GeoMat Testing Laboratories, Inc.

Soil Engineering, Environmental Engineering, Materials Testing, Geology

April 24, 2020

Project No.: 20112-01

TO: Steven Walker Communities
7111 Indiana Avenue, Suite 300
Riverside, California 92504

SUBJECT: Preliminary Soil Investigation Report, Gas Station and Convenience Store, SEC of Arlington Avenue and Monroe Street, APN: 191-150-031, 032 & 033, Riverside, California

In accordance with your authorization, GeoMat Testing Laboratories, Inc. (GeoMat) is pleased to present our Preliminary Soil Investigation Report for the proposed commercial development at APN: 191-150-031, 032 & 033 at the southeast corner of Arlington Avenue and Monroe Street, in the City of Riverside, California. This report is in fulfillment of our proposal dated April 13, 2020 and your subsequent authorization. The accompanying report presents a summary of our findings, recommendations, and limitation of work for the proposed site development.

The primary purpose of our investigation and report is to provide an evaluation of the existing geotechnical conditions at the site as they relate to the design and construction of the proposed commercial building(s). More specifically, this investigation was primarily to address the following potential geotechnical conditions:

- Site Excavatability
- Temporary Excavation Recommendations
- Earthwork Recommendations
- Shallow Foundation and Slab-on-Grade Recommendations
- Retaining Wall Design Recommendations
- Pavement Recommendations
- Soil Infiltration Testing Rates

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

Submitted for GeoMat Testing Laboratories, Inc.

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1.0 INTRODUCTION

1.1 EXISTING SITE CONDITIONS

The subject site is located on the southeastern corner of Arlington Avenue and Monroe Street, in the City of Riverside, California. The site is bordered to the north and west by Arlington Avenue and Monroe Street, respectively and to the south and east by residential properties. Access onsite can be made from either Arlington Avenue or Monroe Street. Arlington Avenue is a fully developed street but with concrete curb and gutter improvements limited to the eastern half of the property. Monroe Street is a fully developed street with existing concrete curb and gutter. The geographical relationship of the site and surrounding vicinity is shown on the Site Location Map, Figure 1.

The site is roughly rectangular in shape measuring approximately 330 feet long and 170 feet wide. The site is currently vacant, covered mostly in light seasonal grasses and several mature trees. The existing grades onsite are generally level without any abrupt major grade changes. Previously, a structure was located in the center of the property that was removed prior to 2002.

1.2 PROPOSED DEVELOPMENT

We have not been provided with any site plans but we understand that the site is proposed for a commercial development. The development will consist of one to several commercial buildings with associated concrete hardscape, AC pavement drive lanes/parking spaces, and landscaping.

No structural details or foundation plans were provided for our review at the time of this report. We assume that the commercial building will be constructed at or near existing grade (no basements) and supported on shallow concrete foundation with slab-on-grade. Continuous wall loads will not exceed 2 kips per linear foot and isolated column loads of up to 30 kips.

Once the design phase and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Any changes in the design, location or elevation of any structure, as outlined in this report, should be reviewed by this office. GeoMat should be contacted to determine the necessity for review and possible revision of this report.

1.3 FIELD WORK

Three exploratory boreholes were drilled up to 50 feet below ground surface on April 16, 2020 utilizing a CME-75 mobile drill rig equipped with 8-inch diameter hollow stem augers. Refer to Plate 1 for borehole locations.

Relatively undisturbed samples were obtained with the California Ring Sampler (ASTM D 1587). This sampler has three inches external diameter, 2.5-inches inside diameter, and is lined with 1-inch high brass rings, with an inside diameter of 2.41-inches. The sample barrel is driven into the ground at the bottom of the boring with 140-pound hammer with a free fall of approximately 30-inches. Sampler driving resistance, expressed as blows per six inches of penetration, is presented on the boring logs at the respective sampling depths. Ring samples were retained in close-fitting, moisture tight canisters for transport to our laboratory for testing.

Additional representative samples have been recovered with the SPT (Standard Penetration Test, ASTM D 1586) sampler. This sampler consists of steel driving shoe and tube that split longitudinally in half, and a coupling at the top. The coupling connects the sampler to the drill rod. The standard split tube has an inside diameter of 1 3/8-inch (1 1/2-inch inside diameter without liners) and an outside diameter of 2-inches. Unless noted otherwise, liners are usually not used. The standard driving weight and free fall for this test is similar to California Ring Sampler. Blow counts required to drive the samplers 18-inches are recorded on the boring logs. The sum of the number of blows for the last 12-inches on an 18-inch penetration represents the SPT count. This data is shown on the boring logs when obtained in the field.

Bulk samples were also collected from the auger cuttings during drilling. The samples were collected in plastic bags, tied, and tagged for the location and depth. The geotechnical boring logs are presented in Appendix B and may include a description and classification of each stratum, sample locations, blow counts, groundwater conditions encountered during drilling, results from selected types of laboratory tests, and drilling information.

1.4 SUBSURFACE FINDINGS

According to the Geologic Map of the Riverside West/South 1/2 of Fontana Quadrangles (Dibblee Foundation Map DF-128), the subject site is underlain by older alluvial fan deposits (Qoa). Alluvium is weathered bedrock material and sediments that have been eroded from natural slopes and deposited in generally flat lying areas.

Based on our exploratory borings, this alluvial soil generally consists of medium dense to very firm clayey sand (USCS "SC") and sandy lean clay (USCS "CL"). Bedrock was encountered in all three exploratory borings at depths generally between 5 and 10 feet below ground surface. The bedrock is mapped as quartz diorite (qdi) of the peninsular ranges.

1.5 GROUNDWATER CONDITIONS

Groundwater study is not within the scope of this work. Groundwater was encountered in all of our exploratory borings between 15 and 25 feet below existing ground surface. During the infiltration testing, several days later, standing water was observed in the percolation test holes P-1 and P-2 at 7 feet below ground surface.

A contour map showing minimum depths to ground water in the Santa Ana River Valley Region was constructed by the United States Geological Survey (USGS) and subsequently, a report (USGS Map MF-1802) was published in 1985. The map was constructed by contouring the shallowest water level measurements reported to the California Department of Water Resources (CDWR) for the period from 1973-1979. Based on our review of the map, the minimum depth to ground water in the project site area, during this period, was indicated to be around 10 feet below ground surface.

Please note that the potential for rain or irrigation water locally seeping through from elevated areas and showing up near grades cannot be precluded. Our experience indicates that surface or near-surface groundwater conditions can develop in areas where groundwater conditions did not exist prior to site development, especially in areas where a substantial increase in surface water infiltration results from landscape irrigation. Fluctuations in perched water elevations are likely to occur in the future due to variations in precipitation, temperature, consumptive uses, and other factors including mounding of perched water over bedrock or natural soil. Mitigation for nuisance shallow seeps moving from elevated lower areas will be needed if encountered. These mitigations may include subdrains, horizontal drains, toe drains, french drains, heel drains or other devices.

1.6 LABORATORY TESTING

Laboratory tests were performed on selected soil samples collected throughout our subsurface investigation. The tests consisted primarily of moisture content, dry density, sieve analysis, direct shear, hydrocollapse, expansion potential, and water soluble sulfate content. The soil classifications are in conformance with the Unified Soil Classifications System (USCS), as outlined in the Classification and Symbols Chart (Appendix B). A summary of our laboratory testing, ASTM designation, and graphical presentation of test results is presented in Appendix C.

2.0 GEOTECHNICAL HAZARDS

We have reviewed the potential geotechnical hazards at the subject site and have summarized our findings in the following table.

SUMMARY OF POTANTIAL GEOTECHNICAL HAZARDS

HAZARD	ONSITE?	COMMENT
Regional and Local Faults	No	There are no mapped active or potentially active faults with surface expression that trend through or are adjacent to the subject property based on the references cited. The site does not lie within a designated Alquist-Priolo Earthquake Fault Zone (CDMG, 2000).
Seismic Ground Shaking	Yes	The subject site, as is the case with most of the tectonically-active Southern California area, will be periodically subject to moderate to intense earthquake-induced ground shaking from nearby faults. Considerable damage can occur to the site and structural improvements during a strong seismic event. Neither the location nor magnitude of earthquakes can accurately be predicted at this time.
Liquefaction, lateral spreading, and Seismic Settlement*	Yes	According to the City of Riverside General Plan, the site is located in an area considered to be susceptible to liquefaction. Therefore, the potential for liquefaction and dynamic settlement has been evaluated as outlined in Chapter 6 of the California Division of Mines and Geology (DMC) Special Publication 117 ("Guidelines for Evaluation and Mitigation of Seismic Hazards in California") and "Recommended Procedures for Implementation of DMG Special Publication 117 - Guidelines for Analyzing and Mitigating Liquefaction in California", published by the Southern California Earthquake Center, 2008 edition. The design and construction recommendations presented in this report include results of liquefaction and dynamic settlement evaluation. The analysis results are included in Appendix E.
Slope Stability	No	There are no slopes on the subject site or adjacent properties and no slopes are proposed.
Expansive Soil	No	Based on laboratory testing, the upper foundation soil onsite is expected to have a very low expansion potential (EI<20), as defined in ASTM D4829. This would require verification subsequent to completion of new footing excavations.
Collapsible Soil	No	A soil sample, representing the upper ten feet of native soil, was tested in the laboratory for collapse potential. Test results indicate that less than 1% of hydro-collapse occurred in the tested samples. Therefore, the severity of hydrocollapse potential onsite is considered "No Problem" based on NAVFAC DM7.01, see Appendix C for Results.
Corrosive Soil	No	To preliminarily assess the sulfate exposure of concrete in contact with the site soils, a representative soil sample was tested for water-soluble sulfate content. The test results suggest the site soils have a negligible potential for sulfate attack (0.0165 percent) based on commonly accepted criteria. We recommend following the procedures provided in ACI 318-19, Section 19.3, Table 19.3.2.1 for exposure "S0". We recommend Type II cement for all concrete work in contact with soil.
Caving Potential	No	The onsite soil is composed of older alluvial soil and relatively shallow bedrock which is expected to have a low potential for caving.

*The analysis indicates that total dynamic settlement is negligible (0.01 inches) during a large earthquake episode. An estimated dynamic differential settlement of 0.01-inch may be anticipated. The historical high ground water during a seismic event has been assumed at ground surface.

Based on SCEC (1999) guidelines, a potential for loss of bearing capacity due to liquefaction is not expected at the site since there is not an upper potentially liquefiable layer at a depth shallower than the estimated depth where the induced vertical stress in the soil is 10% of the bearing pressure imposed by the proposed foundation systems.

In significant conformance with Youd, Hanson, and Bartlett (ASCE Geotechnical Jr. April 1995, and Lecture by Youd on July 7, 1999), no lateral spreading due to liquefaction is expected at this site due to the following reasons:

- Alluvial subsurface soils are essentially horizontally layered.
- There is not a free-face, onsite, toward which liquefied soils could move laterally.
- No saturated liquefiable sand with values of N1(60) <15 exist at the site.

If loose clean sand exists between sampling intervals, their occurrence is expected to be thin and considered to be scattered or have minimal occurrence throughout the site, and cannot reasonably be connected to form a hypothetical "continuous" line of significant length that could reasonably be expected to "exit" on a slope or a free-face, or move significantly below the gentle slope of the site.

Although it is extremely difficult to predict the overall behavior of any site during seismic shaking, it is our opinion that proper design of foundation can substantially improve the structure's resistance to deformation. This is most commonly accomplished by providing adequate lateral connections between all footings with reinforced grade beams and strengthened stem walls. If the owner wishes a higher degree of confidence, then the structures should be designed for higher probable events.

Please note that foundation design is under the purview of the structural engineer. All foundations should be designed by a qualified structural engineer in accordance with the CBC and the latest applicable building codes and structural considerations may govern.

2.1 SOIL SITE CLASS

The subsurface soil profile at the site can be classified from a seismic standpoint based on the conditions encountered in our exploratory borings, and anticipated within the upper 100 feet of the site based on geologic mapping, as being "very dense soil and soft rock" with an average undrained shear strength (s_u) of more than 2,000 psf and SPT N-values of more than 50 blows per foot. Based on the soils encountered in the borings drilled within the subject site and with consideration of the geologic units mapped in the area, it is our opinion that the site soil profile corresponds to Site Class C in accordance with Section 1613.2.2 of the California Building Code (CBC 2019) and Chapter 20 of ASCE/SEI 7-16.

2.2 SEISMIC DESIGN PARAMETERS:

We have determined the seismic design parameters in accordance with the provisions of the current California Building Code (CBC, 2019) and ASCE/SEI 7-16 Standard using the Structural Engineers Association of California, OSHPD Seismic Design Maps Web Application (https://seismicmaps.org). The CBC Site Class was determined to be Site Class "C" based on the results of our explorations and a review of the local soil and geologic conditions. The mapped seismic parameters are presented in Appendix D of this report.

The 2019 CBC requires that Site Class F be assigned to any profile containing soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils. For Site Class F, the site coefficients are to be determined in accordance with Section 11.4.8 of ASCE/SEI 7-16. However, Section 20.3.1 of ASCE/SEI 7-16 indicates that for sites with structures having a fundamental period of vibration equal to or less than 0.5 seconds, the site coefficient factors (F_a and F_v) may be determined using the standard procedures.

The seismic design parameters provided in this report were calculated using the site coefficient factors for Site Class C, assuming that the fundamental periods of the structures are less than 0.5 seconds. However, the results of the liquefaction evaluation indicate that the subject site is underlain by potentially liquefiable soils. If the proposed structure has a fundamental period greater than 0.5 seconds, a site specific seismic hazards analysis will be required and additional subsurface exploration will be necessary.

2.3 GEOTECHNICAL CONCLUSION

Based on the findings summarized in this report, it is our professional opinion that the proposed development will not be subject to a hazard from settlement, slippage, or landslide, provided the recommendations of this report are incorporated into the proposed development. It is also our opinion that the proposed development will not adversely affect the geologic stability of the site or adjacent properties provided the recommendations contained in this report are incorporated into the proposed construction.

3.0 TENTATIVE RECOMMENDATIONS

3.1 SITE PREPARATION

All grading should be performed in accordance with our General Earthwork and Grading Specifications presented in Appendix F except as modified within the text of this report.

3.1.1 Clearing, Grubbing, & Fill Removal

All debris, undocumented fill, abandoned utility lines, roots, irrigation appurtenances, underground structures, deleterious materials, etc., should be removed and hauled offsite. Cavities created during site clearance should be backfilled in a controlled manner.

3.1.2 Overexcavation & Subgrade Preparation

Overexcavation and recompaction is recommended to replace the existing surficial soils and to reduce the potential for excessive settlement of the onsite soils. Care should be taken during overexcavation operations to maintain sidewall stability and personal safety.

The recommended overexcavations described below may be extended deeper if loose soil is encountered in the bottom of the overexcavation. Once the bottom of the excavation is observed by a representative of this office to be in competent native soil, the bottom of the overexcavation should be scarified, moisture conditioned, and recompacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557 Test Method; prior to placement of fill. Deeper overexcavations, especially to remove loose soils or deleterious material, may be required depending upon field observations of excavation bottom by the soil engineer or his representative.

3.1.2.1 Building Pad

Subsequent to site clearance, the proposed building pad should be overexcavated to a depth of at least 5 feet below existing grade or 3 feet below the proposed footings, whichever is greater. The lateral extent of the overexcavations should be the greater of 5 feet beyond building lines or equal to the depth of fill, where possible.

3.1.2.2 Retaining and Screening Walls

If proposed, screening and retaining walls should be overexcavated to a depth of 2 feet below foundation bearing grade and replaced as compacted structural. Any undocumented fill soils or disturbed native alluvium within any of these foundation areas should be removed in their entirety. The overexcavated subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning, and recompacting. The previously excavated soils may then be replaced as compacted structural fill. The subgrade preparation should extend beyond the proposed improvements a horizontal distance of at least 2 feet, where achievable.

3.1.2.3 Pavement and Hardscape Subgrade

Subsequent to site clearance, pavement and hardscape areas should be overexcavated to a depth of at least 2 feet below proposed finish grade or existing grade, whichever is greater. The subgrade preparation should extend beyond the proposed improvements a horizontal distance of at least 2 feet, where achievable. The excavation bottom should be scarified 8 inches, moisture conditioned to near optimum moisture content and be recompacted to at least 90% relative compaction. Engineered fill should then be moisture conditioned, placed in suitable lifts, and compacted to a minimum of 90% relative compaction. Aggregate base should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557

3.1.3 Compacted Fills/Imported Soils

Any soil to be placed as fill, whether presently onsite or import, should be approved by the soil engineer or his representative prior to their placement. All onsite soils to be used as fill should be cleansed of any roots, or other deleterious materials. Rocks larger than 8-inches in diameter should be removed from soil to be used as compacted fill.

All fills should be placed in 6- to 8-inch loose lifts, thoroughly watered, or aerated to near optimum moisture content, mixed and compacted to at least 90 or 95 percent relative compaction depending on the material (subgrade soil or aggregate base) and application (pavement subgrade, building pad, etc.). This is relative to the maximum dry density determined by ASTM D1557 Test Method.

Any imported soils should be sandy (preferably USCS "SM" or "SW", and very low in expansion potential) and approved by the soil engineer. The soil engineer or his representative should observe the placement of all fill and take sufficient tests to verify the moisture content and the uniformity and degree of compaction obtained. should be designated to allow for variations in the indicated shrinkage and subsidence estimates.

3.1.4 Trench Backfill

All utility trenches and retaining wall backfills should be mechanically compacted to the minimum requirements of at least 90 percent relative compaction. Onsite soils derived from trench excavations can be used as trench backfill except for deleterious materials. Soils with sand equivalent greater than 30 may be utilized for pipe bedding and shading. Pipe bedding should be required to provide uniform support for piping. Excavated material from footing trenches should not be placed in slab-on-grade areas unless properly compacted and tested.

3.2 TEMPORARY EXCAVATIONS

All excavations must comply with applicable local, state, and federal safety regulations including the current OSHA Excavation and Trench Safety Standards. Construction site safety generally is the sole responsibility of the Contractor, who should also be solely responsible for the means, methods, and sequencing of construction operations.

3.2.1 Excavation Characteristics

The onsite soil is generally composed of dense to hard clayey sand and sandy lean clay which may exhibit difficult excavating resistance for smaller excavating equipment such as rubber-tired backhoes.

3.2.2 Safe Vertical Cut

All applicable requirements of the California Construction and general Industry Safety Orders, the Occupational Safety and Health Act, and current amendments, and the Construction safety Act should be met. Cuts should be observed during excavation by the project's geotechnical consultant. If potentially unstable soil conditions are encountered, modifications of slope ratios for temporary cuts may be required.

Temporary un-surcharged excavations of 4 feet high may be made at a vertical gradient for short periods of time. Temporary un-surcharged excavations greater than 4 feet may be trimmed back at 1H:1V gradients.

The exposed vertical cut conditions should be verified by the project geotechnical engineer during construction. No excavations should take place without the direct supervision of the project geotechnical engineer.

3.2.3 Excavation Setbacks

No excavations should be conducted, without special considerations, along property lines, public right-of-ways, or existing foundations, where the excavation depth will encroach within the "zone of influence". The "zone of influence" of the existing footings, property lines, or public right-of-way may be assumed to be below a 45-degree line projected down from the bottom edge of the footing, property line, or right-of-way.

3.2.4 Precaution for Excavations

The Contractor should be aware that unsupported excavation depths should in no case exceed those specified in local, state, and/or federal safety regulations (e.g., OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations).

Such regulations are strictly enforced and, if they are not followed, the Owner, Contractor, and/or earthwork and utility subcontractors could be liable for substantial penalties. The contractor's "responsible person", as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures.

3.3 FOUNDATION RECOMMENDATIONS

The proposed building may be supported on conventional shallow foundation systems deriving support in compacted fill. All foundation excavations must be observed and approved by the Geotechnical Engineer's representative, prior to placing steel reinforcement or concrete.

3.3.1 Bearing Value

Spread, continuous, or pad-type foundations carried at least 18-inches below the lowest adjacent grade may be designed to impose a net dead-plus-live load pressure of 2,500 psf. The bearing capacity may be increased 15 percent for every additional foot of embedment, to a maximum allowable bearing pressure of 3,000 psf. A one-third increase may be used for wind or seismic loads.

3.3.2 Lateral Resistance

Resistance to lateral footing will be provided by passive earth pressure and base friction. For footings bearing against firm native material, passive earth pressure may be considered to be developed at a rate of 260 psf per foot of depth to a maximum of 2,000 psf. Base friction may be computed at 0.38 times the normal load. If passive earth pressure and friction are combined to provide required resistance to lateral forces, the value of the passive pressure should be reduced to two-thirds the value.

3.3.3 Total Settlement

The onsite soils below the foundation depth have relatively high strengths and will not be subject to significant stress increases from foundations of a new structure. Therefore, estimated total long-term static and seismic settlement between similarly loaded adjacent foundation systems should not exceed 1-inch. The structures should be designed to tolerate a differential settlement on the order of 1/2-inch over a 30-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch.

3.3.4 Reinforcement

Footing reinforcement should be determined by the structural engineer; however, minimum reinforcement should be at least two No. 5 reinforcing bars, top and bottom. Reinforcement and size recommendations presented in this report are considered the minimum necessary for the soil conditions present at the foundation level and are not intended to supersede the design of the project structural engineer or criteria of the governing agencies for the project.

3.4 SLABS-ON-GRADE

Office slabs should be at least 4-inches thick. Storage slabs and slabs subject to traffic should be at least 6-inches thick. The structural engineer should design the actual slab thickness and reinforcement based on structural load requirements.

3.4.1 Modulus of Subgrade Reaction

A coefficient of vertical subgrade reaction (K_V) of 120 psi/in may be assumed for the building pad compacted fill soils. The modulus of subgrade reaction was estimated based on the NAVFAC 7.1 design charts. This value is for a small loaded area (1 sq. ft or less) such as for wheel loads or point loads and should be adjusted for larger loaded areas, as necessary.

3.4.2 Capillary Break & Moisture Barrier

If moisture sensitive floor coverings are proposed, we recommend that the floor slab in those areas be underlain by a minimum 10-mil visqueen moisture barrier, or equivalent, over a 2-inch thick capillary break layer of clean sand. The 2-inch thick layer of sand should be placed between the subgrade soil and the membrane to decrease the possibility of damage to the membrane. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications.

3.4.3 Slab Curling

A low-slump concrete should be used to minimize possible curling of the slab. A 2-inch thick layer of sand may be placed over the vapor retarding membrane to reduce slab curling. If this sand bedding is used, care should be taken during the placement of the concrete to prevent displacement of the sand. The concrete slab should be allowed to cure properly before placing vinyl or other moisture-sensitive floor covering.

However, 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.

3.4.4 Subgrade Exposure

Construction activities and exposure to the environment can cause deterioration of the prepared subgrade. Therefore, we recommend that our field representative observe the condition of the final subgrade soils immediately prior to slab-on-grade construction, and, if necessary, perform further density and moisture content tests to determine the suitability of the final prepared subgrade.

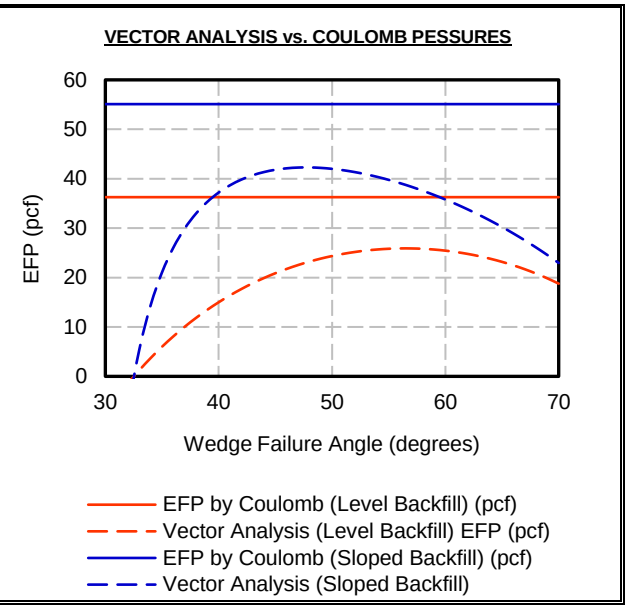
Additionally, the slab subgrade should be moisture conditioned to 2 to 4 percent above the optimum moisture content, to a depth of 12 inches. The moisture content of the floor slab subgrade soils should be verified by the geotechnical engineer within 24 hours prior to placing the vapor retarding membrane or concrete.

3.5 RETAINING WALLS

The following lateral earth pressures, in conjunction with the lateral resistance parameters provided in the Foundation Recommendations section of this report, may be used for the design of retaining walls with free draining compacted backfills. If passive earth pressure and friction are combined to provide required resistance to lateral forces, the value of the passive pressure should be reduced to two-thirds the following recommendations.

Earth Pressure Condition	Backfill Slope	Equivalent Fluid Pressure (pcf)	Earth Pressure Coefficient
Active Case (Fully Drained)	Level	36	K_a = 0.30
	2H:1V	55	K_a = 0.46
At-Rest Case (Fully Drained)	Level	56	K_o = 0.46
	2H:1V	81	K_o = 0.67
Total Unit Weight of Soil		120 pcf	

The lateral earth pressures presented in the table above were determined by comparing the calculated earth pressures utilizing the classical methods presented by Coulomb (active case) and Jaky (at-rest case) with the free-body diagram method (vector analysis) and the greater value is the governing design value.



All retaining walls and block walls footings should be founded in compacted fill or firm native soils. We recommend drainage for retaining walls to be provided in accordance with Plate 2 of this report. Maximum precautions should be taken when placing drainage materials and during backfilling. All wall backfills should be properly compacted to at least 90 percent relative compaction. We also recommend that a waterproofing specialist provide any recommendations and design of a waterproofing system.

3.6 CONVENTIONAL PAVEMENT RECOMMENDATIONS

3.6.1 Subgrade Uniformity

The proposed pavement subgrade should be prepared in accordance with the Overexcavation & Subgrade Preparation section of this report. The subgrade for pavement support must be firm, unyielding, and uniform with no abrupt horizontal changes in degree of support. The subgrade soil should be uniform materials and density. Soft spots, if encountered, should be excavated and recompacted with the same type of soil as found in adjacent subgrade.

3.6.2 Aggregate Base

The aggregate base should conform to Caltrans Class 2 Aggregate Base or the Standard Specifications for Public Works for Crushed Miscellaneous Base, should be firm and unyielding, and without pumping conditions prior to placement of pavement. Aggregate base should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557.

3.6.3 Flexible Pavement Design

The following recommended pavement section is based on the following assumed Traffic Index and R-value. The minimum recommended asphalt concrete (AC) pavement thickness is as follows:

Location	Traffic Index (Assumed)	R-Value (Assumed)	Minimum Recommended Pavement Section	
			AC	AB
Auto Parking	5	10	3.0”	9.0”
Heavy Truck Traffic	7	10	4.0”	14.5”

AC: Asphalt Concrete, AB: Aggregate Base,

Final pavement design recommendations should be based on laboratory test results of representative pavement subgrade soils upon the completion of rough grading.

3.6.4 Portland Cement Concrete

For interior private drives, 8-inches minimum concrete over compacted native subgrade is recommended. Pavement subgrade should be saturated to a depth of 12-inches and compacted to at least 95 percent relative compaction. Saturated subgrade should be tested for moisture by the soil engineer.

Concrete pavement should be air entrained Portland Cement Concrete Pavement and must have a minimum 28-day flexural strength of 450 psi (compressive strength of approximately 3500 psi).

No reinforcing is necessary. Joint design and spacing should be in accordance with ACI recommendations. Construction joints should contain dowels or be tongue and grooved to provide load transfer. Tie bars are recommended on the joints adjacent to unsupported edges. Maximum joint spacing in feet should not exceed 2 to 3 times the thickness in inches. Joint sealing with a quality silicone sealer is recommended to prevent water from entering the subgrade allowing pumping and loss of support.

Proper subgrade preparation and joint sealing will reduce (but not eliminate) the potential for slab movements (thus cracking) on native soils. Frequent jointing will reduce uncontrolled cracking and increase the efficiency of aggregate interlock joint transfer.

3.7 PERVIOUS PAVEMENTS (OPTIONAL)

As an alternative option for managing stormwater run-off, the onsite system may also utilize permeable pavement options. The type of pavement should be selected based on a review of the pavement specifications and properties and designed according to the product manufacturer's recommendations. The permeable pavement should be designed in accordance with the Riverside County Design Handbook for Low Impact Development Best Management Practices (http://rcflood.org/npdes/LIDBMP.aspx). Refer to Plate 3 for a typical permeable pavement detail.

3.7.1 Pervious Cement Concrete Pavement

In order to provide porous concrete pavement recommendation, we started by estimating the conventional concrete pavement thickness. Once the conventional concrete thickness is known, we add 1.5 inches to the conventional thickness to obtain the porous pavement thickness for a Modulus of Rupture of 375 psi.

3.7.1.1 Storing Stormwater for Evaporation

This pavement option is for storing storm water in the pavement voids until evaporation. Thicker concrete may be utilized to ensure sufficient void space to accommodate the volume of water generated during a storm event.

	Porous Concrete Thickness	Impermeable Liner	Aggregate Base	Subgrade
Option I	9.5" (min)	15 mil (min)	--	95% Compacted
Option II	9.5" (min)	--	4" Class II	90% Compacted

3.7.1.2 Storing Stormwater for Infiltration

This option is for storing storm water in the gravel reservoir voids until water infiltrates into the soil. The recommend thickness of porous concrete is at least 5.5 inches. We recommend to install a reservoir layer consists of stone underneath the structural pavement. The total thickness of the reservoir layer is determined by runoff storage needs and the infiltration rate of in situ soils. A 12" maximum reservoir layer consisting of AASHTO #57 washed gravel vibrated in place or equivalent with a minimum of 40% void ratio should be utilized.

The reservoir layer should be lightly compacted to a firm unyielding condition. Over-compaction of the reservoir layer can result in reduced permeability. The prepared reservoir layer should be observed by an engineer to ensure that it has been adequately compacted prior to placement of the permeable pavement section.

If treatment of the collected stormwater is necessary before infiltration, a minimum 6-inch thick filter layer of sand or permeable treatment media could be included in the pavement section and located below the permeable reservoir.

3.7.2 Porous Asphalt Concrete Pavement

This option is for storing storm water in the reservoir voids below asphalt concrete porous pavement until water infiltrates into the soil. We recommend the porous asphalt to be at least 4 inches thick underlain with a reservoir layer. A 12" maximum reservoir layer consisting of AASHTO #57 washed gravel vibrated in place or equivalent with a minimum of 40% void ratio may be utilized. The total thickness of the reservoir layer is determined by runoff storage needs and the infiltration rate of in situ soils.

The reservoir layer should be lightly compacted to a firm unyielding condition. Over-compaction of the reservoir layer can result in reduced permeability. The prepared reservoir layer should be observed by an engineer to ensure that it has been adequately compacted prior to placement of the permeable pavement section.

If treatment of the collected stormwater is necessary before infiltration, a minimum 6-inch thick filter layer of sand or permeable treatment media could be included in the pavement section and located below the permeable reservoir.

3.7.3 Maintenance

All porous pavement should be inspected twice a month for three months after construction, and annually thereafter. Inspections should be conducted after large storms to check for ponding that might indicate local clogging. In addition, it is also important to frequently inspect pavement for debris, mud, and vegetation growth. The pavement should be designed to flow in case of run off producing event.

3.8 STORMWATER INFILTRATION

Infiltration testing was conducted utilizing the shallow percolation test method at depths of 2, 3 and 9 feet below existing ground surface. The infiltration testing was performed in general accordance with the guidelines published in the Riverside County Design Handbook for Low Impact Development Best Management Practices, Infiltration Testing Guidelines. The following table summarizes the result of the infiltration feasibility study. Refer to Appendix G for field infiltration test data.

Test No.	Test Depth Below Ground Surface	Adjusted Infiltration Rate (in/hr)
P-1	36"	3.25
P-2	24"	2.40
P-3	108"	0.05
P-4	108"	0.28

The raw percolation rate is the rate of water infiltration in the horizontal and vertical direction. This percolation rate is adjusted using the "Porchet Method" to obtain the adjusted water infiltration rate in the vertical direction only.

Long-term infiltration rates may be reduced significantly by factors such as soil variability and inaccuracy in the infiltration rate measurement. The correction factor for site variability is between 3 and 10. Safety factors for operating the system, maintenance, siltation, biofouling, etc. should also be considered by the design civil engineer at his discretion. Minimum safety factor required by the County of Riverside for tests conducted when deep exploratory borehole has been drilled at the site is 3.

The soil infiltration results were inconsistent with soil classification. Additionally, groundwater elevation varied significantly several days after drilling. Infiltration in older alluvium (soil above granitic material) is expected to be much slower that the results obtained. Infiltration is expected to be good in weathered granitic material estimated to be the upper four to five feet. Caution should be exercised when encroaching groundwater levels. The depth to groundwater levels will vary depending on a variety of conditions including season, irrigation activities and rainfall events. Groundwater mounding (localized rising of groundwater level due to concentration of stormwater infiltration) may be estimated once the location and size of infiltration facility is known.

The infiltration system must be located such that the closest distance between an adjacent foundation is at least 10 feet in all directions from the zone of saturation. The zone of saturation may be assumed to project downward from the discharge of the infiltration facility at a gradient of 1H:1V. Additional property line or foundation setbacks may be required by the governing jurisdiction and should be incorporated into the stormwater infiltration system design as necessary.

However, due to the relatively shallow groundwater at the subject site (7 feet below ground surface) an alternative isolated system located within the landscape areas may be utilized. Isolation should consist of a barrier, such as a planter box, that prevents water from passing through to the site soils. The filtration system may consist of soil media underlain by an underdrain system with a geotextile fabric. Refer to Plate 4 for a typical stormwater planter detail.

3.9 SITE DRAINAGE

Positive drainage should be provided and maintained for the life of the project around the perimeter of all structures (including slopes and retaining walls) and all foundations toward streets or approved drainage devices to minimize water infiltrating into the underlying natural and engineered fill soils. In addition, finish subgrade adjacent to exterior footings should be sloped down (at least 2%) and away to facilitate surface drainage. Perimeter water collection devices may be installed around the structure to collect roof/irrigation/natural drainage. Roof drainage should be collected and directed away from foundations via nonerosive devices. Over the slope drainage must not be permitted.

Water, either natural or by irrigation, should not be permitted to pond or saturate the foundation soils. Planter areas and large trees adjacent to the foundations are not recommended. All planters and terraces should be provided with drainage devices. Internal drainage should be directed to approved drainage collection devices.

Location of drainage device should be in accordance with the design civil engineer's drainage and erosion control recommendations. The owner should be made aware of the potential problems, which may develop when drainage is altered through construction of retaining walls, patios and other devices. Ponded water, leaking irrigation systems, over watering or other conditions which could lead to ground saturation should be avoided. Surface and subsurface runoff from adjacent properties should be controlled. Area drainage collection should be directed through approved drainage devices. All drainage devices should be properly maintained.

4.0 ADDITIONAL SERVICES

Plan Reviews

The recommendations provided in this report are based on preliminary information and subsurface conditions as interpreted from limited exploratory boreholes at the site. We should be retained to review final grading and foundation plans to revise our conclusions and recommendations, as necessary. Professional fees will apply for each review.

Our conclusions and recommendations should also be reviewed and verified during site grading and revised accordingly if exposed geotechnical conditions vary from our preliminary findings and interpretations.

Additional Observation and/or Testing

GeoMat Testing Laboratories, Inc. should observe and/or test at the following stages of construction.

- During overexcavation and compaction of subgrade soil
- Following footing excavation and prior to placement of footing materials.
- During wetting of slab subgrade and prior to placement of slab materials.
- During all trench and wall backfill.
- When any unusual conditions are encountered.

Final Report of Compaction During Grading

A final report of compaction control should be prepared subsequent to the completion of grading. The report should include a summary of work performed, laboratory test results, and the results and locations of field density tests performed during grading.

5.0 GEOTECHNICAL RISK

The concept of risk is an important aspect of the geotechnical evaluation. The primary reason for this is that the analytical methods used to develop geotechnical recommendations do not comprise an exact science. The analytical tools which geotechnical engineers use are generally empirical and must be used in conjunction with engineering judgment and experience. Therefore, the solutions and recommendations presented in the geotechnical evaluation should not be considered risk-free and, more importantly, are not a guarantee that the interaction between the soils and the proposed structure will perform as planned.

The engineering recommendations presented in the preceding sections constitute GeoMat Testing Laboratories professional estimate of those measures that are necessary for the proposed development to perform according to the proposed design based on the information generated and referenced during this evaluation, and GeoMat Testing Laboratories experience in working with these conditions.

6.0 LIMITATION OF INVESTIGATION

This report was prepared for the exclusive use on the new construction. The use by others, or for the purposes other than intended, is at the user's sole risk.

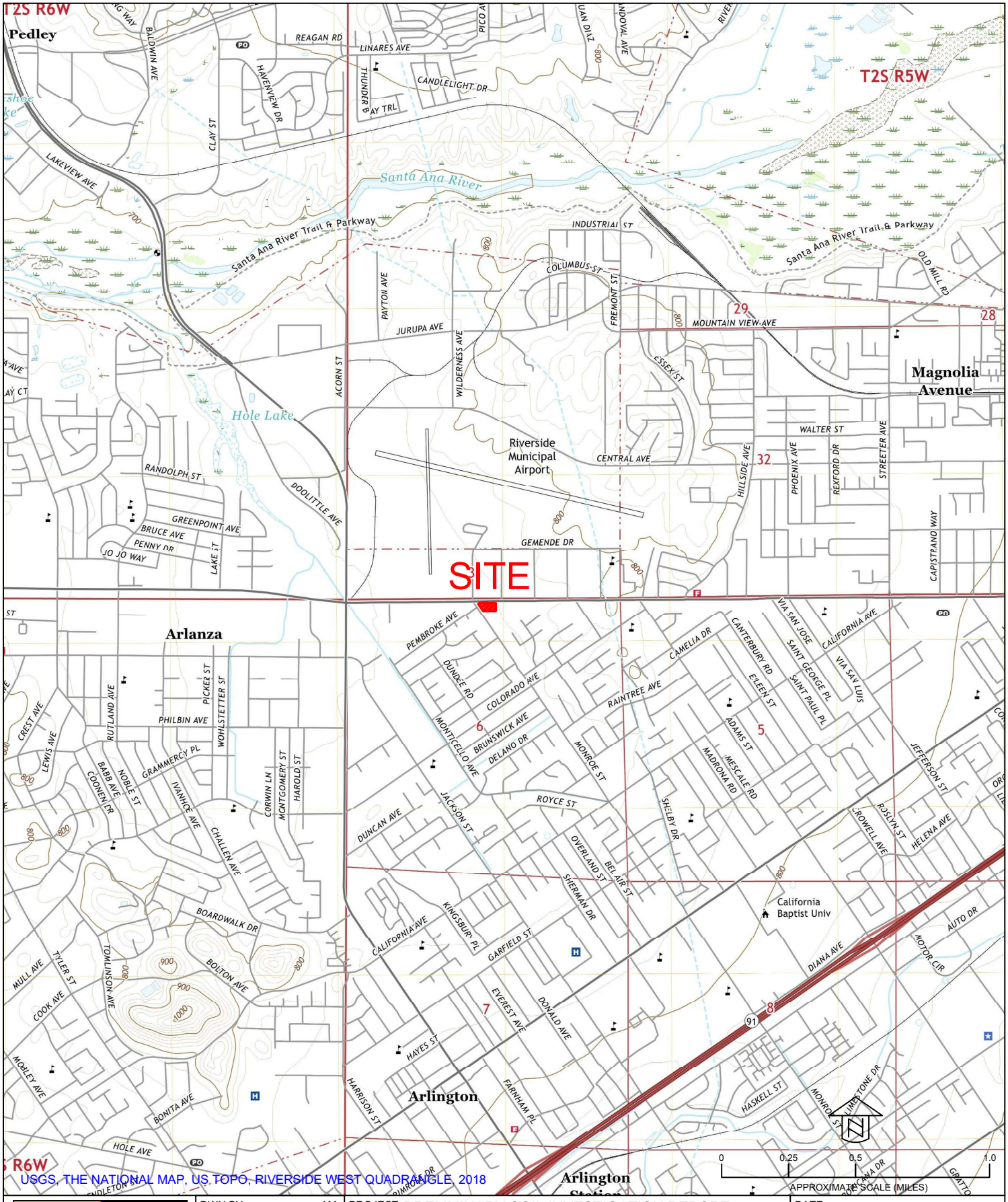
Our investigation was performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable Geotechnical Engineers practicing in this or similar locations within the limitations of scope, schedule, and budget. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

The field and laboratory test data are believed representative of the site; however, soil conditions can vary significantly. As in most projects, conditions revealed during construction may be at variance with preliminary findings. If this condition occurs, the possible variations must be evaluated by the Project Geotechnical Engineer and adjusted as required or alternate design recommended.

This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the information and recommendations contained herein are brought to the attention of the engineer for the addition and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractor carry out such recommendations in the field.

This firm does not practice or consult in the field of safety engineering. We do not direct the contractor's operations, and we cannot be responsible for other than our own personnel on the site; therefore, the safety of others is the responsibility of the contractor. The contractor should notify the owner if he considers any of the recommended actions presented herein to be unsafe.

The findings, conclusions, and recommendations presented herein are based on our understanding of the addition and on subsurface conditions observed during our site work, and are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge.



geomat GeoMat Testing Laboratories, Inc. 9980 Indiana Avenue, Suite 14 Riverside, California 92503	DWN BY: AM CHK'D BY: MN DATUM: -- PROJECTION: -- SCALE: 1" = 1/2 MILE REV. NO.: --	PROJECT: PRELIMINARY SOIL INVESTIGATION REPORT APN: 191-150-031, 032 & 033 RIVERSIDE, CALIFORNIA TITLE: SITE LOCATION MAP	DATE: APRIL 2020 PROJECT NO.: 20112-01 FIGURE NO.: Figure 1

P-03

P-04

B-01

B-02

B-03

P-02

P-01

INITIAL PERCOLATION TEST HOLES CANCELLED (STANDING WATER AT 7' BELOW GROUND SURFACE)

LEGEND:

B-03 EXPLORATORY BORING

P-04 INFILTRATION TEST

ALL LOCATIONS ARE APPROXIMATE

BASE MAP:
GOOGLE EARTH

EXPLORATORY BOREHOLE LOCATION MAP

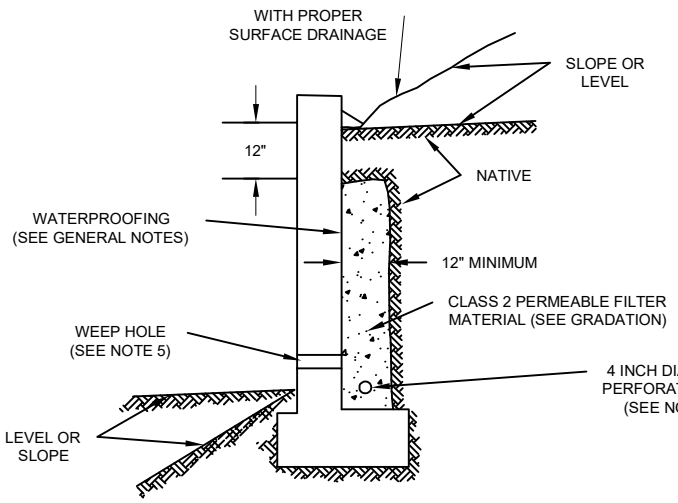
APN: 191-150-031, 032 & 033
RIVERSIDE, CALIFORNIA

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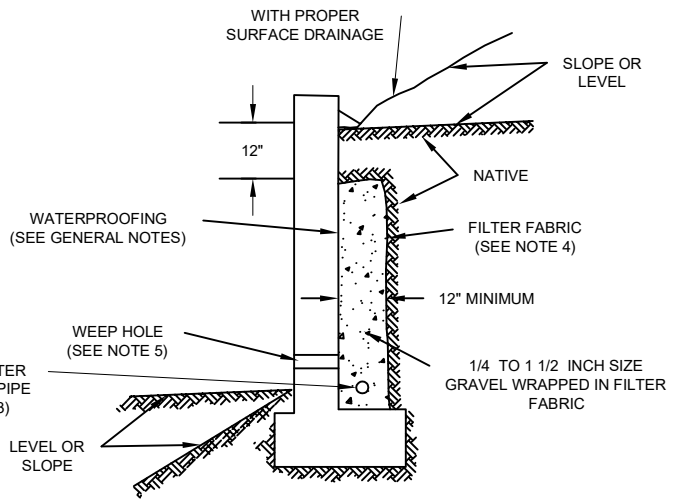
DRAWN BY: AM	DATE: APRIL 2020	PLATE 1
SCALE: 1" = 40' (APPROXIMATE, 11"X17" SHEET)	PROJECT NO.: 20112-01	

SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX ≤ 50

OPTION 1: PIPE SURROUNDED WITH CLASS 2 PERMEABLE MATERIAL



OPTION 2: GRAVEL WRAPPED IN FILTER FABRIC



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

GENERAL NOTES:

*Waterproofing should be provided where moisture nuisance problem through the wall is undesireable.
*Water proofing of the walls is not under the purview of the geotechnical engineer.
*All drains should have a gradient of 1 percent minimum.
*Outlet portion of the subdrain should have a 4-inch diamater solid pipe discharged into a suitable disposal area designed by the project engineer. The subdrain pipe should be accessible for maintenance (rodding).
*Other subdrain backfill options are subject to the review by the geotechnical engineer and modification of design parameters.

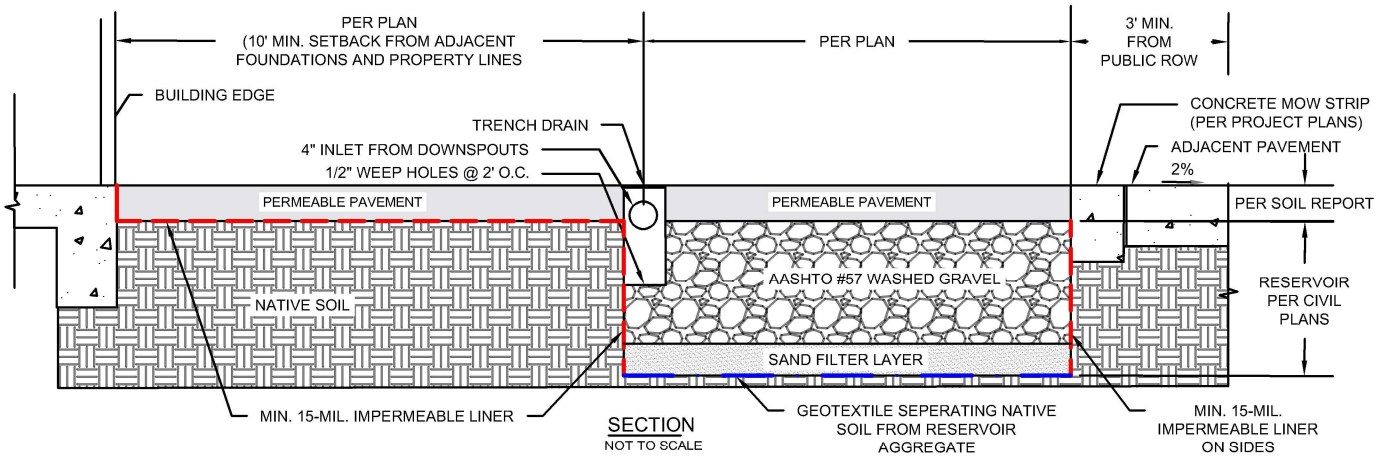
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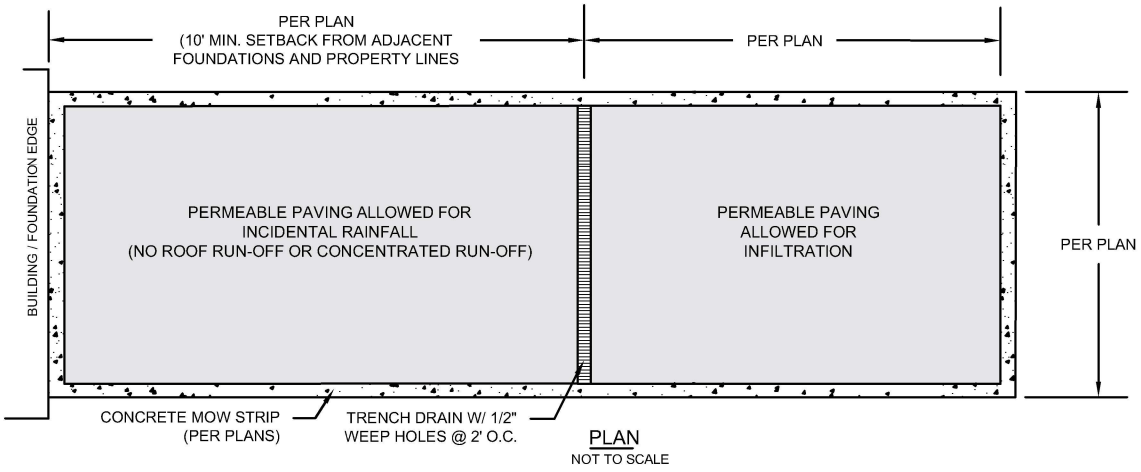
1) Sand should have a sand equivalent of 30 or greater and may be densified by water jetting.
2) 1 Cu. ft. per ft. of 1/4 - to 1 1/2 -inch size gravel wrapped in filter fabric
3) Pipe type should be ASTM D1527 Acrylonitrile Butadiene Styrene (ABS) SDR35 or ASTM D1785 Polyvinyl Chlorise plastic (PVC), Schedule 40, Armco A2000 PVC, or approved equivalent. Pipe should be installed with perforations down. Perforations should be 3/8 -inch in diameter placed at the ends of a 120-degree arc in two rows at 3-inch on center (staggered).
4) Filter Fabric should be Mirafi 140NC or approved equivalent.
5) Weephole should be 3-inch minimum diameter and provided at 10-foot maximum intervals. if exposure is permitted, weepholes should be located 12-inches above finished grade. If exposure is not permitted, such as for a wall adjacent to a sidewalk/curb, a pipe under the sidewalk to be discharged through the curb face or equivalent should be provided. For a basement-type wall, a proper subdrain outlet system should be provided.
6) Retaining wall plans should be reviewed and approved by the geotechnical engineer.
7) Walls over six feet in height are subject to a special review by the geotechnical engineer and modifications to the above requirements.

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RETAINING WALL BACKFILL AND SUBDRAIN DETAIL

PLATE 2





NOTES:

1. SITE SOILS SHALL HAVE ADEQUATE DRAINAGE (AT LEAST 0.5 INCHES PER HOUR).
2. INFILTRATION SHALL NOT CAUSE GEOTECHNICAL HAZARDS RELATED TO EXPANSIVE SOIL MOVEMENT, TUNNEL EROSION, OR SLOPE STABILITY.
3. IF INFILTRATION HARZARDS ARE A CONCERN, AN UNDERDRAIN SHALL BE INSTALLED TO DRAIN WATER INTO STORM DRAIN INLET OR ONSITE BMP.
4. ANY OVERFLOW SHALL BE DISCHARGED PER BUREAU OF ENGINEERING AND BUILDING & SAFETY REQUIREMENTS.
5. SLOPE IS NOT GREATER THAN 3 PERCENT.
6. FLOW DIRECTED TO PERMEABLE PAVEMENT SHALL BE DISPERSED SO AS NOT TO BE CONCENTRATED AT A SMALL AREA OF PAVEMENT.
7. PRE-FABRICATED PRODUCTS HAVE BEEN INSTALLED PER ALL APPROPRIATE MANUFACTURER'S SPECIFICATIONS. IF REQUIRED, SUB-GRADE SOIL SHALL BE COMPACTED IN ACCORDANCE WITH PRODICT INSTALLATION SPECIFICATION.

GeoMat Testing Laboratories, Inc.
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Riverside, California 92503

DWN BY:	AM
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PROJECTION:	--
SCALE:	NTS
REV. NO.:	--

PROJECT: PRELIMINARY SOIL INVESTIGATION REPORT
APN: 191-150-031, 032 & 033
RIVERSIDE, CALIFORNIA

TITLE: TYPICAL PERMEABLE PAVEMENT DETAIL

DATE: APRIL 2020

PROJECT NO.: 20112-01

FIGURE NO.: Plate 3