corner of Oleander Avenue and Decker Road, Assessor's Parcel Numbers (APN's): 314-040-001, -002, -003, & -008, Western Perris Area, County of Riverside, California," dated September 30, 2014.
Prior to scheduling this project for public hearing, the following clarification and/or additional information shall be submitted to the County Geologist for review and approval:
1.Please provide a discussion of the regional geologic setting including geologic province description, geomorphology of the project site, and geology of the vicinity.
2.Section 1.2 Location and Site Description (Page 4, 4th paragraph) contains the statement, "Confirmation of this condition is directly related to the upstream surface soil in excavator pit no.___ having highly saturated soil present at a depth of 20 feet." Please provide the missing excavator pit number.
3.Section 2.4 Groundwater states that groundwater was encountered perched on top of the Tonalite. However, Section 2.6.2 Liquefaction & Seismically Induced Settlement states that "Groundwater was not identified below existing grade." Please clarify and revise.
4.Please provide a PGAm and Seismic Design Category for the site.
5.Please provide a list of significant faults within 100 km of the project site, indicate name, magnitude and distance of each individually.
6.Please provide a listing of significant seismic events in the site vicinity (100 km).
7.Provide a reference for the aerial photography used for your analysis of geologic hazards on the site.
8.Appendix C states that consolidation testing was performed and results are presented on Plate C-1, C-2, and C-3; however, these plates are not included in the report copies submitted to the County. Please provide.
9.Please provide a north arrow and base map source for the

PARCEL MAP Parcel Map #: PM36950 Parcel: 314-040-008

5. DRT CORRECTIONS REQUIRED

5.PLANNING. 1 DRT - GEO02491 (cont.) REQUIRED

Geotechnical Map.

It should be noted that no engineering review of this report or formal review of provided building code information are a part of this review. Formal review of engineering design and code data will be made by the County Of Riverside, as appropriate, at the time of grading and/or building permit submittal to the County.