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GEOTECHNICAL INVESTIGATION

Tadema Business Park
SWC Archibald and Merrill Avenues
Ontario, CA

Prepared For:
Caprock Land & Development Fund I, L.P.
2050 Main Street, Suite 240
Irvine, CA 92614

Prepared By:
MTGL, Inc.
2992 La Palma, Suite A
Anaheim, California 92806

September 10, 2015

MTGL Project No. 4815A04
MTGL Log No. 15-2115

MTGL

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September 10, 2015

MTGL Project No. 4815A04
MTGL Log No. 15-2115

Patrick Daniels, Chief Operating Officer
Caprock Land & Development Fund I, L.P.
2050 Main Street, Suite 240
Irvine, CA 92614

Subject: **GEOTECHNICAL INVESTIGATION**
Tadema Business Park
SWC of Archibald and Merrill Ave's
Ontario, California

Dear Mr. Daniels:

In accordance with your request and authorization we have completed a Geotechnical Investigation for the subject site. We are pleased to present the following report which addresses both engineering geologic and geotechnical conditions including a description of the site conditions, results of our field exploration and laboratory testing, and our conclusions and recommendations for grading and foundations design.

The project will consist of constructing several concrete tilt-up buildings ranging in size from 30,300 square feet to 537,600 square feet. The buildings will be located on the southwest corner of Archibald Avenue and Merrill Avenue in the city of Ontario, CA.

Our subsurface investigation consisted of a general site evaluation of the proposed business park. Additional geotechnical evaluations should be conducted for each individual structure once the building layouts are finalized.

Based on our investigation, the site will be suitable for construction, provided the recommendations presented herein are incorporated into the plans and specifications for the proposed construction. Details related to geologic conditions, seismicity, site preparation, foundation and pavement design, and construction considerations are also included in the subsequent sections of this report.

We appreciate this opportunity to be of continued service and look forward to providing additional consulting services during the planning and construction of the project. Should you have any questions regarding this report, please do not hesitate to contact us at your convenience.

Respectfully submitted,

MTGL, Inc.

Bryan Miller-Hicks, P.G., C.E.G.
Engineering Geologist

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ATTACHMENTS:

1.00 INTRODUCTION

In accordance with your request and authorization, MTGL, Inc. has completed a Geotechnical Investigation for the subject site. The following report presents a summary of our findings, conclusions and recommendations based on our investigation, laboratory testing, and engineering analysis.

1.01 PLANNED CONSTRUCTION

The project will consist of constructing several concrete tilt-up buildings ranging in size from 30,300 square feet to 537,600 square feet. The buildings will be located on the southwest corner of Archibald Avenue and Merrill Avenue in the city of Ontario, CA. The approximate location of the site is depicted on the accompanying Site Location Map (Figure 1).

1.02 SCOPE OF WORK

The scope of our Geotechnical services included the following:

- Review of geologic, seismic, ground water and geotechnical literature.
- Logging, sampling and backfilling of 20 exploratory borings drilled with an 8" hollow stem auger drill rig to a maximum depth of 51.5 feet below existing grades.
- One soil percolation test was performed to a depth of 15 feet located at the proposed retention basin area.
- Laboratory testing of representative samples (See Appendix C).
- Geotechnical engineering review of data and engineering recommendations.
- Preparation of this report summarizing our findings and presenting our conclusions and recommendations for the proposed construction.

1.03 SITE DESCRIPTION

The site is located on the southwest corner of Archibald and Merrill Avenues in the city of Ontario, county of San Bernardino, California. It is irregular in shape bounded by Merrill Avenue to the north, Archibald Avenue to the east, Remington (Hellman) Avenue to the south, and the Cucamonga Creek to the west. Topographically, the site is essentially planar gently sloping to the south at approximately a 1% grade. Elevation at the site ranges from approximately 640 to 660 feet above mean sea level. The approximate site location is shown on the accompanying Site Location Map (Figure 1).

At the time of our investigation, the north portion of the site was an active dairy farm and the south portion was actively cultivated cropland.

The dairy farm portion of the site contains numerous existing structures including residences, sheds, barns and storage structures, shade structures, fences and watering troughs. A large excavated borrow area lies within the southwesterly one-quarter of the dairy farm area. This area is approximately 15 feet deep, and 500 feet long by 100 feet wide.

The cultivated portion of the site contains irrigated agricultural fields where various vegetable crops are grown. No significant structures other than surface aluminum irrigation pipes running north-south are present on this portion of the site.

1.04 FIELD INVESTIGATION

Prior to the field investigation, a site reconnaissance was performed by a staff member from our office to mark the boring locations, as shown on the boring location plan, and to evaluate the boring locations with respect to obvious subsurface structures and access for the drilling rig. Underground Service Alert was then notified of the marked locations for utility clearance.

Our subsurface investigation consisted of drilling test borings utilizing a truck mounted drill rig equipped with an 8" diameter hollow stem auger. See Appendix B for further discussion of the field exploration including logs of test borings.

Borings were logged and sampled using Modified California Ring (Ring) samplers at selected depth intervals. Samplers were driven into the bottom of the boring with successive drops of a 140-pound weight falling 30 inches. Blows required driving the last 12 inches of the 18-inch Ring and SPT samplers are shown on the boring logs in the "blows/foot" column (Appendix B). SPT was performed in the borings in general accordance with the American Standard Testing Method (ASTM) D1586 Standard Test Method. Representative bulk soil samples were also obtained from our borings.

Each soil sample collected was inspected and described in general conformance with the Unified Soil Classification System (USCS). The soil descriptions were entered on the boring logs. All samples were sealed and packaged for transportation to our laboratory. After completion of drilling, borings were backfilled with the soil cuttings.

1.05 LABORATORY TESTING

Laboratory tests were performed on representative samples to verify the field classification of the recovered samples and to determine the geotechnical properties of the subsurface materials. All laboratory tests were performed in general conformance with ASTM or State of California Standard Methods. The results of our laboratory tests are presented in Appendix B of this report.

2.00 FINDINGS

2.01 REGIONAL GEOLOGIC CONDITIONS

The site lies within the Northeastern Block of the Transverse Ranges geomorphic province of California. The site is approximately 52 miles inland from the shore of the Pacific Ocean at an approximate elevation of 640 to 660 feet MSL (Google Earth, 2015). According to Norris and Web, the Transverse Ranges province is composed of a unique east-west trending unit of mountain ranges rising in places to altitudes of more than 10,000 feet above sea level, and containing four of the eight islands off the southern California coast. This province extends southward about 320 miles from Point Arguello on the west to the mountains of Joshua Tree National Park on the east. The province is between 40 and 60 miles wide and is bounded on its north side by the San Andreas Fault. The province is subdivided into numerous individual valleys and ranges all having a general east-west trend. These ranges are generally separated by broadly alluviated, synclinal valleys and prominent faults. The main mountain ranges have been uplifted through orogenic and compressional events and are associated with numerous east-west trending fault zones and smaller northeast trending faults, including the San Andreas, San Gabriel, Cucamonga and San Jacinto among others.

The northeastern block is situated between the Whittier fault zone and the base of the San Gabriel Mountains. The Raymond Hill fault separates the northeastern block from the northwestern block. Structurally, the northeastern block is a deep synclinal basin that contains mostly Cenozoic sedimentary rocks but also contains some Miocene volcanic rocks in the eastern portion. Mesozoic basement rocks underlie between approximately 12,000 and 22,000 feet of Cenozoic sedimentary cover.

2.02 SITE GEOLOGIC CONDITIONS

The site is located on generally level terrain at an elevation of approximately 640 to 660 feet above sea level (Google Earth, 2015). This region is characterized by a deep clastic valley fill from the nearby San Gabriel Mountains and alluvial deposits from numerous miscellaneous creeks and drainages. Soils immediately underlying the site consist of clastic non-marine sediments mapped as Younger Fan Deposits.

Twenty (20) 8-inch diameter hollow stem auger soil borings were advanced to characterize near-surface geologic conditions and to obtain soil samples for analyses. Boring locations and pertinent data for each boring are presented in the table below.

Boring No.	Depth (ft)	Elevation (ft)	Latitude	Longitude	Groundwater Depth (ft. bgs)
B1	5	656	33.9814°	-117.5970°	No GW
B2	51.5	651	33.9803°	-117.5969°	No GW
B3	21.5	651	33.9806°	-117.5956°	No GW
B4	5	653	33.9806°	-117.5949°	No GW
B5	21.5	660	33.9826°	-117.5984°	No GW
B6	21.5	654	33.9794°	-117.5985°	No GW
B7	21.5	657	33.9822°	-117.5977°	No GW
B8	21.5	660	33.9826°	-117.5958°	No GW
B9	21	660	33.9823°	-117.5939°	No GW
B10	21	654	33.9806°	-117.5937°	No GW
B11	21.5	654	33.9797°	-117.5937°	No GW
B12	51.5	647	33.9776°	-117.5970°	No GW
B13	21	651	33.9788°	-117.5967°	No GW
B14	14	652	33.7989°	-117.5940°	No GW
B15	21	648	33.9776°	-117.5939°	No GW
B16	21	646	33.9776°	-117.5951°	No GW
B17	5	647	33.9781°	-117.5959°	No GW
B18	5	644	33.9771°	-117.5994°	No GW
B19	21	645	33.9768°	-117.5986°	No GW
B20	15	643	33.9760°	-117.5985°	No GW

As shown on the attached boring logs, the site is underlain by alluvium. The alluvium consists of deposits of silty sand, layers of poorly graded fine to coarse sands, silts, interlayered sands and silts, and scattered lenses and layers of pebbles, gravels and gravelly sand. Some of the silt sediments are porous. Colors of the various materials were also variable: olive gray, light gray, tan, yellow gray, medium brown and gray brown. Moisture content of the alluvium ranged from dry in the top one to two feet, to wet. Groundwater was not encountered in our borings during our investigation.

Our field investigation was intended to evaluate overall site conditions. Additional field investigations will be required for each individual structure to evaluate specific subsurface soil conditions and to provide specific geotechnical recommendations for grading and foundations.

2.03 FLOODING POTENTIAL

The site is included within the FEMA Zone X, Other Flood Areas, as having a 0.2% chance of annual flooding. This zone is a designation of moderate flooding hazard from the 100-year and 500-year floods.

2.04 SURFACE AND GROUNDWATER CONDITIONS

Some small areas of ponding or standing water were present in the cultivated fields, due to agricultural irrigation, at the time of our study. No springs or areas of natural seepage were found.

Historic high groundwater levels in the immediate site vicinity are approximately 100 feet below existing ground surface (Carson and Matti, 1985 and Fife, 1974). At the time of our field investigation, groundwater was not encountered during drilling.

2.05 FAULTING AND SEISMICITY

Earthquake Faults comprise a significant geologic hazard to development in large areas of southern and northern California. Faults of most concern are those designated as active (showing evidence of movement within the last 11,000 years, the Holocene epoch on the geologic time scale) and potentially active (evidence of movement between 11,000 to about 1.6 million years). The site is within 25 miles of a number of active and potentially active faults in the seismically active southern California region, therefore, the potential for future strong ground shaking at the site is high. The site and site region will likely experience future earthquakes of moderate to large size. Some of the near-future earthquakes on nearby (less than about 50 miles) active faults may be greater than Richter magnitude 8.0. The site is not located within a mapped Alquist-Priolo Special Studies Zone. The closest Alquist-Priolo Special Studies zone, for the Chino fault, lies approximately 6 miles to the west, in the community of Los Serranos.

Numerous additional faults are present within 25 miles of the site, and it is reasonable to assume that the site will be subjected to severe seismic ground shaking from movement along one of the faults listed above or from any number of nearby faults. See Appendix D for a listing of all faults within the site vicinity.

2.06 LIQUEFACTION POTENTIAL

Liquefaction is a phenomenon wherein earthquake induced ground vibrations increase the pore pressure in saturated, granular soils until it is equal to the confining, overburden pressure. When this occurs, the soil can completely lose its shear strength and enter a liquefied state. The possibility of liquefaction is dependent upon grain size, relative density, confining pressure, saturation of the soils, strength of the ground motion and duration of ground shaking. In order for liquefaction to occur three general criteria must be met: underlying loose, coarse-grained (sandy) soils, a groundwater depth of less than about 50 feet and a nearby large magnitude earthquake.

Based on the relative density of the subsurface soils and depth to groundwater (on the order of 100 feet below existing ground surface), the potential for liquefaction at the site is very low.

2.07 LANDSLIDES

The site is not located in a hillside area of the county where earthquake induced landslides would cause permanent ground displacements. No reported occurrences of landslides or mudflows are known to have recently affected the site. Therefore, the potential for landslides and mudflows is considered to be very low at the site.

2.08 TSUNAMI AND SEICHE HAZARD

Given the inland location of the site at an elevation of approximately 650 feet MSL, the inundation hazard posed by tsunami is considered to be very low. Seiches are not considered a hazard due to the absence of above-ground tanks or reservoirs located immediately up gradient from the site.

3.00 CONCLUSIONS

3.01 GENERAL CONCLUSIONS

Based on our Geotechnical review of the planned construction, it is our opinion that the site is suitable for the proposed construction provided our conclusions are taken into consideration during design, and our recommendations are incorporated into the construction plans and specifications and implemented during grading and construction.

Given the findings of the investigation, it appears that the site geology is suitable for the proposed construction. Based on the investigation, it is our opinion that the proposed development is safe against landslides and settlement provided the recommendations presented in our report are incorporated into the design and construction of the project. Grading and construction of the proposed project will not adversely affect the geologic stability of adjacent properties. The nature and extent of the investigation conducted for the purposes of this declaration are, in our opinion, in conformance with generally accepted practice in this area. Therefore, the proposed project appears to be feasible from a geologic standpoint.

3.02 SEISMIC DESIGN PARAMETERS

The USGS Seismic Design Maps application, was used to calculate the CBC site specific design parameters as required by the 2013 California Building Code. Based upon the subsurface data, the site can be classified as Site Class D.

The spectral acceleration values for 0.2 second and 1 second periods obtained from the computer program and in accordance with the 2013 California Building Code are tabulated below.

Ground Motion Parameter	Design Value
S_S	1.500g
S_1	0.600g
Site Class	D
F_a	1.0
F_v	1.5
S_{MS}	1.500g
S_{M1}	0.900g
S_{DS}	1.000g
S_{D1}	0.600g

3.03 SOILS INFILTRATION TESTING

To establish the design infiltration rate, we have utilized the Percolation test Procedures of the San Bernardino County Department of Environmental Health as specified in the Technical Guidance Document Appendices of the Model Water Quality Management Plan.

One 8 inch diameter percolation test hole was drilled to a depth of 15 feet. The test hole was pre-soaked to allow the water flow to hold a constant level at least 5 times the hole's radius above the bottom of the hole. The approximate location of the test hole (B-20) is depicted in the accompanying Boring Location Map (Figure 2).

Based on the samples obtained from the exploratory borings, the subsurface materials are classified as silty sand to sandy silt (SM/ML) in accordance with the Unified Soil Classification System as shown on the accompanying boring logs (Appendix A). Our field test data sheets are attached in Appendix E of this report.

The grain size distribution affects soil permeability. Coarse-grained soils with large median particle sizes will yield higher infiltration rates. Finer grained soils will yield lower percolation rates. The percolation rates measured during our testing are as follows.

Test Hole	**Test Depth**	**Measured Percolation Rate**
P-1 (B-20)	15 ft	0.67 minutes / inch

Long-term sustainable infiltration rates may be affected by several factors including the degree of saturation of the adjacent ground and the infiltration of finer grained soils into the system. To account for these factors, the application of these rates should therefore consider the use of an appropriate factor of safety. The development of the factor of safety should be based upon the impacts of deteriorated performance and should anticipate that the measured test rates will be reduced over time.

4.00 RECOMMENDATIONS

Our recommendations are considered minimum and may be superseded by more conservative requirements of the architect, structural engineer, building code, or governing agencies. The foundation recommendations are based on the expansion index and shear strength of the onsite soils. Import soils, if necessary should possess very low expansion potential and should be approved by the Geotechnical Engineer prior to importing to the site. In addition to the recommendations in this section, additional general earthwork and grading specifications are included in Appendix F.

As previously noted, our subsurface investigation consisted of a general site evaluation of the proposed business park. Additional geotechnical evaluation will be required for each individual structure once the building layouts are finalized.

4.01 EXCAVATION CHARACTERISTICS/SHRINKAGE

Our exploratory borings encountered no oversize materials during our subsurface investigation. Accordingly we expect that all earth materials will be rippable with conventional heavy duty grading equipment.

Shrinkage is the decrease in volume of soil upon removal and recompaction expressed as a percentage of the original in-place volume, which will account for changes in earth volumes that will occur during grading. Our estimate for shrinkage of the onsite native soils are expected to range from 15 to 20 percent.

4.02 SETTLEMENT CONSIDERATIONS

Foundations should be designed to resist the anticipated settlements. Settlement of an individual foundation will vary depending on the plan dimensions of the foundation and the actual load supported. We estimate maximum settlement of foundations designed and constructed in accordance with the recommendations presented to be on the order of ½ inch. Differential settlement between similarly loaded and adjacent footings are expected to be approximately ¼ inch across 40 feet, provided footings are founded on similar materials. Settlement of all foundations is expected to occur rapidly and should be essentially complete shortly after initial application of the loads.

4.03 SITE CLEARING RECOMMENDATIONS

All surface vegetation, trash, debris, asphalt concrete, portland cement concrete and underground pipes should be cleared and removed from the proposed construction site. Underground facilities such as utilities, pipes or underground storage tanks may exist at the site. Removal of underground tanks is subject to state law as regulated by the County, City and/or Fire Department. If storage tanks containing hazardous or unknown substances are encountered, the proper authorities must be notified prior to any attempts at removing such objects.

Any water wells, if encountered during construction, should be exposed and capped in accordance with the requirements of the regulating agencies.

Depressions resulting from the removal of foundations of existing buildings, underground tanks and pipes, buried obstructions and/or tree roots should be backfilled with properly compacted material.

4.04 SITE GRADING RECOMMENDATIONS

All fill materials should be compacted to at least 90 percent of maximum dry density as determined by ASTM Test Method D1557. Fill materials should be placed in loose lifts, no greater than 8 inches prior to applying compactive effort. All engineered fill materials should be moisture-conditioned and processed as necessary to achieve a uniform moisture content that is near optimum moisture content and within moisture limits required to achieve adequate bonding between lifts.

4.05 SITE OVEREXCAVATION

Building plans, grading plans and foundation elevations were not available at the time of our investigation. Therefore, once formal plans are prepared and available for review, this office should review these plans from a geotechnical viewpoint, comment on any changes, and revise the recommendations of this report as necessary.

All artificial fills, organics, debris, trash and topsoil should be removed from the grading area and hauled offsite. Recommendations for site grading to prepare the building pad area for the support of structures are as follows.

It is recommended that the existing soils within the pad area be over excavated to a minimum depth of 3 feet below the bottom of the proposed footings or 5 feet below the existing grade, whichever is greater. The required horizontal limits of the over excavated area shall be defined as the area extending from the edge of the perimeter footing for a distance of 5 feet.

Hardscape areas which include all paved areas will require a minimum depth of 2 feet of removal and recompaction. Processing for hardscape areas should extend a minimum distance of 2 feet outside the hardscape limits.

4.06 FILL MATERIALS

Removed and/or overexcavated soils may be moisture-conditioned to near optimum moisture content and recompacted as engineered fill, except for soils containing detrimental amounts of organic material. Our subsurface investigation indicates that the near surface materials are generally at or below its optimum moisture content. The fill materials should be compacted to a minimum of 90% of the maximum dry density per ASTM D-1557.

Imported materials shall be coarse grained, non-expansive, and non-plastic in nature. The materials should be free from vegetable matter and other deleterious substances, shall not contain rocks or lumps of a greater dimension than 4 inches, and shall be approved by the geotechnical consultant. Soils of poor gradation, expansion, or strength properties shall be placed in areas designated by the geotechnical consultant or shall be mixed with other soils providing satisfactory fill material.

4.07 FOUNDATIONS

For preliminary design purposes, conventional spread and/or continuous footings on compacted fill materials may be used to support the proposed structures and designed using an allowable bearing pressure of 2,000 psf. The allowable bearing capacity may also be increased by one-third for considerations of short term wind or seismic loads. The recommended minimum footing width and embedment depth below the lowest adjacent grade are as follows:

Foundation Type	**Minimum Width**	**Minimum Depth**
Continuous (Interior)	12 inches	18 inches
Continuous (Perimeter)	18 inches	24 inches
Spread Footings	24 inches	24 inches

Soil resistance developed against lateral structural movement can be obtained from the passive pressure value of 300 pcf. For sliding resistance, a friction coefficient of 0.30 may be used at the concrete and soil interface. The passive pressure and the friction of resistance could be combined without reduction. In addition, the lateral passive resistance is taken into account only if it is ensured that the soil against embedded structures will remain intact with time.

The near surface soils have an expansion index classification of very low. Therefore, nominal reinforcement consisting of two #5 bars placed within 3 inches of the top of footings and two #5 bars placed within 3 inches of the bottom of footings are recommended. However, the structural engineer may require heavier reinforcement.

Due to the preliminary nature of this investigation, additional site specific field investigations for each building structure should be conducted to verify these preliminary recommendations.

4.08 CONCRETE SLABS ON GRADE AND MISCELLANEOUS FLATWORK

Concrete slabs on grade and miscellaneous flatwork that are not subjected to heavy loads or vehicular loads may be designed with a minimum thickness of 5.0 inches for normal loading conditions. However, if heavier loads are anticipated, a modulus of subgrade reaction of 120 pounds per cubic inch may be used when the slabs are supported by compacted fill.

All slabs and flatwork should be reinforced with a minimum #4 bars, 18 inches on center, each direction at the mid-height of the slab. The structural engineer may require heavier reinforcement. Special care should be taken so that reinforcement is placed at the slab mid-height. The floor slab should be separated from footings, structural walls, and utilities and provisions made to allow for settlement or swelling movements at these interfaces. If this is not possible from a structural or architectural design standpoint, it is recommended that the slab connection to footings be reinforced such that there will be resistance to potential differential movement.

Control joints should be constructed on all slabs on grade to create squares or rectangles with a maximum spacing of 12 feet on large slab areas. Where flatwork is adjacent to curbs, reinforcing bars should be placed between the flatwork and the curbs. Expansion joint material should be used between flatwork and curbs, and flatwork and buildings.

Subsurface moisture and moisture vapor naturally migrate upward through the soil and where the soil is covered by a building or pavement. To reduce the impact of the subsurface moisture and potential impact of future introduced moisture (such as landscape irrigation or precipitation) damp proofing should be provided under all slabs on grade with moisture sensitive floor coverings. The damp proofing should consist of a minimum 10 mil polyethylene liner placed with 2 inches of sand below and 2 inches of sand above the polyethylene liner. The liner should be carefully fitted around service openings with joints lapped not less than 6 inches.

Damp proofing typically will not necessarily assure that floor slab moisture transmission rates will meet floor-covering manufacturer standards. Other factors such as surface grades, adjacent planters, the quality of slab concrete and the permeability of the on-site soils will affect slab moisture. In many cases, floor moisture problems are the result of either improper curing of floors slabs or improper application of flooring adhesives. We recommend contacting a flooring consultant experienced in the area of concrete slab-on-grade floors for specific recommendations regarding the proposed flooring applications. We make no guarantee nor provide any assurance that use of a vapor retarder system will reduce concrete slab-on-grade floor moisture penetration to any specific rate or level, particularly those required by floor covering manufacturers. The builder and designers should consider all available measures for floor slab moisture protection.

Special precautions must be taken during the placement and curing of all concrete slabs. Excessive slump (high water-cement ratio) of the concrete and/or improper curing procedures used during either hot or cold weather conditions could lead to excessive shrinkage, cracking, or curling of the slabs. High water-cement ratio and/or improper curing also greatly increase the water vapor permeability of concrete. We recommend that all concrete placement and curing operations be performed in accordance with the American Concrete Institute (ACI) manual of practice.

The subgrade soils beneath all concrete flatwork should be compacted to a minimum of 90% relative compaction for a minimum depth of 24 inches. The geotechnical engineer should monitor the compaction of the subgrade soils and perform testing to verify that proper compaction has been obtained.

4.09 PREWETTING RECOMMENDATION

Prior to placing concrete slabs and flatwork, the underlying soils should be brought to a minimum of 2% and a maximum of 4% above their optimum moisture content for a depth of 12 inches prior to the placement of concrete. The geotechnical consultant should perform insitu moisture tests to verify that the appropriate moisture content has been achieved a maximum of 24 hours prior to the placement of concrete or moisture barriers.

Once the slab subgrade soil has been pre-wetted and compacted, the soil should not be allowed to dry prior to concrete placement. If the subgrade soil is dry, the moisture content of the soil should be restored prior to placement of concrete and re-tested.

Proper moisture conditioning and compaction of subgrade soils prior to placement is very important prior to concrete placement. Even with proper site preparation, some soil moisture changes of the subgrade soils supporting the concrete flatwork due to edge effects (shrink/swell) may occur. Drying and/or wetting of subgrade soils adjacent to landscaped areas or open fields may increase the potential of shrink/swell effects beneath concrete flatwork areas. To help reduce edge effects, lateral cutoffs, such as inverted curbs are recommended. Control joints should be used to reduce the potential for flatwork panel cracks as a result of minor soil shrink/swell.

4.10 CORROSIVITY

Soluble sulfate tests indicate that concrete at the subject site will have a negligible exposure to water soluble sulfate in the soil. Our recommendations for concrete exposed to sulfate-containing soils are presented below.

RECOMMENDATIONS FOR CONCRETE EXPOSED TO SULFATE CONTAINING SOILS

Sulfate Exposure Severity	Class	Water soluble sulfate (SO_4) in soil (% by wgt)	Sulfate (SO_4) in water (ppm)	Max Water to Cement Ratio by Weight	Minimum Compressive Strength (psi)	Cement Type	Calcium Chloride Admixture
Negligible	S0	0.00 - 0.10	0-150	---	2,500	---	No Restriction
Moderate	S1	0.10 - 0.20	150-1,500	0.50	4,000	II/V	No Restriction
Severe	S2	0.20 - 2.00	1,500-10,000	0.45	4,500	V	Not Permitted
Very Severe	S3	Over 2.00	Over 10,000	0.45	4,500	V Plus Pozzolan	Not Permitted

Corrositivity testing consisting of soils reactivity (pH) and resistivity (ohms-cm) were also tested on representative soils. The test results indicate that the soils have a soil reactivity ranging from 7.7 to 7.8, and a resistivity ranging from 300 to 1,200 ohms-cm. A neutral or non-corrosive soil has a reactivity value ranging from 5.5 to 8.4. Generally, soils that could be considered corrosive to metal have resistivities less than 3,000 ohms. Those soils with resistivity values of less than 1000 ohms-cm can be considered extremely corrosive.

Based on our test results, the finer grained soils at the near surface will have a moderate corrosion potential. Protection of buried metal with sand bedding and protective coating may be used to further reduce corrosion potential. A qualified corrosion engineer should be consulted to further assess the corrosion potential, as necessary.

4.11 RETAINING WALLS

Embedded structural walls should be designed for lateral earth pressures exerted on the walls. The magnitude of these earth pressures will depend on the amount of deformation that the wall can yield under the load. If the wall can yield sufficiently to mobilize the full shear strength of the soils, it may be designed for the active condition. If the wall cannot yield under the applied load, then the shear strength of the soil cannot be mobilized and the earth pressures will be higher. These walls such as basement walls and swimming pools should be designed for the at rest condition. If a structure moves towards the retained soils, the resulting resistance developed by the soil will be the passive resistance.

For design purposes, the recommended equivalent fluid pressure for each case for walls constructed above the static groundwater table and backfilled with non-expansive soils is provided below. Retaining wall backfill should be compacted to at least 90% relative compaction based on the maximum density defined by ASTM D1557.

Retaining structures may be designed to resist the following lateral earth pressures.

- Allowable Bearing Pressure – 2,000 psf
- Coefficient of Friction (Soil to Footing) – 0.30
- Passive Earth Pressure - equivalent fluid weight of 300 pcf (Maximum of 2,500 psf)
- At rest lateral earth pressure - 60 pcf
- Active Earth Pressures – equivalent fluid weights:

Slope of Retained Material	Equivalent Fluid Weight (pcf)
Level	40
2:1 (H:V)	65

It is recommended that all retaining wall footings be embedded at least 24 inches below the lowest adjacent finish grade. In addition, the wall footings should be designed and reinforced as required for structural considerations. The wall areas should be over excavated to a minimum depth of 2 feet below the bottom of the proposed footings. The required horizontal limits of the over excavated area shall be defined as the area extending from the edge of the footing for a minimum distance of 2 feet.

Lateral resistance parameters provided above are ultimate values. Therefore, a suitable factor of safety should be applied to these values for design purposes. The appropriate factor of safety will depend on the design condition and should be determined by the project Structural Engineer. If any super-imposed loads are anticipated, this office should be notified so that appropriate recommendations for earth pressures may be provided.

Retaining structures should be drained to prevent the accumulation of subsurface water behind the walls. Back drains should be installed behind all retaining walls exceeding 3.0 feet in height. A typical detail for retaining wall back drains is presented as Figure 8. All back drains should be outlet to suitable drainage devices. Walls and portions thereof that retain soil and enclose interior spaces and floors below grade should be waterproofed and damp-proofed accordingly.

4.12 SEISMICALLY INDUCED LATERAL EARTH PRESSURES

A seismic lateral increment of 22 pcf (equivalent fluid weight) may be applied as an incremental force which should be applied to the back of the wall in the upper 1/3 of the wall and also applied as a reduction of force to the front of the wall in the upper 1/3 of the footing.

4.13 PAVEMENT RECOMMENDATIONS

Recommended pavement structural sections are based on the procedures outlined in "Design Procedures for Flexible Pavements" of the Highway Design Manual, California Transportation Department. This procedure uses the principal that the pavement structural section must be of adequate thickness to distribute the load from the design traffic (TI) to the subgrade soils in such a manner that the stresses from the applied loads do not exceed the strength of the soil (R value).
Pavement sections were designed based on an R-Value of 49 and assumed Traffic Index of 5.5 for local traffic (light auto parking and drive lanes), 7.0 for collector streets, and 10.0 for truck access/fire lanes. The recommend structural sections are as follows:

ASPHALT PAVEMENT STRUCTURAL SECTION

Pavement Area	Traffic Index	Asphalt Thickness	Aggregate Base Thickness
Local Streets (Light Auto Parking / Drive Lanes)	5.5	4.0"	6.0"
Collector Streets (Commercial Vehicles)	7.0	6.0"	8.0"
Truck Access/Fire Lane (Heavy Truck Traffic)	10.0	7.0"	8.0"

PORTLAND CEMENT CONCRETE (PCC) STRUCTURAL SECTION

Pavement Area	Traffic Index	PCC Thickness	Aggregate Base Thickness
Local Streets (Light Auto Parking / Drive Lanes)	5.5	7.0"	6.0"
Collector Streets (Commercial Vehicles)	7.0	8.0"	6.0"
Truck Access/Fire Lane (Heavy Truck Traffic)	10.0	9.0"	6.0"

The subgrade soils beneath all pavement sections should be compacted to a minimum of 90% relative compaction for a minimum depth of 24 inches. All aggregate base courses should be compacted to a minimum of 95% relative compaction. The geotechnical engineer should monitor the compaction of the subgrade soils to verify that proper compaction has been obtained.

4.14 CONSTRUCTION CONSIDERATIONS

4.14.1 MOISTURE SENSITIVE SOILS/WEATHER RELATED CONCERNS

The upper soils encountered at this site may be sensitive to disturbances caused by construction traffic and to changes in moisture content. During wet weather periods, increases in the moisture content of the soil can cause significant reduction in the soil strength and its support capabilities. In addition, soils that become excessively wet may be slow to dry and thus significantly delay the progress of the grading operations. Therefore, it will be advantageous to perform earthwork and foundation construction activities during the dry season. Much of the on-site soils may be susceptible to erosion during periods of inclement weather. As a result, the project Civil Engineer/Architect and Grading Contractor should take appropriate precautions to reduce the potential for erosion during and after construction.

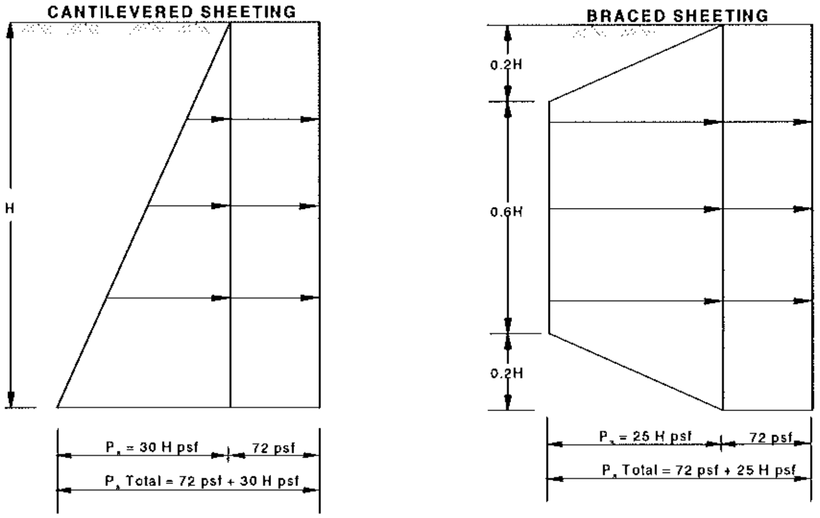
4.14.2 DRAINAGE AND GROUNDWATER CONSIDERATIONS

Historic high groundwater levels in the immediate site vicinity are approximately 100 feet below grade. Since this is well below the anticipated depths of grading, the installation of subdrains is not expected to be necessary. However, variations in the ground water table may result from fluctuation in the ground surface topography, subsurface stratification, precipitation, irrigation, and other factors such as impermeable and/or cemented formational materials overlain by fill soils. In addition, during retaining wall excavations, seepage may be encountered. Therefore, we recommend that a representative of MTGL, Inc. be present during grading operations to evaluate areas of seepage. Drainage devices for reduction of water accumulation can be recommended should these conditions occur.

Water should not be allowed to collect in the foundation excavation, on floor slab areas, or on prepared subgrades of the construction area either during or after construction. Undercut or excavated areas should be sloped to facilitate removal of any collected rainwater, groundwater, or surface runoff. Positive site drainage should be provided to reduce infiltration of surface water around the perimeter of the building and beneath the floor slabs. The grades should be sloped away from the building and surface drainage should be collected and discharged such that water is not permitted to infiltrate the backfill and floor slab areas of the building.

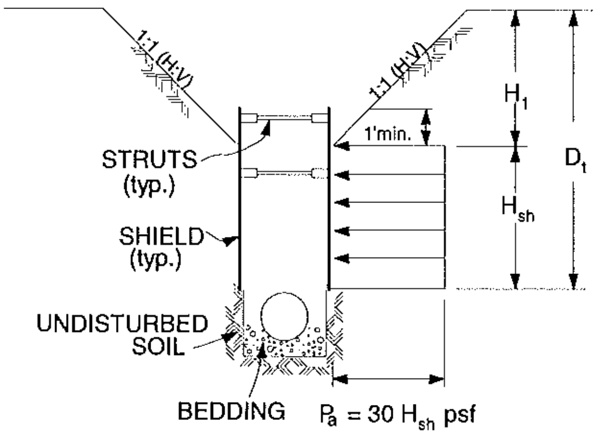
4.14.3 TEMPORARY EXCAVATIONS AND SHORING

Short term temporary excavations in existing soils may be safely made at an inclination of 1:1 (horizontal to vertical) or flatter. If vertical sidewalls are required in excavations greater than 5 feet in depth, the use of cantilevered or braced shoring is recommended. Excavations less than 5 feet in depth may be constructed with vertical sidewalls without shoring or shielding. Our recommendations for lateral earth pressures to be used in the design of cantilevered and/or braced shoring are presented below. These values incorporate a uniform lateral pressure of 72 psf to provide for the normal construction loads imposed by vehicles, equipment, materials, and workmen on the surface adjacent to the trench excavation. However, if vehicles, equipment, materials, etc. are kept a minimum distance equal to the height of the excavation away from the edge of the excavation, this surcharge load need not be applied.



SHORING DESIGN: LATERAL SHORING PRESSURES

Design of the shield struts should be based on a value of 0.65 times the indicated pressure, Pa, for the approximate trench depth. The wales and sheeting can be designed for a value of 2/3 the design strut value.



HEIGHT OF SHIELD, H_{sh} = DEPTH OF TRENCH, D_t , MINUS DEPTH OF SLOPE, H_1

TYPICAL SHORING
DETAIL

Placement of the shield may be made after the excavation is completed or driven down as the material is excavated from inside of the shield. If placed after the excavation, some overexcavation may be required to allow for the shield width and advancement of the shield. The shield may be placed at either the top or the bottom of the pipe zone. Due to the anticipated thinness of the shield walls, removal of the shield after construction should have negligible effects on the load factor of pipes. Shields may be successively placed with conventional trenching equipment.

Vehicles, equipment, materials, etc. should be set back away from the edge of temporary excavations a minimum distance of 15 feet from the top edge of the excavation. Surface waters should be diverted away from temporary excavations and prevented from draining over the top of the excavation and down the slope face. During periods of heavy rain, the slope face should be protected with sandbags to prevent drainage over the edge of the slope, and a visqueen liner placed on the slope face to prevent erosion of the slope face.

Periodic observations of the excavations should be made by the geotechnical consultant to verify that the soil conditions have not varied from those anticipated and to monitor the overall condition of the temporary excavations over time. If at any time during construction conditions are encountered which differ from those anticipated, the geotechnical consultant should be contacted and allowed to analyze the field conditions prior to commencing work within the excavation. All Cal/OSHA construction safety orders should be observed during all underground work.

4.14.4 UTILITY TRENCHES

All Cal/OSHA construction safety orders should be observed during all underground work. All utility trench backfill within street right of way, utility easements, under or adjacent to sidewalks, driveways, or building pads should be observed and tested by the geotechnical consultant to verify proper compaction. Trenches excavated adjacent to foundations should not extend within the footing influence zone defined as the area within a line projected at a 1:1 (horizontal to vertical) drawn from the bottom edge of the footing. Trenches crossing perpendicular to foundations should be excavated and backfilled prior to the construction of the foundations. The excavations should be backfilled in the presence of the geotechnical engineer and tested to verify adequate compaction beneath the proposed footing.

Utilities should be bedded and backfilled with clean sand or approved granular soil to a depth of at least 1-foot over the pipe. The bedding materials shall consist of sand, gravel, crushed aggregates, or native soils that are free draining with a sand equivalence of not less than 30. The bedding should be uniformly watered and compacted to a firm condition for pipe support.

The remainder of the backfill shall be typical on-site soil or imported soil which should be placed in lifts not exceeding 8 inches in thickness, watered or aerated to near optimum moisture content, and mechanically compacted to at least 90% of maximum dry density (ASTM D1557).

The bedding and backfill materials and placement shall conform to the requirements of the latest Standard Specifications for Public Works Construction (Greenbook).

4.14.5 SITE DRAINAGE

The site should be drained to provide for positive drainage away from structures in accordance with the building code and applicable local requirements. Unpaved areas should slope no less than 2% away from structure. Paved areas should slope no less than 1% away from structures. Concentrated roof and surface drainage from the site should be collected in engineered, non-erosive drainage devices and conducted to a safe point of discharge. The site drainage should be designed by a civil engineer.

4.15 GEOTECHNICAL OBSERVATION/TESTING OF EARTHWORK OPERATIONS

The recommendations provided in this report are based on preliminary design information and subsurface conditions as interpreted from the investigation. Our preliminary conclusion and recommendations should be reviewed and verified during site grading, and revised accordingly if exposed Geotechnical conditions vary from our preliminary findings and interpretations. The Geotechnical consultant should perform Geotechnical observation and testing during the following phases of grading and construction:

- During site grading and overexcavation.
- During foundation excavations and placement.
- Upon completion of retaining wall footing excavation prior to placing concrete.
- During excavation and backfilling of all utility trenches
- During processing and compaction of the subgrade for the access and parking areas and prior to construction of pavement sections.
- When any unusual or unexpected Geotechnical conditions are encountered during any phase of construction.

5.00 LIMITATIONS

The findings, conclusions, and recommendations contained in this report are based on the site conditions as they existed at the time of our investigation, and further assume that the subsurface conditions encountered during our investigation are representative of conditions throughout the site. Should subsurface conditions be encountered during construction that are different from those described in this report, this office should be notified immediately so that our recommendations may be re-evaluated.

This report was prepared for the exclusive use and benefit of the owner, architect, and engineer for evaluating the design of the facilities as it relates to geotechnical aspects. It should be made available to prospective contractors for information on factual data only, and not as a warranty of subsurface conditions included in this report.

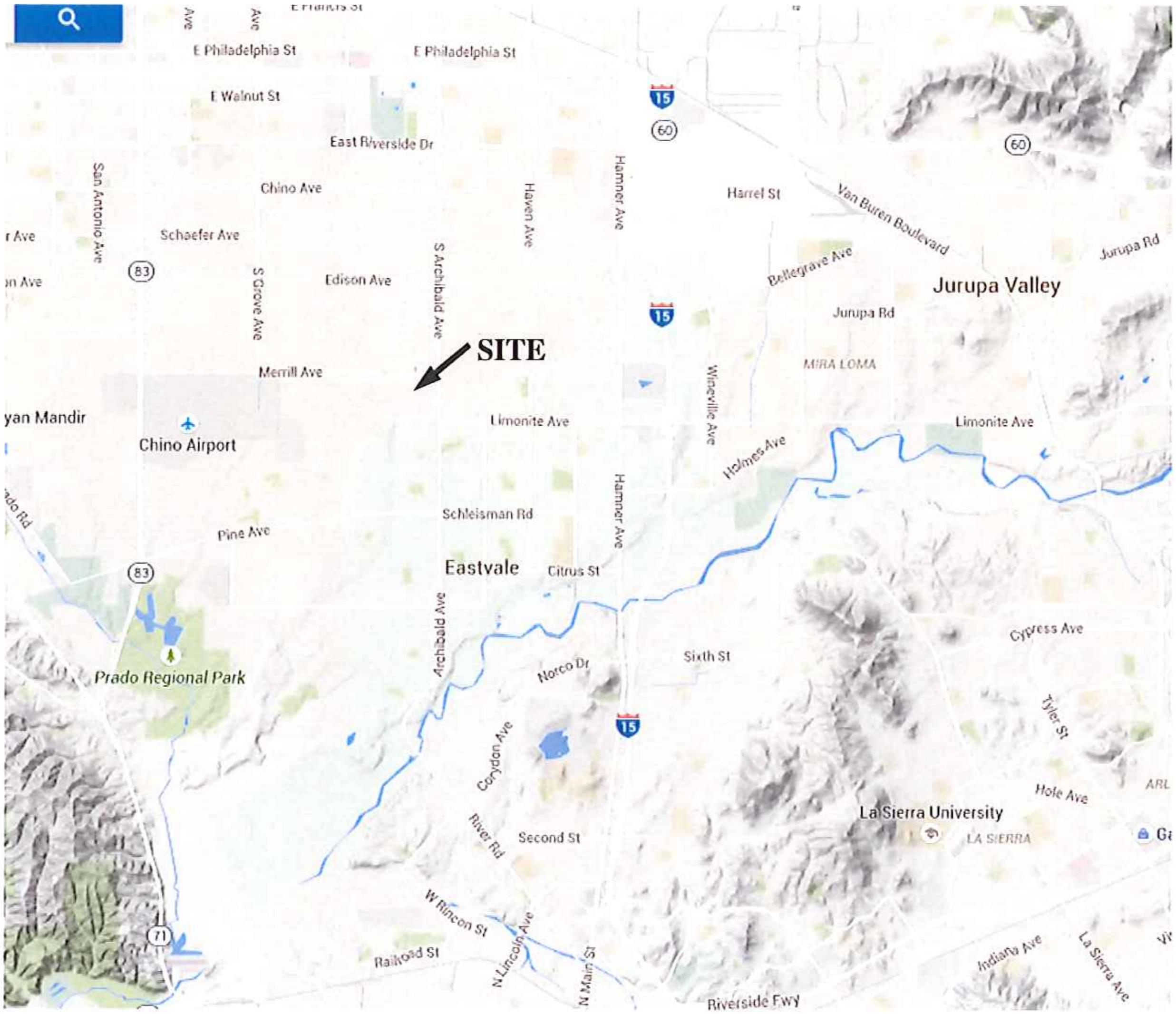
Our investigation was performed using the standard of care and level of skill ordinarily exercised under similar circumstances by reputable soil engineers and geologists currently practicing in this or similar localities. No other warranty, express or implied, is made as to the conclusions and professional advice included in this report.

This firm does not practice or consult in the field of safety engineering. We do not direct the Contractor's operations, and we are not responsible for their actions. The contractor will be solely and completely responsible for working conditions on the job site, including the safety of all persons and property during performance of the work. This responsibility will apply continuously and will not be limited to our normal hours of operation.

The findings of this report are considered valid as of the present date. However, changes in the conditions of a site can occur with the passage of time, whether they are due to natural events or to human activities on this or adjacent sites. In addition, changes in applicable or appropriate codes and standards may occur, whether they result from legislation or the broadening of knowledge.

Accordingly, this report may become invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and revision as changed conditions are identified.

FIGURES

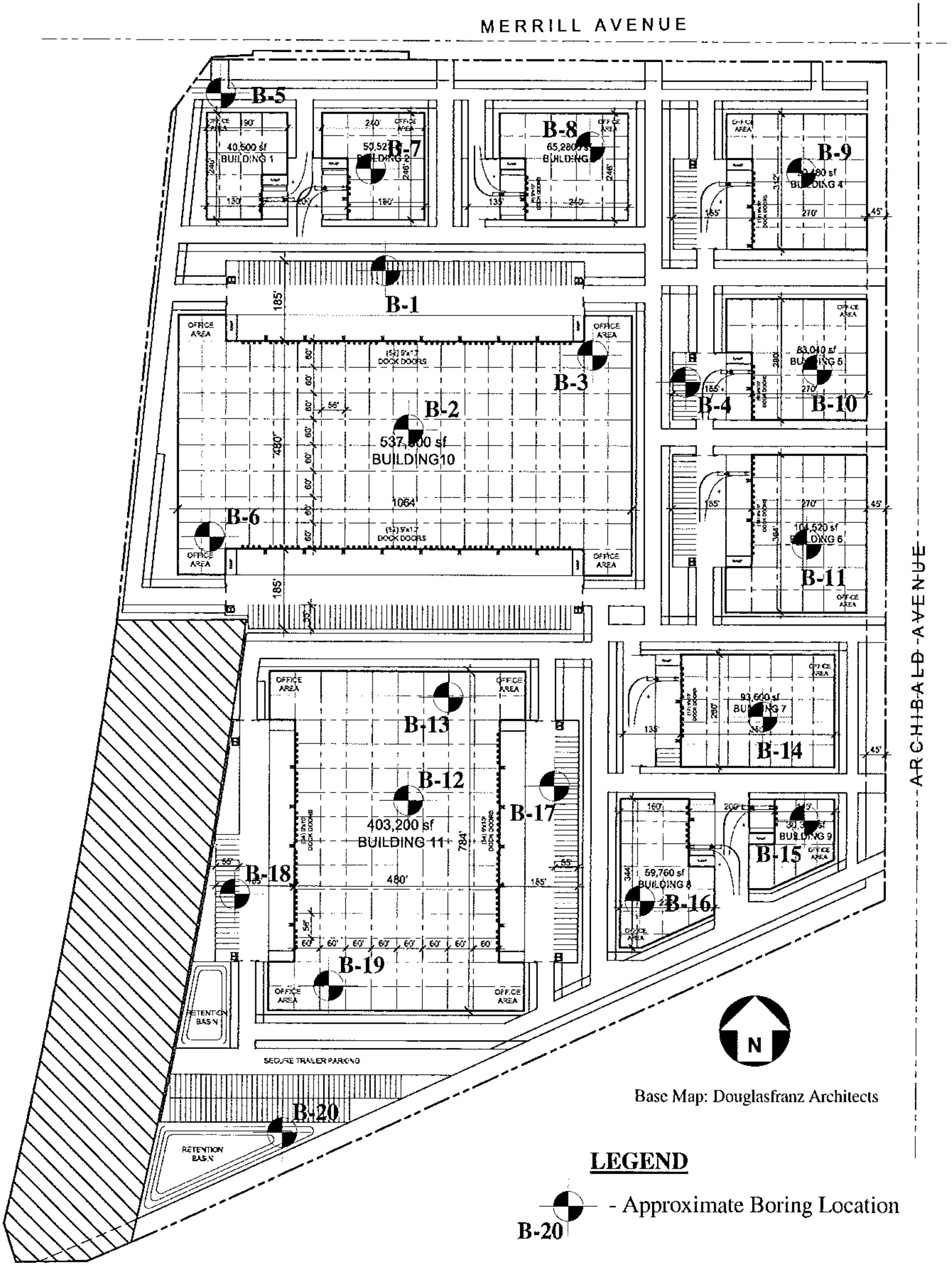


N

Base Map: Google Earth

SITE LOCATION MAP
Not to Scale

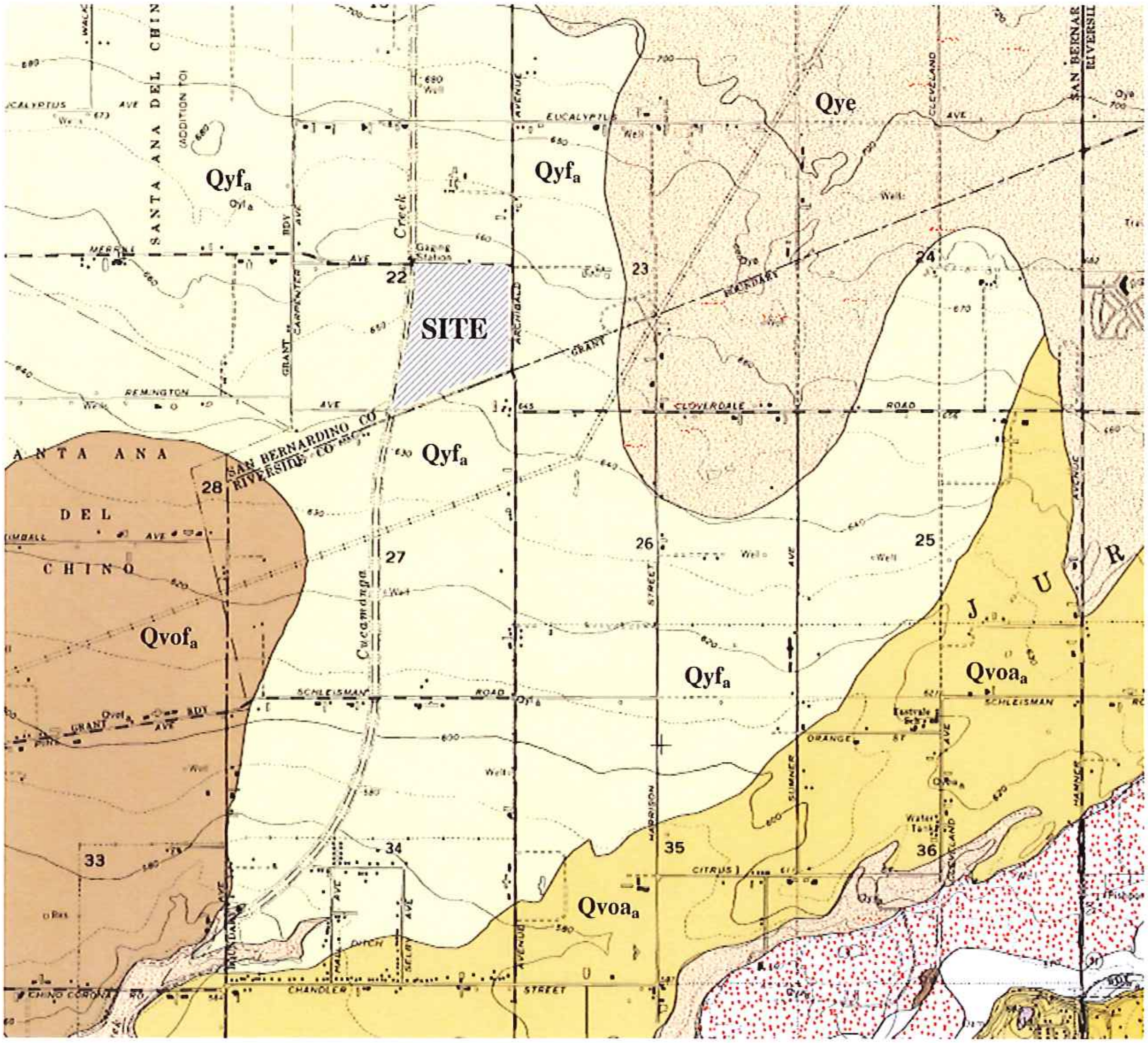
Figure 1



BORING LOCATION MAP

Not to Scale

Figure 2



Source: Morton, M. and Miller F., USGS Preliminary Geologic Map of Corona North 7.5 Minute Quadrangle, OFR 02-22

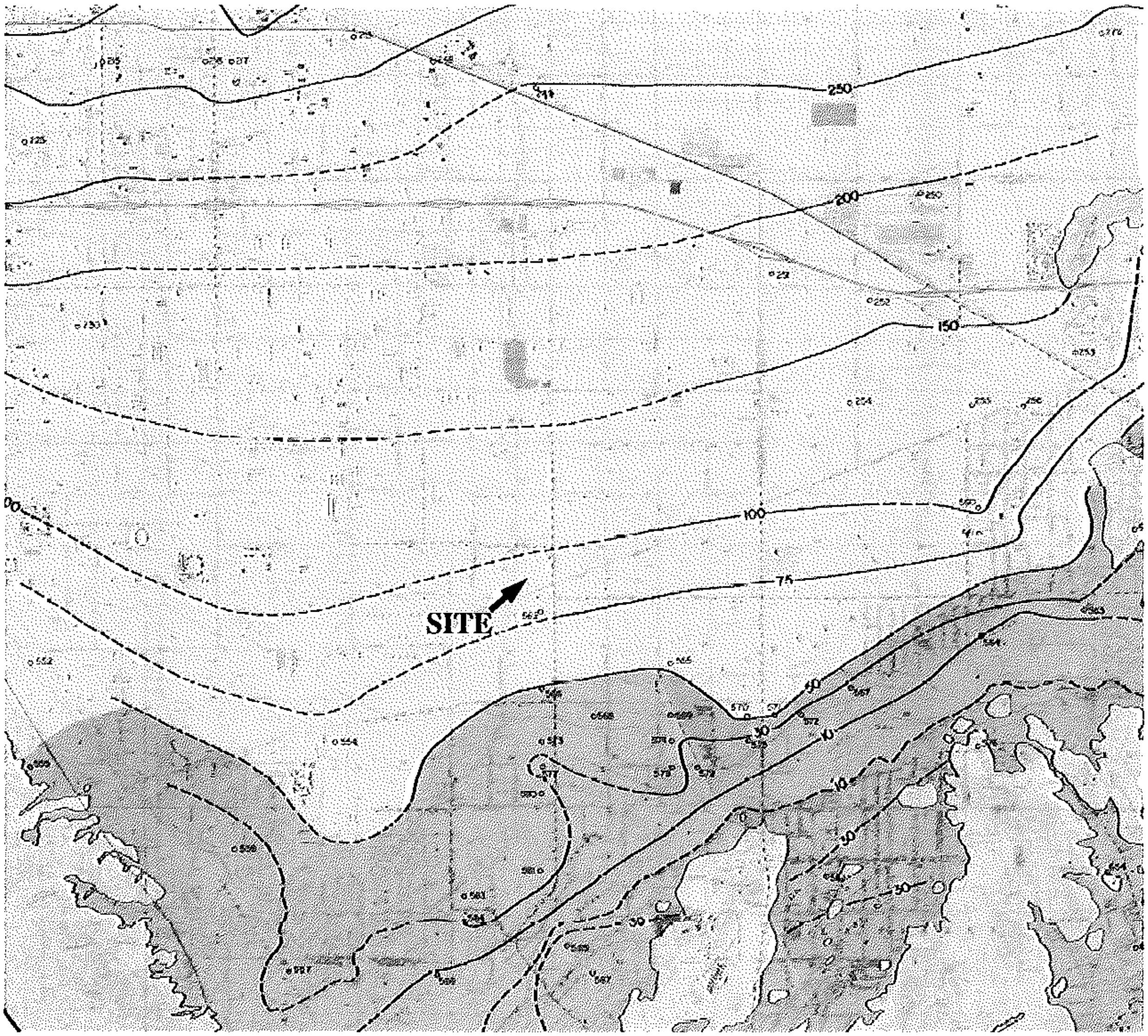
DESCRIPTION OF UNITS IN IMMEDIATE SITE VICINITY:

Qyf – Young Alluvial Fan Deposits Late Holocene
Qvof – Very Old Alluvial Fan Deposits Early Pleistocene
Qvoa– Very old alluvial channel deposits Early Pleistocene

REGIONAL GEOLOGIC MAP

Not to Scale

Figure 3

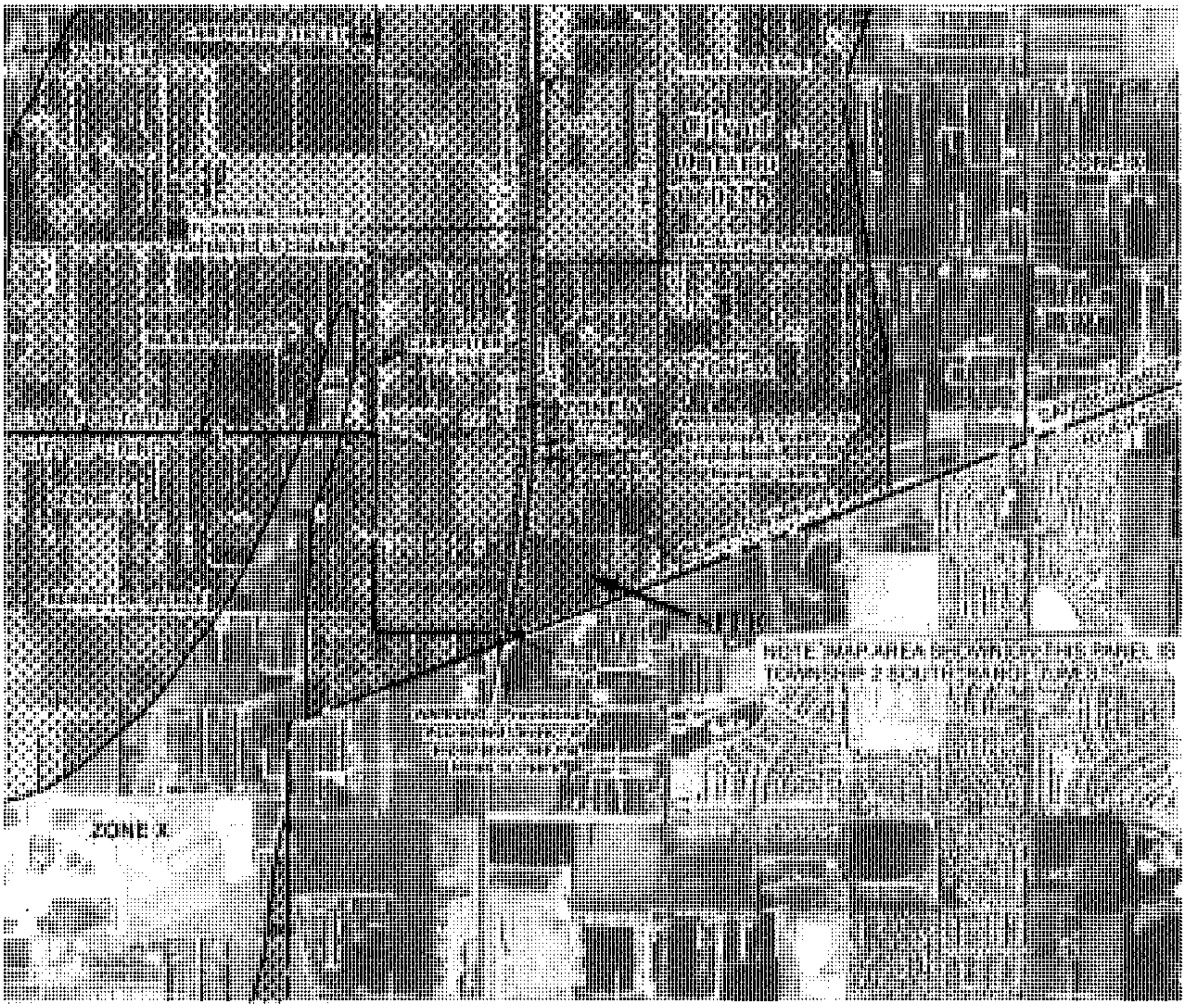


REGIONAL GROUNDWATER MAP

Not to Scale

Source: Carson, S.E. and Matti, J.C., 1985, Contour Map Showing Minimum Depth to Ground Water, Upper Santa Ana River Valley, California, 1973 1979: U.S. Geological Survey Map MF 1802.

Figure 4

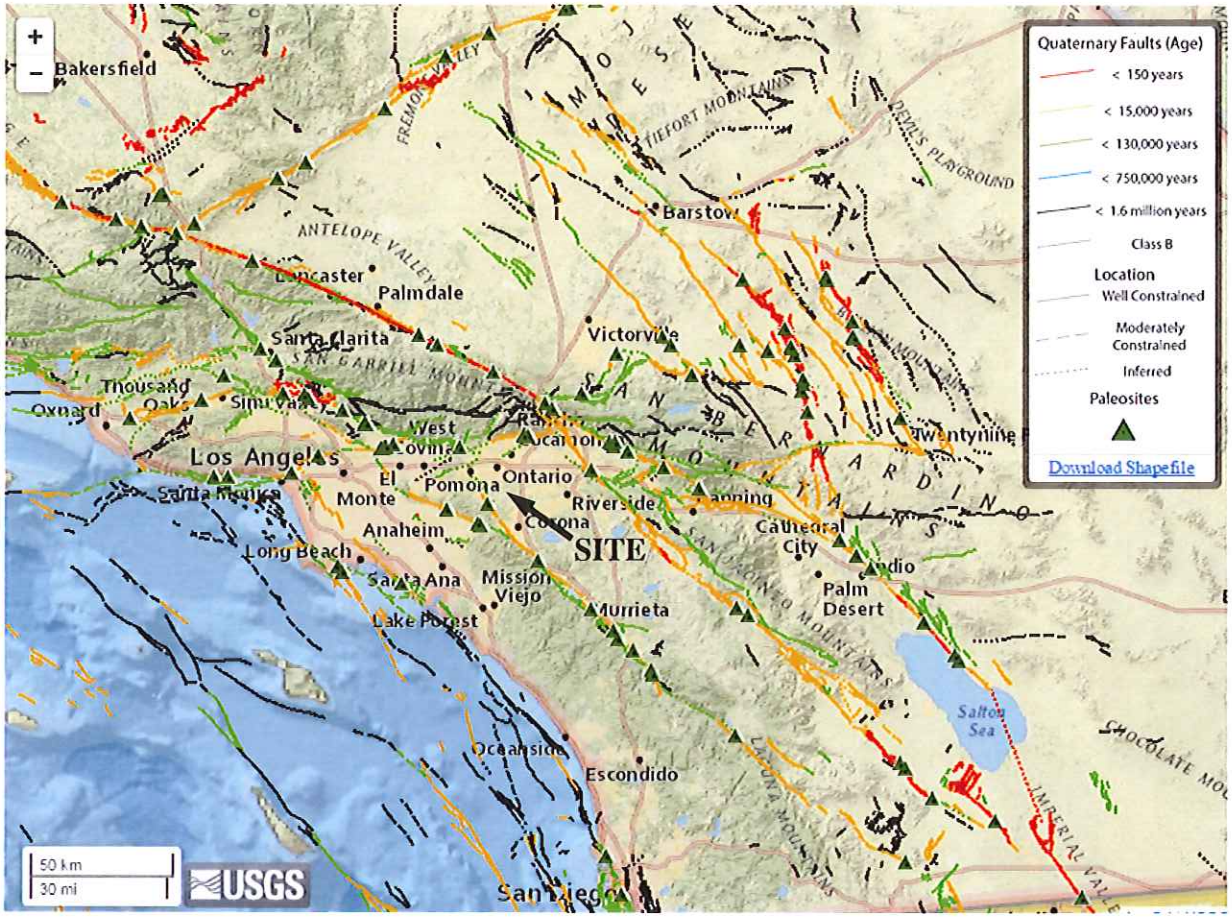


FLOOD ZONE MAP

Not to Scale

Source: FEMA, Flood Insurance Rate Map, Panel 9375H of 9400, Map No. 06071C9375H, August 28, 2008

Figure 5



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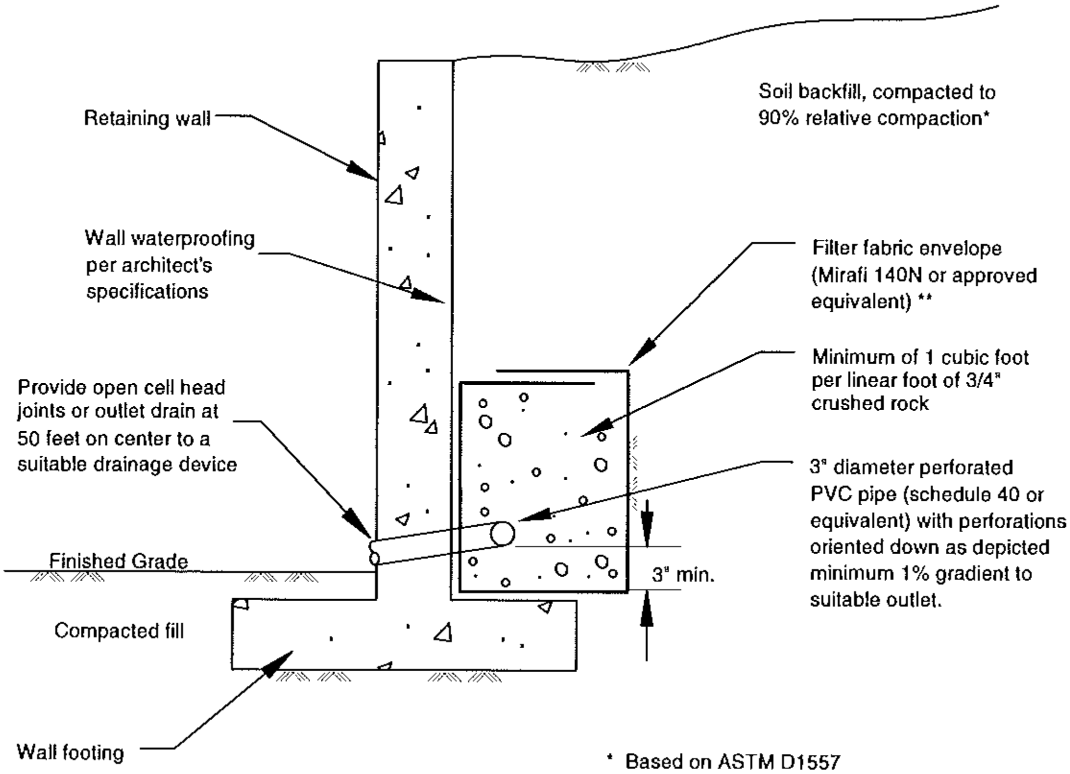
Source: USGS Interactive Fault Map

REGIONAL FAULT MAP

Scale: As Shown

Figure 6



* Based on ASTM D1557

** If class 2 permeable material (See gradation to left) is used in place of 3/4" - 1 1/2" gravel. Filter fabric may be deleted. Class 2 permeable material compacted to 90% relative compaction. *

SPECIFICATIONS FOR CLASS 2
PERMEABLE MATERIAL
(CAL TRANS SPECIFICATIONS)

Sieve Size	% 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

RETAINING WALL DRAINAGE DETAIL

Figure 7

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

Bortungo, E.J. and Spittler, T.E., 1986, Geologic Map of the San Bernardino Quadrangle: California Department of Mines and Geology Map 3A.

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APPENDIX B

FIELD EXPLORATION PROGRAM

APPENDIX B

FIELD EXPLORATION PROGRAM

The subsurface conditions for this Geotechnical Investigation were explored by excavating exploratory borings with an 8-inch hollow-stem-auger to a maximum depth of 51.5 feet below existing grade. All drive samples were obtained by SPT or California Tube Sampler. The approximate locations of the borings are shown on the Boring Location Plan (Figure 2). The field exploration was performed under the supervision of our Geologist who maintained a continuous log of the subsurface soils encountered and obtained samples for laboratory testing.

Subsurface conditions are summarized on the accompanying Logs of Borings. The logs contain factual information and interpretation of subsurface conditions between samples. The stratum indicated on these logs represents the approximate boundary between earth units and the transition may be gradual. The logs show subsurface conditions at the dates and locations indicated, and may not be representative of subsurface conditions at other locations and times.

Identification of the soils encountered during the subsurface exploration was made using the field identification procedure of the Unified Soils Classification System (ASTM D2488). A legend indicating the symbols and definitions used in this classification system and a legend defining the terms used in describing the relative compaction, consistency or firmness of the soil are attached in this appendix. Bag samples of the major earth units were obtained for laboratory inspection and testing, and the in-place density of the various strata encountered in the exploration was determined

The exploratory borings were located in the field by using cultural features depicted on a preliminary site plan provided by the client. Each location should be considered accurate only to the scale and detail of the plan utilized.

The exploratory borings were backfilled with native soil cuttings, compacted, and patched where appropriate.

UNIFIED SOIL CLASSIFICATION SYSTEM

No. 200 U.S. Standard Sieve is the smallest particle visible	Coarse-grained soils >1/2 of materials is larger than #200 sieve	GRAVELS are more than half of coarse fraction larger than #4 sieve	Clean Gravels (less than 5% fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
			Gravels with fines	GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
		SANDS are more than half of coarse fraction larger than #4 sieve	Clean Sands (less than 5% fines)	GM	Silty Gravels, poorly-graded gravel-sand-silt mixtures
			Sands with fines	GC	Clayey Gravels, poorly-graded gravel-sand-clay mixtures
	Fine-grained Soils >1/2 of materials is smaller than #200 sieve	SILTS AND CLAYS Liquid Limit Less than 50		SW	Well-graded sands, gravelly sands, little or no fines
				SP	Poorly-graded sands, gravelly sands, little or no fines
				SM	Silty Sands, poorly-graded sands-gravel-clay mixtures
				SC	Clayey Sands, poorly-graded sand-gravel-silt mixtures
				ML	Inorganic clays of low to med plasticity, gravelly, sandy, silty, or lean clays
		SILTS AND CLAYS Liquid Limit Greater than 50		CL	Inorganic clays of low to med plasticity, gravelly, sandy, silty, or lean clays
				OL	Organic silts and clays of low plasticity
				MH	Inorganic silts, micaceous or diatomaceous fine sands or silts
				CH	Inorganic clays of high plasticity, fat clays
				OH	Organic silts and clays of medium to high plasticity
Highly Organic Soils				PT	Peat, humus swamp soils with high organic content

GRAIN SIZE

Description		Sieve Size	Grain Size	Approximate Size
Boulders		>12”	>12”	Larger than basketball-sized
Cobbles		3”- 12”	3”- 12”	Fist-sized to basketball-sized
Gravel	Coarse	¾”- 3”	¾”- 3”	Thumb-sized
	Fine	#4 - ¾”	0.19” - 0.75”	Peat-sized to thumb-sized
Sand	Coarse	#10 - #4	0.079” - 0.19”	Rock salt-sized to pea-sized
	Medium	#40 - #10	0.017” - 0.079”	Sugar-sized to rock salt-sized
	Fine	#200 - #40	0.0029” - 0.017”	Flour-sized to sugar-sized
Fines		Passing #200	<0.0029”	Flour-sized or smaller

SIZE PROPORTION
Trace – Less than 5%
Few – 5% to 10%
Little – 15% to 20%
Some – 30% to 45%
Mostly – 50% to 100%

MOISTURE CONTENT
Dry – Absence of moisture
Moist – Damp but not visible
Wet – Visible free water

CONSISTENCY FINE GRAINED SOILS

Apparent Density	SPT (Blows/Foot)	Mod CA Sampler (Blows/Foot)
Very Soft	<2	<3
Soft	2-4	3-6
Firm	5-8	7-12
Stiff	9-15	13-25
Very Stiff	16-30	26-50
Hard	>30	>50

RELATIVE DENSITY COARSE GRAINED SOILS

Apparent Density	SPT (Blows/Foot)	Mod CA Sampler (Blows/Foot)
Very Loose	<4	<5
Loose	4-10	5-12
Medium Dense	11-30	13-35
Dense	31-50	36-60
Very Dense	<50	<60

BORING NO. B-1

Logged by: BMH
Date Drilled: 8/5/2015
Method of Drilling: 8-inch diameter hollow-stem auger
Elevation: N/A

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1			▨			**ALLUVIUM:** Silty sand (SM), olive gray, moist, loose upper one foot	SE = 9 R-Value = 53
2			▨				
3			▨				
4			▨				
5			▨				
6						Boring terminated at 5 feet No groundwater Backfilled 8/5/15	
7							
8							
9							
10							
11							
12							
13							
14							
15							
16							
17							
18							
19							
20							
21							
22							
23							
24							
25							
26							
27							
28							
29							
30							

BORING NO. B-2

Logged by: BMH

Date Drilled: 8/5/2015

Method of Drilling: 8-inch diameter hollow-stem auger

Elevation: N/A

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1						**ALLUVIUM:** Silty sand (SM), light olive gray, moist, loose upper one foot	
2							
3	23	SPT			14.4	Silty sand, (SM), dense	
4							
5						Silty sand (SM), blue gray and olive gray, moist, medium dense	
6	17	SPT			21.3		
7							
8							
9							
10							
11	13	SPT			20.2	Silty sand (SM), gray, moist, medium dense	
12							
13							
14							
15							
16	11	SPT			8.6	Sand, medium-grained (SP), light gray, moist, medium dense	
17							
18							
19							
20							
21							
22							
23							
24							
25						Silty sand (SM), tan, moist, medium dense, contains 1/4" subrounded pebbles	
26	18	SPT			13.3		
27							
28							
29							
30							

BORING NO. B-2

Logged by: BMH
Method of Drilling: 8-inch diameter hollow-stem auger
Date Drilled: 8/5/2015
Elevation: N/A

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
31						**ALLUVIUM:** Silty sand (SM), tan, moist, medium dense, contains 1/4" subrounded pebbles	
32							
33							
34							
35							
36	21	SPT			20.2	Interlayered fine sand (SP), and silt (ML), yellow gray, moist, medium dense hard	
37							
38							
39							
40						Fine sand (SP), yellow gray, moist, medium dense	
41	22	SPT			15.4		Sieve Analysis
42							
43							
44							
45							
46	21	SPT			17.7	Fine sand (SP), orange and yellow gray, moist, medium dense	
47							
48							
49							
50						Sand, (SW), fine to coarse, tan to yellow, moist, very dense, contains 1/4" subrounded pebbles	
51	48	SPT			3.2		
52						Boring terminated at 51.5 feet No groundwater noted Backfilled 8/5/15	
53							
54							
55							
56							
57							
58							
59							
60							

BORING NO. B-3

Logged by: BMH
Date Drilled: 8/5/2015
Method of Drilling: 8-inch diameter hollow-stem auger
Elevation: N/A

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1			▨			**ALLUVIUM:** Silty sand (SM), medium brown, moist, loose upper foot	Sieve Analysis Max = 119.7 pcf Opt Moist = 10.5% EI = 31 (Low) pH=7.7 Sulfates = 1,012 ppm Chlorides = 844 ppm Resist. = 300 ohm-cm
2			▨				
3							
4							
5							
6	11	CAL				Sand, fine, (SP), yellow brown, medium dense, moist	Direct Shear c=834 psf / ϕ=29.7°
7							
8							
9							
10							
11	8	CAL				Sandy silt, olive brown, moist, firm	Consolidation
12							
13							
14							
15							
16	39	CAL		104	3.5	Sand, medium-grained, (SW), yellow brown, moist, dense	
17							
18							
19							
20							
21	11	SPT			15.7	Interbedded olive gray silt (ML), orange gray fine sand (SP), moist, medium dense	
22							
23							
24						Boring terminated at 21.5 feet No groundwater encountered Backfilled on 8/5/15	
25							
26							
27							
28							
29							
30							

BORING NO. B-4

Logged by: BMH
Method of Drilling: 8-inch diameter hollow-stem auger
Date Drilled: 8/5/2015
Elevation: N/A

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1			▨			**ALLUVIUM:** Silty sand (SM), medium brown, moist, loose upper foot	SE = 32
2			▨				
3			▨				
4			▨				
5			▨				
6						Boring terminated at 5 feet No groundwater encountered Backfilled on 8/5/15	
7							
8							
9							
10							
11							
12							
13							
14							
15							
16							
17							
18							
19							
20							
21							
22							
23							
24							
25							
26							
27							
28							
29							
30							

BORING NO. B-5

Logged by: BMH
Date Drilled: 8/5/2015
Method of Drilling: 8-inch diameter hollow-stem auger
Elevation: N/A

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1						**ALLUVIUM:** Silty sand (SM), medium brown, moist, loose upper foot	
2							
3						medium dense	
4							
5							
6	33	CAL		119.4	11.3	Silty sand, (SM), olive gray, moist, dense	
7							
8							
9							
10							
11	10	CAL		99.3	15.4	Silty sand, (SM), olive gray and tan, moist, dense	
12							
13							
14							
15							
16	12	SPT			23.7	Interlayered olive gray silt (ML) and yellow brown fine sand (SP), moist, medium dense and firm	
17							
18							
19							
20							
21	2	SPT			28.5	Silt, olive brown, wet, soft	
22						Boring terminated at 21.5 feet No groundwater encountered Backfilled on 8/5/15	
23							
24							
25							
26							
27							
28							
29							
30							

BORING NO. B-6

Logged by: BMH

Date Drilled: 8/5/2015

Method of Drilling: 8-inch diameter hollow-stem auger

Elevation: N/A

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1							
2							
3							
4							
5							
6	13	CAL		104.2	13.1	**ALLUVIUM:** Silty sand (SM) and sandy silt (ML), olive brown, moist, medium dense	
7							
8							
9							
10							
11	7	CAL		89.6	18.5	Silt, olive gray, moist, soft, porous	
12							
13							
14							
15							
16	22	CAL		103.5	10.2	Sand, tan, medium grained (SW), moist, dense	
17							
18							
19							
20							
21	18	CAL		106.4	15.6	Silty sand (SM) and sandy silt (ML), olive gray, moist, medium dense	
22						Boring terminated at 21.5 feet No groundwater encountered Backfilled on 8/5/15	
23							
24							
25							
26							
27							
28							
29							
30							

BORING NO. B-7

Logged by: BMH
Method of Drilling: 8-inch diameter hollow-stem auger
Date Drilled: 8/5/2015
Elevation: N/A

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1			▨				
2			▨			<u>ALLUVIUM:</u> Silty sand (SM) medium brown, moist, medium dense	
3			▨				
4			▨				
5			▨				
6	12	CAL		107.7	7.8	Silty sand, yellow gray, moist, medium dense	
7							
8							
9							
10							
11	18	CAL		100.5	14.0	Silty sand, (SM), yellow gray, moist, medium dense	
12							
13							
14							
15							
16	38	CAL		112.1	8.2	Silty sand (SM), yellow gray, moist, dense	
17							
18							
19							
20							
21	37	CAL		109.7	12.2	Sand, fine to medium-grained (SW), yellow gray and orange, moist, dense	
22						Boring terminated at 21.5 feet No groundwater encountered Backfilled on 8/5/15	
23							
24							
25							
26							
27							
28							
29							
30							

BORING NO. B-8

Logged by: BMH

Date Drilled: 8/5/2015

Method of Drilling: 8-inch diameter hollow-stem auger

Elevation: N/A

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1			▨				
2			▨			ALLUVIUM: Fine sand (SP) yellow gray, moist, medium dense	
3			▨				
4			▨				
5			▨				
6	15	CAL		108.4	8.8	Silty sand, yellow gray, moist, medium dense	
7							
8							
9							
10							
11	18	CAL		112.8	12.7	Silty sand, (SM), yellow gray, moist, medium dense	
12							
13							
14							
15							
16	38	CAL		101.5	3.5	Silty sand (SM), yellow gray, moist, dense	
17							
18							
19							
20							
21	37	SPT			20.0	Sand, fine to medium-grained (SW), yellow gray and orange, moist, dense	
22						Boring terminated at 21.5 feet No groundwater encountered Backfilled on 8/5/15	
23							
24							
25							
26							
27							
28							
29							
30							

BORING NO. B-9

Logged by: BMH
Date Drilled: 8/5/2015
Method of Drilling: 8-inch diameter hollow-stem auger
Elevation: N/A

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1						**ALLUVIUM:** Silty sand (SM), gray brown, moist, medium dense	
2							
3							
4							
5							
6	27	CAL		114.3	13.4	Silt (ML), olive gray, moist, hard	
7							
8							
9							
10		SPT				Silt (ML), gray, moist, firm	
11	8				20.2		
12							
13							
14							
15						Silty sand (SM), olive yellow, moist, medium dense	
16	20	CAL		99.1	13.9		
17							
18							
19							
20						Silt, (ML), olive gray, wet, soft	
21	5	CAL		95.3	27.2		
22						Boring terminated at 21 feet No groundwater Backfilled 8/5/15	
23							
24							
25							
26							
27							
28							
29							
30							

BORING NO. B-10

Logged by: BMH
Date Drilled: 8/5/2015
Method of Drilling: 8-inch diameter hollow-stem auger
Elevation: N/A

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1			▨			**ALLUVIUM:** Silty sand (SM), medium brown, moist, medium dense	
2			▨				
3			▨				
4			▨				
5			▨				
6	17	CAL		104.2	9.5	Silty sand (SM), olive gray, moist, medium dense	
7							
8							
9							
10						Silt (ML), pale gray, wet, firm	
11	7	CAL		97.0	24.7		
12							
13							
14							
15						Silty sand (SM), yellow gray, moist, dense	
16	34	CAL		111.4	12.3		
17							
18							
19							
20						Sand, coarse-grained (SP), gray, wet, medium dense	
21	24	CAL			16.1	(disturbed sample)	
22						Boring terminated at 21 feet No groundwater Backfilled 8/5/15	
23							
24							
25							
26							
27							
28							
29							
30							

BORING NO. B-11

Logged by: BMH
Date Drilled: 8/5/2015
Method of Drilling: 8-inch diameter hollow-stem auger
Elevation: N/A

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1			▨			**ALLUVIUM:** Silty sand (SM), medium brow n, moist, medium dense	
2			▨				
3			▨				
4			▨				
5			▨				
6	16	CAL		102.3	6.2		
7							
8							
9							
10						Silt (ML),olive gray, w et, firm	
11	19	CAL		87.3	27.0		
12							
13							
14							
15							
16	5	SPT			36.5	Silt (ML), olive brow n, w et, soft	
17							
18							
19							
20						Sand, fine-grained (SP), gray, moist, dense	
21	29	SPT			5.7		
22						Boring terminated at 21.5 feet No groundw ater Backfilled 8/5/15	
23							
24							
25							
26							
27							
28							
29							
30							

LOG OF BORING

BORING NO. B-12

Logged by: BMH
Date Drilled: 8/6/2015
Method of Drilling: 8-inch diameter hollow-stem auger
Elevation: N/A

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1						**ALLUVIUM:** Silty sand (SM), medium brown, moist, medium dense	
2							
3							
4						Silt (ML), olive gray, moist, firm	
5		SPT					
6	13	SPT			17.2		
7							
8							
9							
10		SPT				Sandy silt (ML),olive gray and olive brown, wet, firm	
11	12	SPT			8.8		
12							
13							
14							
15		SPT					
16	15	SPT			27.3	Silt (ML), olive gray and brown, wet, firm	
17							
18							
19							
20		SPT					
21	12	SPT			16.8	Silty sand, (SM), olive brown, wet, medium dense	Sieve Analysis
22							
23							
24							
25		SPT					
26	20	SPT			23.3	Silt (ML), olive gray, wet, hard	
27							
28							
29							
30						Silty sand to sandy silt (SM/ML), olive yellow, wet, medium dense to hard	

BORING NO. B-12

Logged by: BMH
Date Drilled: 8/6/2015
Method of Drilling: 8-inch diameter hollow-stem auger
Elevation: N/A

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
31	13	SPT			18.2	Silty sand to sandy silt (SM/ML), olive yellow, wet, medium dense to firm	
32							
33							
34							
35							
36	19	SPT			25.8	Sandy silt to silty sand (ML/SM), olive yellow, wet, medium dense to hard	
37							
38							
39							
40						Silty sand (SM), yellow brown, wet, dense	
41	20	SPT			29.4		
42							
43							
44							
45							
46	23	SPT			20.9	Silty sand (SM), yellow gray, wet, medium dense	
47							
48							
49							
50							
51	16	SPT			19.2	Silty sand, (SM), medium brown, wet, medium dense	
52						Boring terminated at 51.5 feet No groundwater noted Backfilled 8/6/15	
53							
54							
55							
56							
57							
58							
59							
60							

BORING NO. B-13

Logged by: BMH
Date Drilled: 8/6/2015
Method of Drilling: 8-inch diameter hollow-stem auger
Elevation: N/A

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1						**ALLUVIUM:** Silty sand (SM), dark brown, moist, medium dense	
2							
3							
4							
5							
6	20	CAL		102.3	13.4	Silt, olive gray, moist, firm	
7							
8							
9							
10							
11	15	CAL		103.9	19.6	Silt (ML) and silty sand (SM), olive gray to dark brown, moist, medium dense firm	
12							
13							
14							
15							
16	14	CAL		92.3	24.7	Silt (ML), olive gray, wet, firm	
17							
18							
19							
20						Interlayered silty sand (SM), yellow brown, and silt (ML), olive gray, wet, hard	
21	19	CAL		97.6	25.2		
22						Boring terminated at 21 feet No groundwater noted Backfilled 8/6/15	
23							
24							
25							
26							
27							
28							
29							
30							

BORING NO. B-14

Logged by: BMH
Date Drilled: 8/6/2015
Method of Drilling: 8-inch diameter hollow-stem auger
Elevation: N/A

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1						**ALLUVIUM:** Silty sand (SM), dark brown, moist, medium dense	
2							
3							
4							
5							
6	26	CAL		105.6	8.1	Silt, olive gray, moist, firm to hard	
7							
8							
9							
10							
11	35	CAL		100.2	8.5	Silt (ML), olive gray, moist,hard firm	
12							
13							
14							
15						Boring terminated at 14 feet due to drilling difficulties No groundwater noted Backfilled 8/6/15	
16							
17							
18							
19							
20							
21							
22							
23							
24							
25							
26							
27							
28							
29							
30							

PROJECT NO. 4815A04

MTGi

LOG OF BORING

BORING NO. B-15

Logged by: BMH **Date Drilled:** 8/6/2015
Method of Drilling: 8-inch diameter hollow-stem auger **Elevation:** N/A

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1			▨			**ALLUVIUM:** Silty sand (SM), medium brown and gray brown, moist, medium dense	Sieve Analysis EI = 0 (Very Low) Max = 125.0 pcf Opt Moist = 10.5% pH = 7.8 Sulfates = 333 ppm Chlorides = 334 Resist=1,200 ohm-cm
2			▨				
3			▨				
4			▨				
5							
6	39	CAL		109.0	15.7	Silty sand (SM) and sandy silt (ML), olive gray, moist, dense and hard	
7							
8							
9							
10							
11	24	CAL				Silt (ML), olive gray , moist, firm firm	Direct Shear c=501 psf / ϕ=37.8°
12							
13							
14							
15							
16	24	CAL				Silty sand, (SM), olive gray with orange oxidation, moist, medium dense	Consolidation
17							
18							
19							
20							
21	26	CAL		88.5	32.4		
22						Boring terminated at 21 feet No groundwater noted Backfilled 8/6/15	
23							
24							
25							
26							
27							
28							
29							
30							

BORING NO. B-16

Logged by: BMH
Date Drilled: 8/6/2015
Method of Drilling: 8-inch diameter hollow-stem auger
Elevation: N/A

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1						**ALLUVIUM:** Silty sand (SM), medium brown and gray brown, moist medium dense	
2							
3							
4							
5							
6	22	CAL		84.8	12.4	Silty sand (SM), olive gray, moist, medium dense; porous	
7							
8							
9							
10							
11	21	CAL		98.9	12.8	Silty sand (SM) and sandy silt (ML), olive gray , moist, medium dense and firm firm	
12							
13							
14							
15							
16	24	CAL		100.7	22.1	Silty sand (SM), yellow gray and sandy silt (ML), olive gray , moist, medium dense and firm	
17							
18							
19							
20							
21	21	CAL		103.4	8.0	Interbedded silty sand (SM), brown, with 1/8" subrounded pebbles, and fine sand, (SP), gray; moist, medium dense	
22						Boring terminated at 21 feet No groundwater noted Backfilled 8/6/15	
23							
24							
25							
26							
27							
28							
29							
30							

BORING NO. B-17

Logged by: BMH **Date Drilled:** 8/5/2015

Method of Drilling: 8-inch diameter hollow-stem auger **Elevation:** N/A

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1			▨			**ALLUVIUM:** Silty sand (SM), medium brown and olive brown, moist, loose upper foot	SE = 14 R-Value = 49
2			▨				
3			▨				
4			▨				
5			▨				
6						Boring terminated at 5 feet No groundwater encountered Backfilled on 8/5/15	
7							
8							
9							
10							
11							
12							
13							
14							
15							
16							
17							
18							
19							
20							
21							
22							
23							
24							
25							
26							
27							
28							
29							
30							

BORING NO. B-18

Logged by: BMH
Date Drilled: 8/5/2015
Method of Drilling: 8-inch diameter hollow-stem auger
Elevation: N/A

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1			▨			**ALLUVIUM:** Silty sand (SM), medium brown and olive brown, moist, loose upper foot	SE = 22
2			▨				
3			▨				
4			▨				
5			▨				
6						Boring terminated at 5 feet No groundwater encountered Backfilled on 8/5/15	
7							
8							
9							
10							
11							
12							
13							
14							
15							
16							
17							
18							
19							
20							
21							
22							
23							
24							
25							
26							
27							
28							
29							
30							

BORING NO. B-19

Logged by: BMH

Date Drilled: 8/6/2015

Method of Drilling: 8-inch diameter hollow-stem auger

Elevation: N/A

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1						**ALLUVIUM:** Silty sand (SM), medium brown and gray brown, moist medium dense	
2							
3							
4							
5							
6	25	CAL		94.0	21.6	Silty sand (SM), medium brown, silt (ML), olive gray, moist, medium dense and stiff	
7							
8							
9							
10							
11	17	CAL		109.7	16.0	Silt (ML), olive gray, moist, firm, porous	
12							
13							
14							
15							
16	18	CAL		97.4	24.2	Pale olive gray silt, (ML), wet, firm	
17							
18							
19							
20						Silt, olive yellow, saturated, firm	
21	22	CAL			30.6		
22						Boring terminated at 21 feet No groundwater noted Backfilled 8/6/15	
23							
24							
25							
26							
27							
28							
29							
30							

BORING NO. B-20

Logged by: BMH
Date Drilled: 8/6/2015
Method of Drilling: 8-inch diameter hollow-stem auger
Elevation: N/A

DEPTH (FT)	BLOWS PER FT	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE (%)	DESCRIPTION	LAB TESTS
1						**ALLUVIUM:** Silty sand (SM), medium brown and gray brown, moist medium dense	
2							
3							
4							
5							
6							
7							
8							
9						Silt (ML), olive gray, moist to wet, firm	
10							
11							
12							
13							
14							
15							
16						Boring terminated at 21 feet; Percolation test hole No groundwater noted Backfilled 8/6/15	
17							
18							
19							
20							
21							
22							
23							
24							
25							
26							
27							
28							
29							
30							

APPENDIX C

LABORATORY TEST PROCEDURES

APPENDIX C

LABORATORY TESTING PROCEDURES

1. Classification

   Soils were classified visually, generally according to the Unified Soil Classification System. Classification tests were also completed on representative samples in accordance with ASTM D422 for Grain Size. The test results are attached to this appendix.

2. Maximum Density

   Maximum density tests were performed on a representative bag sample of the near surface soils in accordance with ASTM D1557.

3. Direct Shear

   Direct Shear Tests were performed on in-place and remolded samples of site soils in accordance with ASTM D3080.

4. Consolidation

   Consolidation tests were performed on representative, relatively undisturbed samples of the underlying soils to determine compressibility characteristics in accordance with ASTM D2435. Test results are presented in this appendix.

5. R-Value Testing

   R-Value testing was completed in substantial compliance with Caltrans Test Method 301. Graphical plots of our tests are included in this appendix.

6. Expansion Index

   Expansion Index testing was completed in accordance with the standard test method ASTM D4829. Test results are presented below.

Sample Location	Expansion Index	Expansion Classification
B-3 @ 0-2’	31	Low
B-15 @ 0-5’	0	Very Low

7. Corrosion

Chemical testing was performed on representative samples to determine the corrosion potential of the onsite soils. Testing consisted of pH, chlorides (CTM 422), soluble sulfates (CTM 417), and resistivity (CTM 643). Test results are as follows:

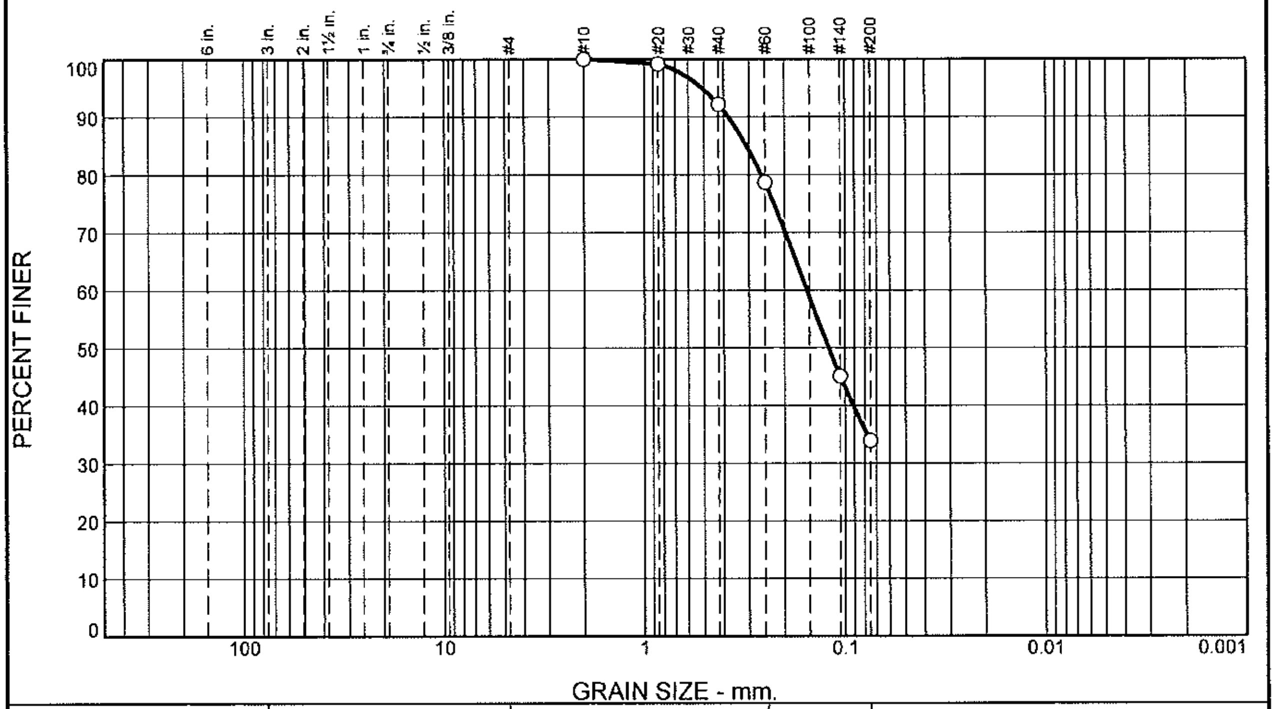
Sample Location	pH	Chlorides (ppm)	Sulfates (ppm)	Resistivity (ohm-cm
B-3 @ 0-2’	7.7	844	1,012	300
B-15 @ 0-5’	7.8	334	333	1,200

8. Sand Equivalence

The sand equivalence of representative soils was determined using the standard test methods of the American Society for Testing and Materials (ASTM D2419). Test results are presented below.

Sample Location	Sand Equivalence
B-1 @ 0-5 ft	9
B-4 @ 0-5 ft	32
B-17 @ 0-5 ft	14
B-18 @ 0-5 ft	22

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	7.8	58.3	33.9	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#10	100.0		
#20	99.2		
#40	92.2		
#60	78.7		
#140	45.1		
#200	33.9		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= 0.3795 D_{85}= 0.3080 D_{60}= 0.1548
D_{50}= 0.1209 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

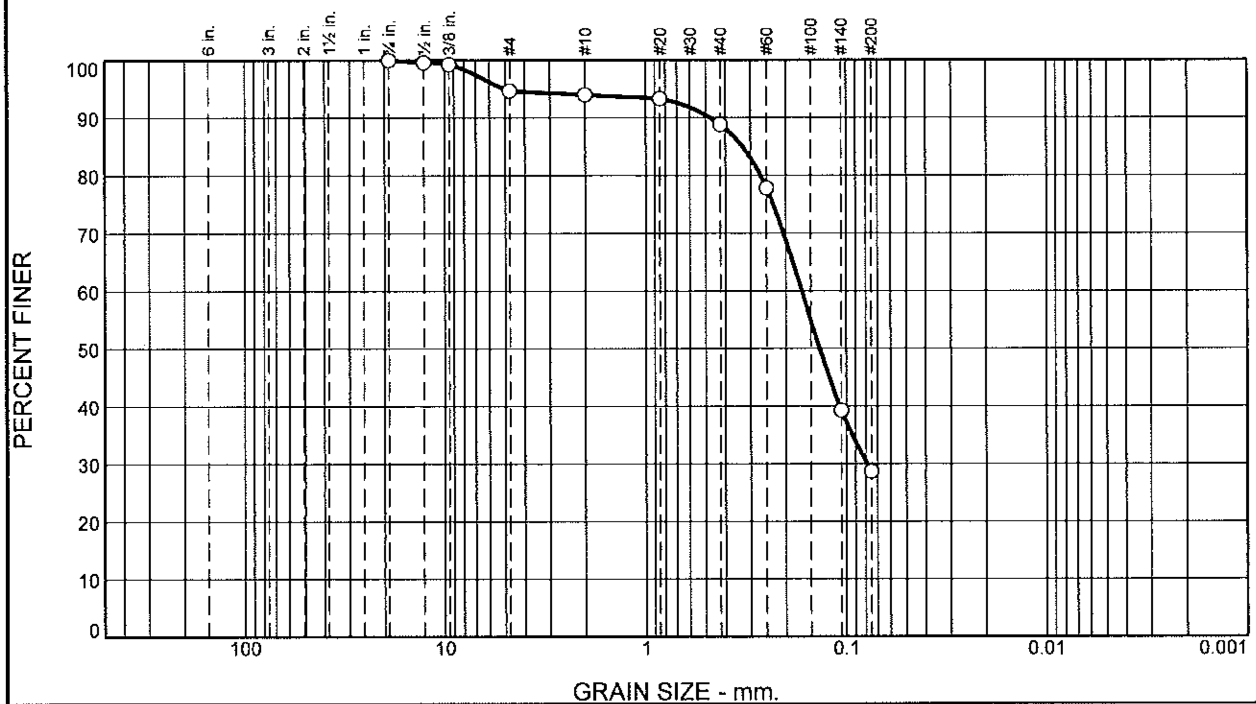
Date Received: 8/10/15 **Date Tested:** 8/14/15
Tested By: JH
Checked By: IC
Title: 8/27/15

Sample Number: B12 **Depth:** 20' **Date Sampled:** 8/10/15

MTGL, Inc.
San Diego, CA

Client:
Project: CAPROCK - TADEMA ONTARIO
Project No: 4815-A04 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	5.3	0.7	5.1	60.3	28.6	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
.75	100.0		
.5	99.6		
.375	99.2		
#4	94.7		
#10	94.0		
#20	93.3		
#40	88.9		
#60	77.8		
#140	39.3		
#200	28.6		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= **LL=** **PI=**

Classification

USCS (D 2487)= **AASHTO (M 145)=**

Coefficients

D_{90}= 0.4739 D_{85}= 0.3282 D_{60}= 0.1664
D_{50}= 0.1357 D_{30}= 0.0788 D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 8/10/15 **Date Tested:** 8/14/15
Tested By: EP
Checked By: IC
Title: ENGINEER

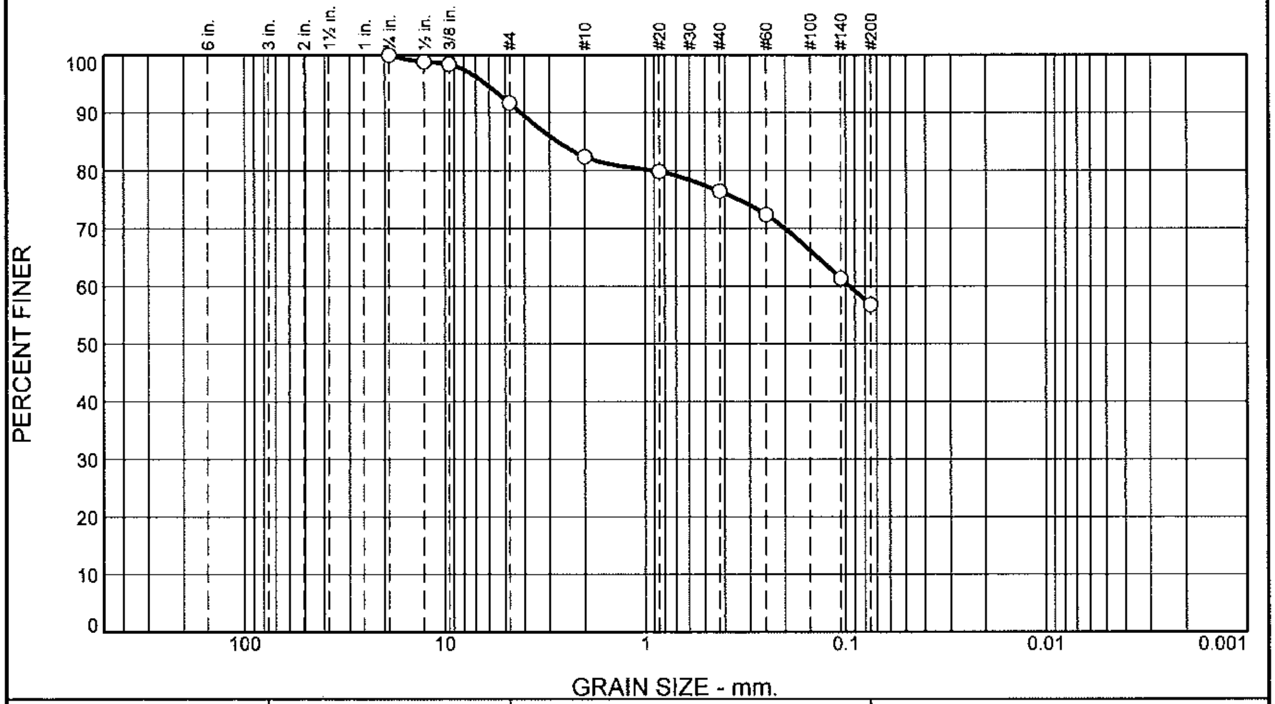
Sample Number: B15 **Depth:** 0-5' **Date Sampled:** 8/10/15

MTGL, Inc.

San Diego, CA

Client:
Project: CAPROCK - TADEMA ONTARIO
Project No: 4815-A04 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	8.3	9.3	6.0	19.6	56.8	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
.75	100.0		
.5	98.8		
.375	98.4		
#4	91.7		
#10	82.4		
#20	79.8		
#40	76.4		
#60	72.4		
#140	61.3		
#200	56.8		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= **LL=** **PI=**

Classification

USCS (D 2487)= **AASHTO (M 145)=**

Coefficients

D_{90}= 4.1617	D_{85}= 2.7190	D_{60}= 0.0959
D_{50}=	D_{30}=	D_{15}=
D_{10}=	C_u=	C_c=

Remarks

Date Received: 8/10/15 **Date Tested:** 8/14/15

Tested By: EP

Checked By: IC

Title: ENGINEER

Sample Number: B3 **Depth:** 0-2' **Date Sampled:** 8/10/15

MTGL, Inc.

San Diego, CA

Client:

Project: CAPROCK - TADEMA ONTARIO

Project No: 4815-A04 **Figure**

COMPACTION TEST REPORT

Project No.: 4815-A04 **Date:** 8/14/15
Project: CAPROCK - TADEMA ONTARIO
Client:
Sample Number: B15 **Depth:** 0-5'

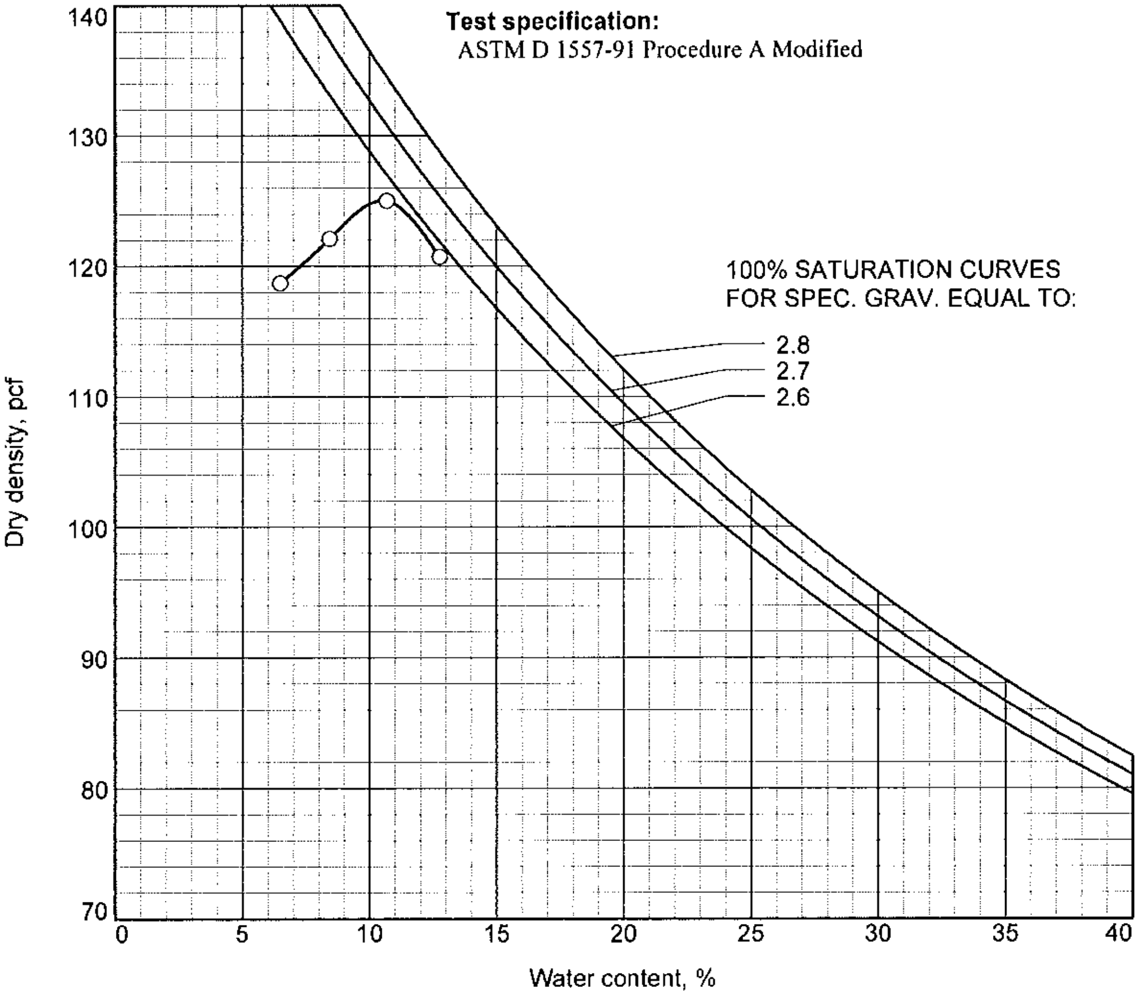
Remarks:

MATERIAL DESCRIPTION

Description:

Classifications - **USCS:** **AASHTO:**
Nat. Moist. = **Sp.G. =**
Liquid Limit = **Plasticity Index =**
% < No.200 = 28.6 %

TEST RESULTS
Maximum dry density = 125.0 pcf
Optimum moisture = 10.5 %



COMPACTION TEST REPORT

Project No.: 4815-A04
Date: 8/14/15
Project: CAPROCK - TADEMA ONTARIO
Client:
Sample Number: B3 **Depth:** 0-2'

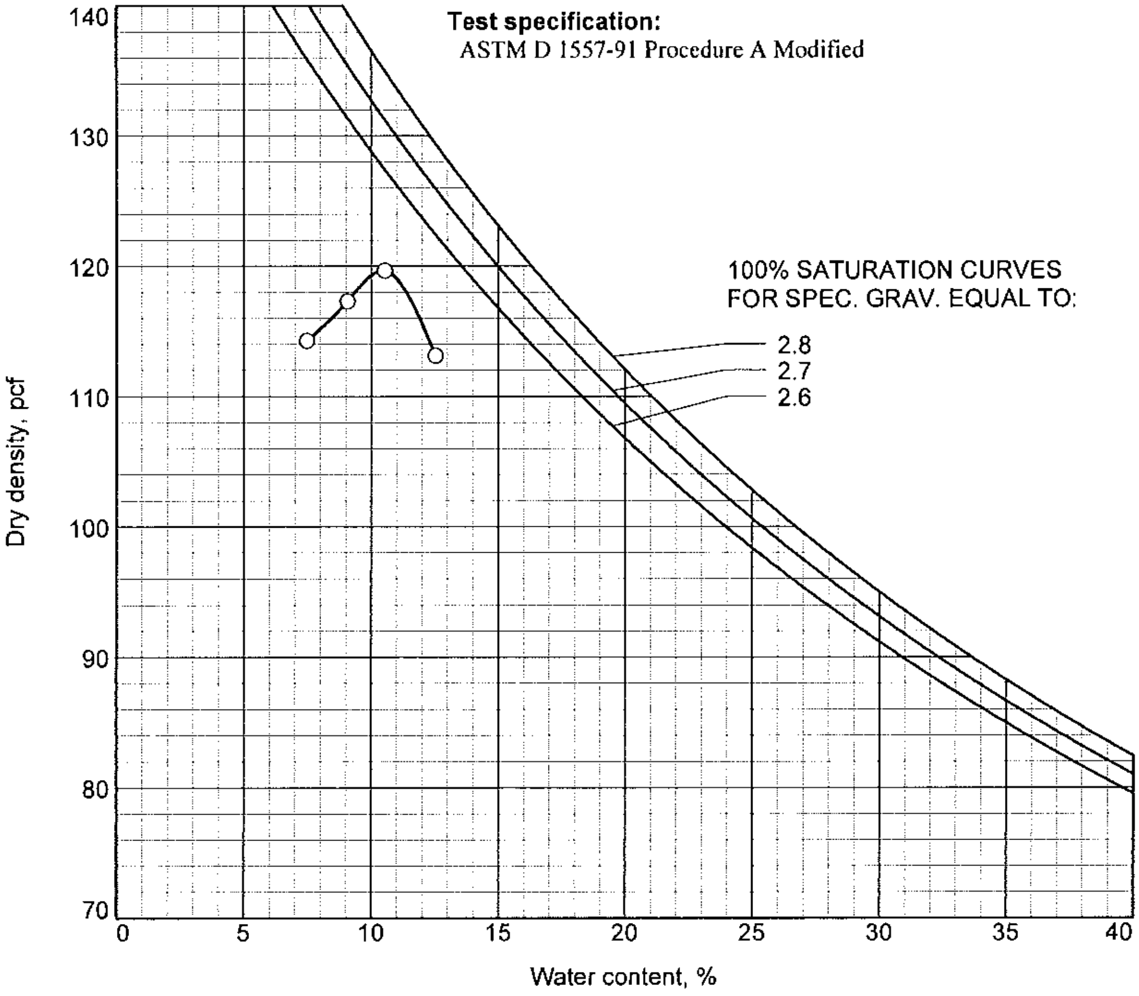
Remarks:

MATERIAL DESCRIPTION

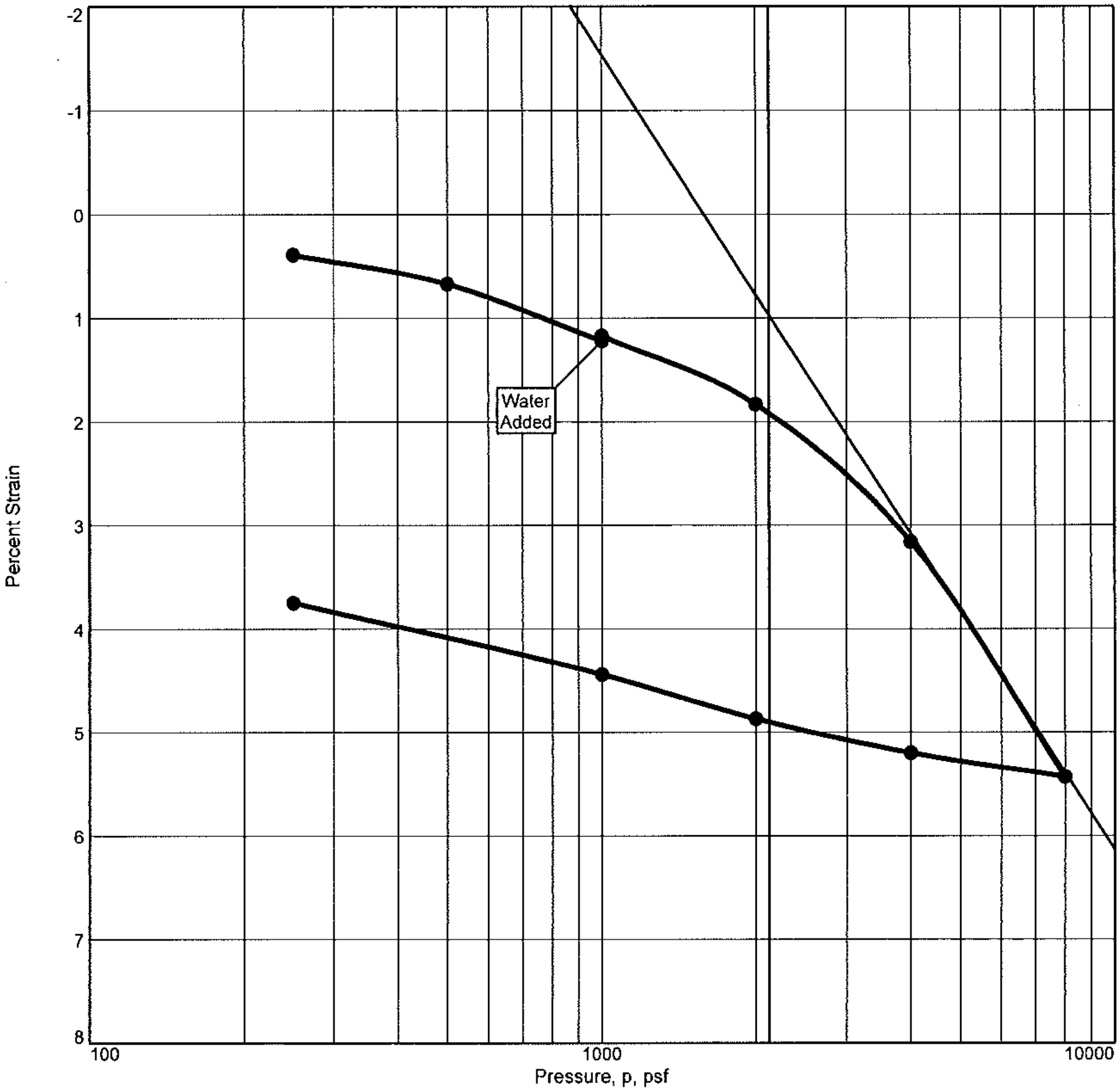
Description:

Classifications - **USCS:** **AASHTO:**
Nat. Moist. = **Sp.G. =**
Liquid Limit = **Plasticity Index =**
% < No.200 = 56.8 %

TEST RESULTS
Maximum dry density = 119.7 pcf
Optimum moisture = 10.5 %



CONSOLIDATION TEST REPORT



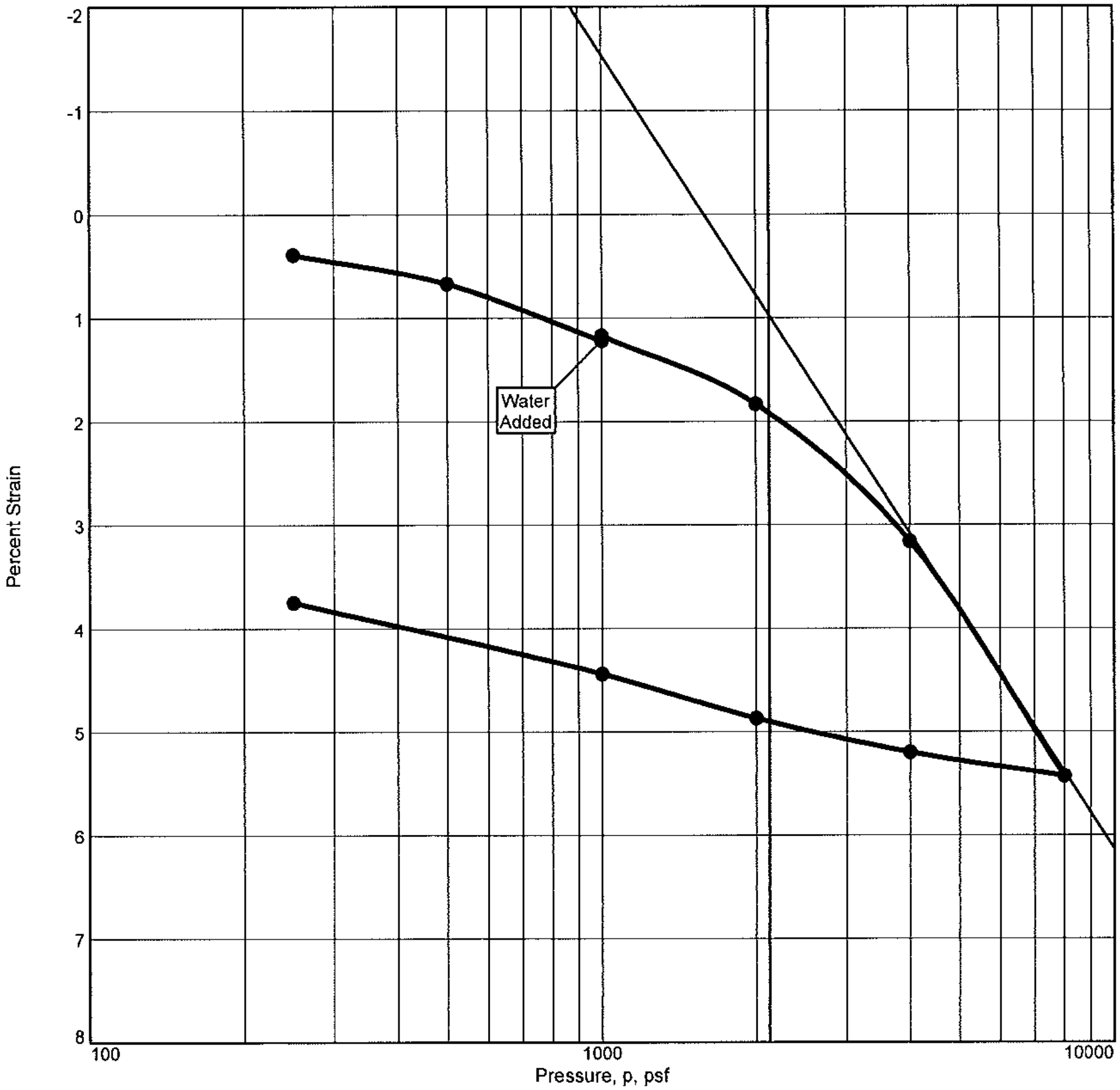
SUMMARY OF TEST RESULTS

	DRY DENSITY (pcf)	MOISTURE CONTENT, (%)	SATURATION (%)	VOID RATIO	SPECIFIC GRAVITY	OVERBURDEN (pcf)	P_C (pcf)	C_C	SWELL PRESS. (pcf)
INITIAL	93.3	27.5	94.3	0.774	2.65	1000	3029	0.14	1056
FINAL		26.6	99.8	0.707					

Depth: 11' **Sample Number:** 3
Material Description:
Remarks:

USCS: **AASHTO:**

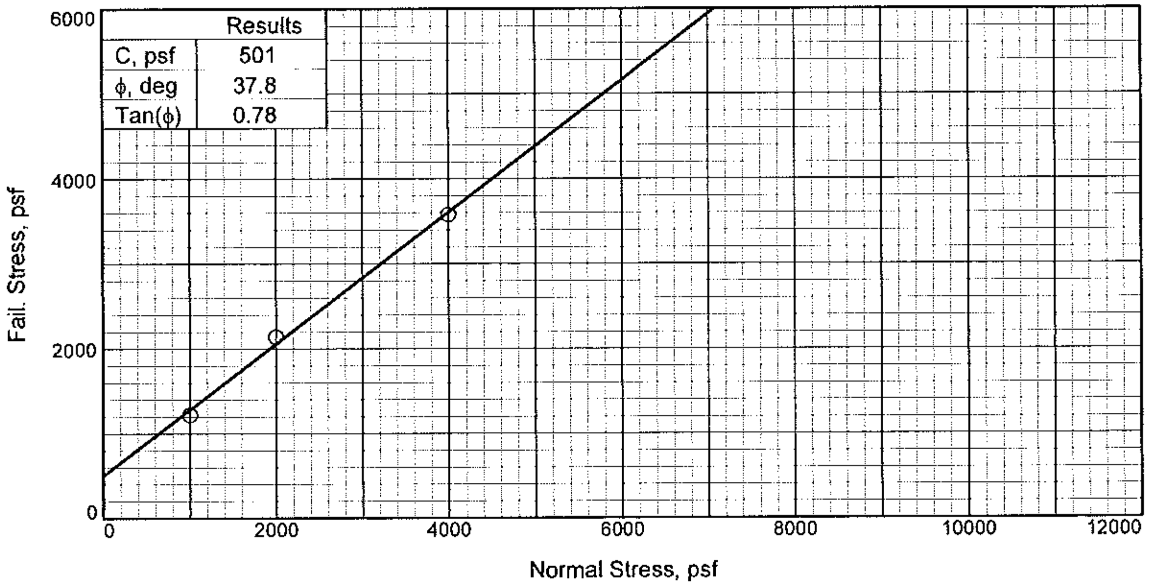
CONSOLIDATION TEST REPORT



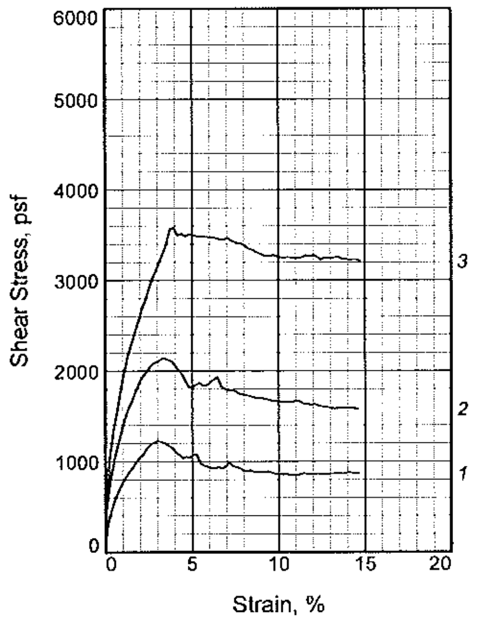
SUMMARY OF TEST RESULTS

	DRY DENSITY (pcf)	MOISTURE CONTENT, (%)	SATURATION (%)	VOID RATIO	SPECIFIC GRAVITY	OVERBURDEN (pcf)	P_C (pcf)	C_C	SWELL PRESS. (pcf)
INITIAL	93.3	27.5	94.3	0.774	2.65	1000	3029	0.14	1056
FINAL		26.6	99.8	0.707					

Depth: 11' **Sample Number:** 3
Material Description: **USCS:** **AASHTO:**
Remarks:



	Results
C, psf	501
ϕ, deg	37.8
Tan(ϕ)	0.78



	Sample No.	1	2	3
Initial	Water Content, %	19.0	19.7	19.3
	Dry Density, pcf	106.0	104.9	106.6
	Saturation, %	89.8	90.7	92.5
	Void Ratio	0.5605	0.5765	0.5520
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	20.1	20.5	19.0
	Dry Density, pcf	108.0	107.1	109.8
	Saturation, %	99.9	99.8	99.5
	Void Ratio	0.5324	0.5450	0.5070
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.98	0.98	0.97
	Normal Stress, psf	1000	2000	4000
	Fail. Stress, psf	1221	2138	3578
	Strain, %	3.0	3.4	4.0
	Ult. Stress, psf			
	Strain, %			
	Strain rate, in./min.	0.01	0.01	0.01

Sample Type:
Description:

Specific Gravity= 2.65
Remarks:

Figure ______

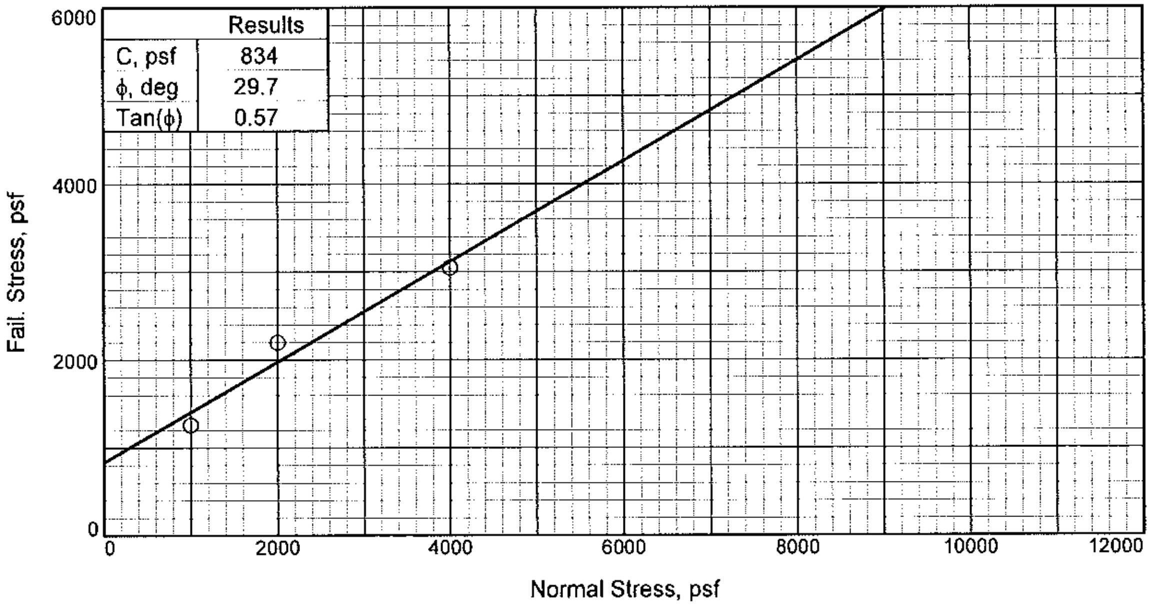
Client:

Project: CAPROCK - TADEMA ONTARIO

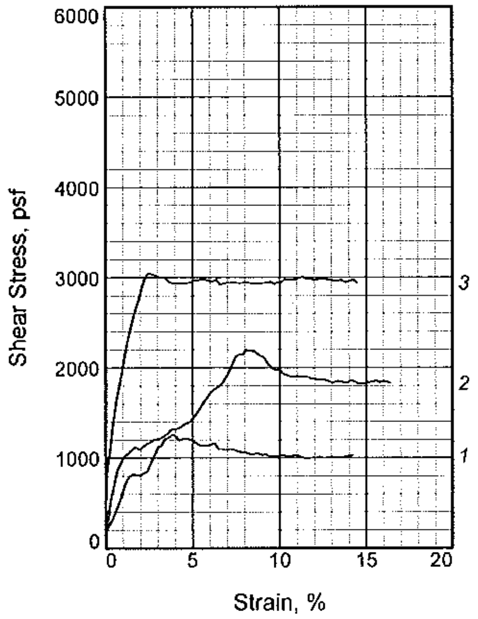
Sample Number: B15 **Depth:** 10'

Proj. No.: 4815-A04 **Date Sampled:**

DIRECT SHEAR TEST REPORT
MTGL, Inc.
San Diego, CA



	Results
C, psf	834
ϕ, deg	29.7
Tan(ϕ)	0.57



Sample No.		1	2	3
Initial	Water Content, %	15.7	14.5	16.7
	Dry Density, pcf	94.3	94.8	93.4
	Saturation, %	55.3	51.6	57.3
	Void Ratio	0.7537	0.7445	0.7708
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	27.9	26.8	27.4
	Dry Density, pcf	95.1	96.4	95.8
	Saturation, %	100.0	99.2	99.9
	Void Ratio	0.7396	0.7166	0.7265
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.99	0.98	0.97
Normal Stress, psf		1000	2000	4000
Fail. Stress, psf		1259	2198	3046
Strain, %		3.8	8.1	2.5
Ult. Stress, psf				
Strain, %				
Strain rate, in./min.		0.01	0.01	0.01

Sample Type:
Description:

Specific Gravity= 2.65
Remarks:

Figure ________

Client:

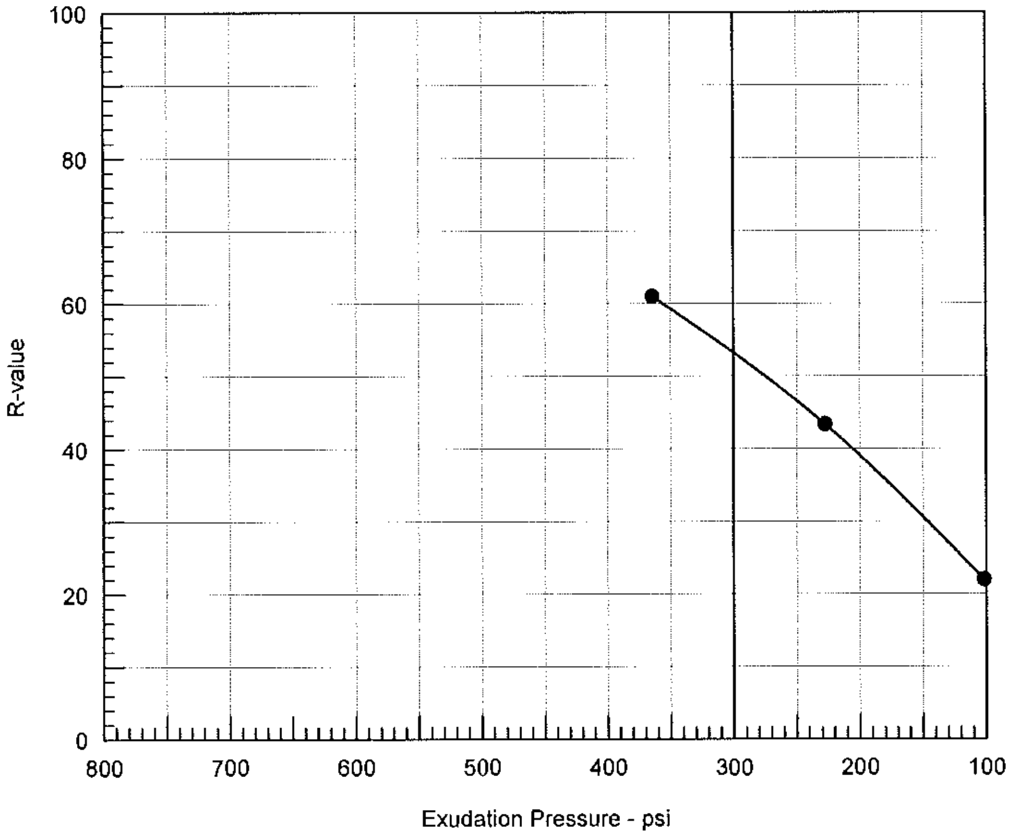
Project: CAPROCK - TADEMA ONTARIO

Sample Number: B3 **Depth:** 6'

Proj. No.: 4815-A04 **Date Sampled:** 8/10/15

DIRECT SHEAR TEST REPORT
MTGL, Inc.
San Diego, CA

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	350	105.8	18.5	0.91	100	2.70	102	19	22
2	350	106.0	16.3	1.06	60	2.52	228	43	43
3	350	108.3	14.9	1.21	40	2.50	364	61	61

Test Results	Material Description
R-value at 300 psi exudation pressure = 53	

Project No.: 4815-A04
Project: CAPROCK - TADEMA ONTARIO
Sample Number: B1 **Depth:** 0-5'

Date: 9/2/2015

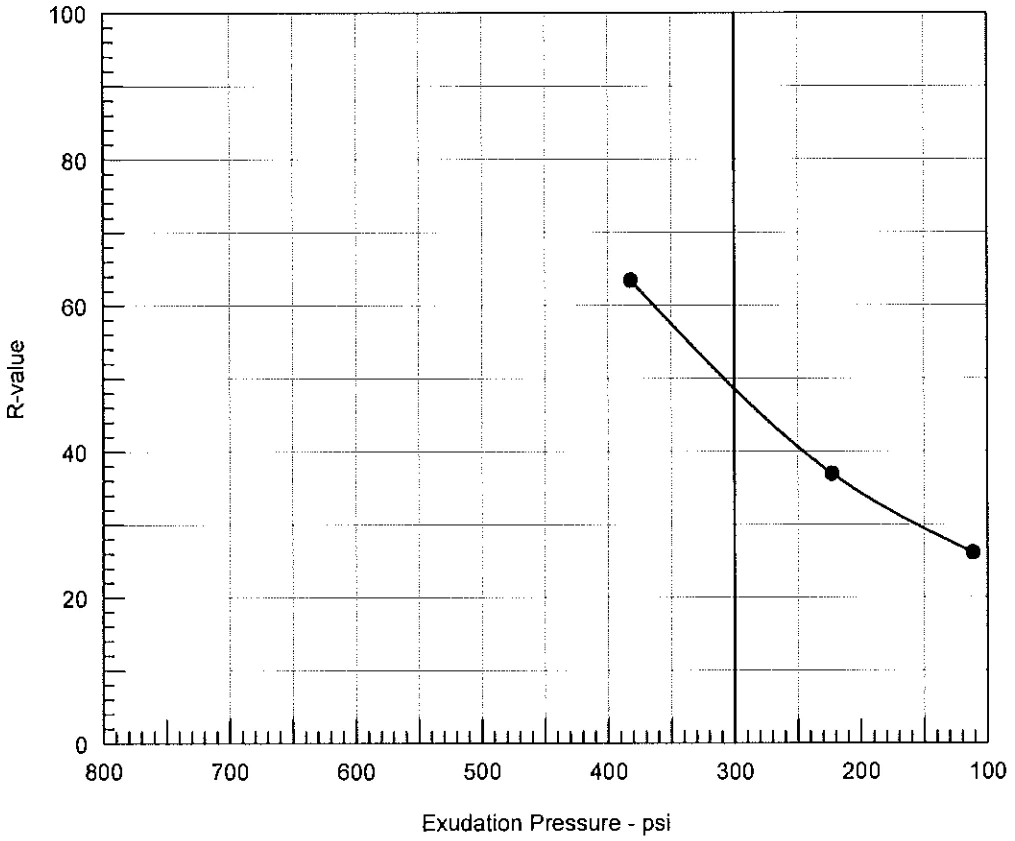
Tested by: JH
Checked by: IC
Remarks:

R-VALUE TEST REPORT

MTGL, Inc.

Figure ______

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	300	113.6	15.3	0.06	94	2.46	111	26	26
2	350	115.1	14.8	0.30	74	2.51	223	37	37
3	350	115.3	14.3	0.79	40	2.50	382	63	63

Test Results	Material Description
R-value at 300 psi exudation pressure = 49	

Project No.: 4815-A04
Project: CAPROCK - TADEMA ONTARIO
Sample Number: B17 **Depth:** 0-5'

Date: 9/2/2015

Tested by: JH
Checked by: IC
Remarks:

R-VALUE TEST REPORT
MTGL, Inc.

Figure ______

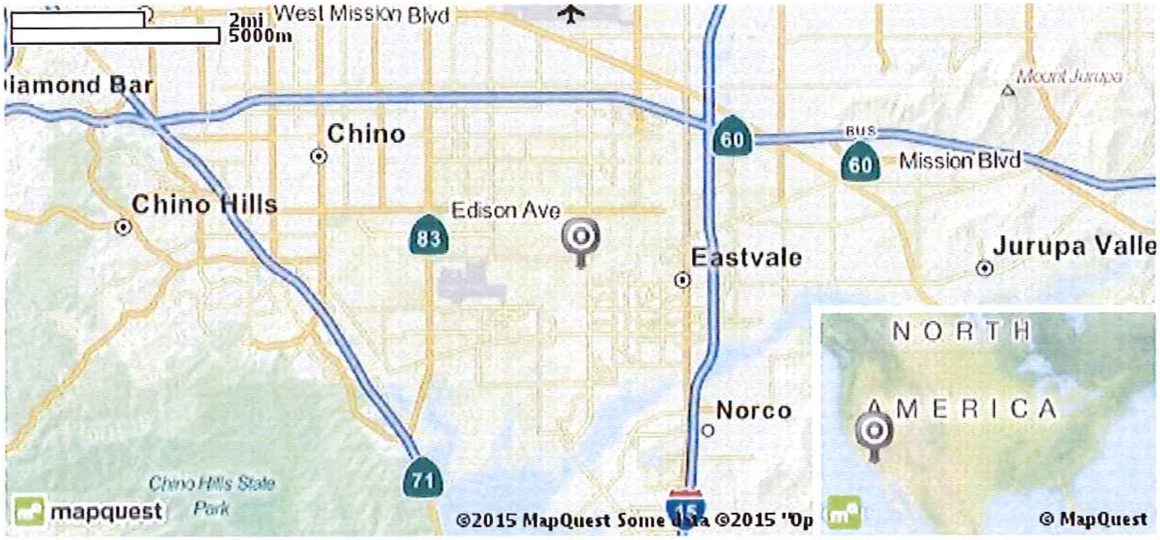
APPENDIX D

SEISMICITY

USGS Design Maps Summary Report

User–Specified Input

Report Title	Tadema Business Park Wed August 19, 2015 22:11:08 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	33.9796°N, 117.5966°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

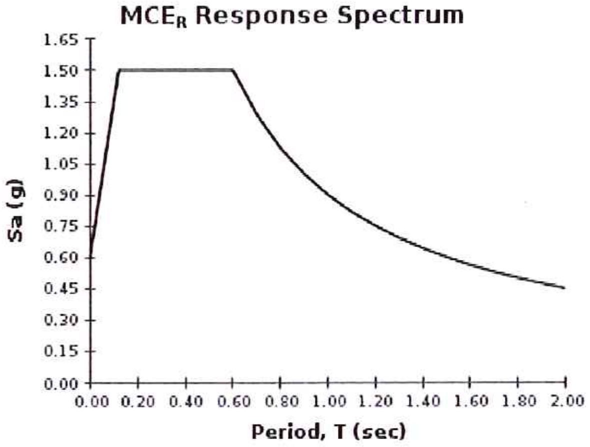


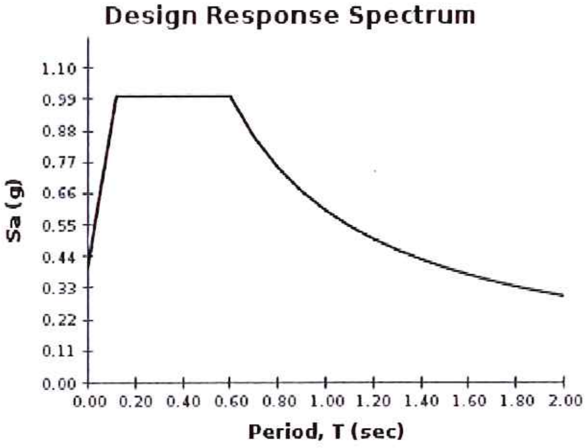
USGS–Provided Output

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.


MCE_R Response Spectrum


Design Response Spectrum

NOTABLE FAULTS WITHIN 100 MILES

Fault Name	Distance (mi)
Chino	5.5
Elsinore;W+GI+T+J	9.1
San Jose	10.5
Cucamonga	12.2
Sierra Madre	12.9
Puente Hills (Coyote Hills)	16.6
San Jacinto;SBV+SJV+A+CC+B+SM	17.7
San Jacinto;SJV+A+CC+B+SM	20.8
S. San Andreas	21.1
Clamshell-Sawpit	23.0
S. San Andreas;CH+CC+BB+NM+SM	23.4
San Joaquin Hills	23.9
Cleghorn	24.0
S. San Andreas;SSB+BG+CO	24.5
Puente Hills (Santa Fe Springs)	24.5
Elsinore;T+J+CM	24.6
Raymond	25.9
San Jacinto;A+CC+B	28.7
Elysian Park (Upper)	29.6
North Frontal (West)	29.9
Puente Hills (LA)	30.5
Newport Inglewood Connected	31.0
Newport-Inglewood (Offshore)	32.5
Verdugo	33.7
Hollywood	37.7
Santa Monica Connected alt 2	40.5
Palos Verdes	41.1
Sierra Madre (San Fernando)	45.1
San Gabriel	45.6
S. San Andreas;BG+CO	45.7
Santa Monica Connected alt 1	47.2
Helendale-So Lockhart	48.8
Northridge	50.3
Pinto Mtn	50.5
Coronado Bank	51.7
North Frontal (East)	51.8
Malibu Coast	53.5

Fault Name	Distance (mi)
Anacapa-Dume, alt 2	55.0
Elsinore;J+CM	55.5
Santa Susana	56.8
Rose Canyon	59.9
Lenwood-Lockhart-Old Woman Springs	60.3
Holser	62.3
Anacapa-Dume	63.1
Johnson Valley (No)	64.5
Simi-Santa Rosa	66.9
Landers	68.1
Burnt Mtn	68.1
Eureka Peak	69.6
San Jacinto;CC+B+SM	70.1
Oak Ridge	70.7
San Jacinto;C	70.9
So Emerson-Copper Mtn	71.3
S. San Andreas;PK+CH+CC+BB+NM	72.0
Gravel Hills-Harper Lk	72.4
San Cayetano	73.8
Calico-Hidalgo	77.3
Blackwater	78.1
S. San Andreas;CO	78.7
Earthquake Valley	80.2
Pisgah-Bullion Mtn-Mesquite Lk	82.6
Santa Ynez (East)	85.7
Santa Ynez Connected	85.7
Garlock	86.0
Pitas Point Connected	92.0
Ventura-Pitas Point	92.0
Garlock;GE+GC	93.1
S. San Andreas;PK+CH+CC+BB	93.4
So Sierra Nevada	93.8
Santa Cruz Island	95.6
Channel Islands Thrust	95.8
Mission Ridge-Arroyo Parida-Santa Ana	96.0
San Jacinto;B+SM	96.3
Oak Ridge (Offshore)	98.0
Pleito	98.5

APPENDIX E

SOIL PERCOLATION TESTING

APPENDIX E

SOIL PERCOLATION TESTING

To establish the design infiltration rate, we have utilized the Percolation test Procedures of the San Bernardino County Department of Environmental Health as specified in the Technical Guidance Document Appendices of the Model Water Quality Management Plan.

An 8 inch diameter percolation test hole was drilled to a depth of 15 feet. The test hole was pre-soaked to allow the water flow to hold a constant level at least 5 times the hole's radius above the gravel at the bottom of the hole. The approximate locations of the test hole is depicted in the accompanying Boring Location Map (Figure 2).

Based on the samples obtained from the exploratory borings, the subsurface materials are classified as silty sand to sandy silt (SM/ML) in accordance with the Unified Soil Classification System as shown on the accompanying boring logs.

The grain size distribution affects soil permeability. Coarse-grained soils with large median particle sizes will yield higher infiltration rates. Finer grained soils will yield lower percolation rates. The percolation rates measured during our testing are as follows:

Test Hole	**Test Depth (ft)**	**Measured Percolation Rate (min/in)**
P-1 (B-20)	15	0.67

Long-term sustainable infiltration rates may be affected by several factors including the degree of saturation of the adjacent ground and the infiltration of finer grained soils into the system. To account for these factors, the application of these rates should therefore consider the use of an appropriate factor of safety.

The development of the factor of safety should be based upon the more conservative rate obtained during testing and include consideration of the impacts of deteriorated performance and should anticipate that the measured test rates will be reduced over time.

PERCOLATION TEST DATA

Project:	Caprock - Tadema	Project No:	4815A04	Date:	8/10/2015
Test Hole No.:	P-1 (B-20)	Tested by:	B. Hulse		
Depth of Hole (in), D_T:	180	USCS Soil Classification:	Silty Sand to Sandy Silt (SM-ML)		

Test Hole Dimensions (inches)			Length	Width	
Diameter (inches) =	8	Sides (if rectangular) =	N/A	N/A	

Sandy Soil Criteria Test *

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in)	Final Depth to Water (in)	Change in Water Level (in)	Greater than or Equal to 6" (y/n)
1	10:11	10:36	25	0.00	31.50	31.50	Y
2	10:37	11:02	25	0.00	30.25	30.25	Y

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

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_o Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min/in)
1	11:09 AM	11:19 AM	10	0.00	16.50	16.50	0.61
2	11:20 AM	11:30 AM	10	0.00	16.00	16.00	0.62
3	11:32 AM	11:42 AM	10	0.00	15.00	15.00	0.67
4	11:43 AM	11:53 AM	10	0.00	15.25	15.25	0.66
5	11:56 AM	12:06 PM	10	0.00	15.25	15.25	0.66
6	12:08 PM	12:18 PM	10	0.00	15.50	15.50	0.65
7	12:20 PM	12:30 PM	10	0.00	15.00	15.00	0.67
8							
9							
10							
11							
12							
13							
14							
15							

COMMENTS:

APPENDIX F

STANDARD GRADING SPECIFICATIONS

APPENDIX F
GENERAL EARTHWORK AND GRADING SPECIFICATIONS

GENERAL

These specifications present general procedures and requirements for grading and earthwork as shown on the approved grading plans, including preparation of areas to be filled, placement of fill, installation of subdrains, and excavations. The recommendations contained in the attached geotechnical report are a part of the earthwork and grading specifications and shall supersede the provisions contained herein in the case of conflict. Evaluations performed by the Consultant during the course of grading may result in new recommendations, which could supersede these specifications, or the recommendations of the geotechnical report.

EARTHWORK OBSERVATION AND TESTING

Prior to the start of grading, a qualified Geotechnical Consultant (Geotechnical Engineer and Engineering Geologist) shall be employed for the purpose of observing earthwork procedures and testing the fills for conformance with the recommendations of the geotechnical report and these specifications. It will be necessary that the Consultant provide adequate testing and observation so that he may determine that the work was accomplished as specified. It shall be the responsibility of the Contractor to assist the Consultant and keep them apprised of work schedules and changes so that he may schedule his personnel accordingly.

It shall be the sole responsibility of the Contractor to provide adequate equipment and methods to accomplish the work in accordance with applicable grading codes or agency ordinances, these specifications and the approved grading plans.

Maximum dry density tests used to determine the degree of compaction will be performed in accordance with the American Society for Testing and Materials Test Method (ASTM) D1557.

PREPARATION OF AREAS TO BE FILLED

Clearing and Grubbing: All brush, vegetation and debris shall be removed or piled and otherwise disposed of.

Processing: The existing ground which is determined to be satisfactory for support of fill shall be scarified to a minimum depth of 6 inches. Existing ground, which is not satisfactory, shall be overexcavated as specified in the following section.

Overexcavation: Soft, dry, spongy, highly fractured or otherwise unsuitable ground, extending to such a depth that surface processing cannot adequately improve the condition, shall be overexcavated down to firm ground, approved by the Consultant.

Moisture conditioning: Overexcavated and processed soils shall be watered, dried-back, blended, and mixed as required to have a relatively uniform moisture content near the optimum moisture content as determined by ASTM D1557.

Recompaction: Overexcavated and processed soils, which have been mixed, and moisture conditioned uniformly shall be recompacted to a minimum relative compaction of 90 percent of ASTM D1557.

Benching: Where soils are placed on ground with slopes steeper than 5:1 (horizontal to vertical), the ground shall be stepped or benched. Benches shall be excavated in firm material for a minimum width of 4 feet.

FILL MATERIAL

General: Material to be placed as fill shall be free of organic matter and other deleterious substances, and shall be approved by the Consultant.

Oversize: Oversized material defined as rock, or other irreducible material with a maximum dimension greater than 12 inches, shall not be buried or placed in fill, unless the location, material, and disposal methods are specifically approved by the Consultant. Oversize disposal operations shall be such that nesting of oversized material does not occur, and such that the oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 feet vertically of finish grade or within the range of future utilities or underground construction, unless specifically approved by the Consultant.

Import: If importing of fill material is required for grading, the import material shall meet the general requirements.

FILL PLACEMENT AND COMPACTION

Fill Lifts: Approved fill material shall be placed in areas prepared to receive fill in near-horizontal layers not exceeding 6 inches in compacted thickness. The Consultant may approve thicker lifts if testing indicates the grading procedures are such that adequate compaction is being achieved with lifts of greater thickness. Each layer shall be spread evenly and shall be thoroughly mixed during spreading to attain uniformity of material and moisture in each layer.

Fill Moisture: Fill layers at a moisture content less than optimum shall be watered and mixed, and wet fill layers shall be aerated by scarification or shall be blended with drier material. Moisture conditioning and mixing of fill layers shall continue until the fill material is at uniform moisture content at or near optimum.

Compaction of Fill: After each layer has been evenly spread, moisture conditioned, and mixed, it shall be uniformly compacted to not less that 90 percent of maximum dry density in accordance with ASTM D1557. Compaction equipment shall be adequately sized and shall be either specifically designed for soil compaction or of proven reliability, to efficiently achieve the specified degree of compaction.

Fill Slopes: Compacting on slopes shall be accomplished, in addition to normal compacting procedures, by backrolling of slopes with sheepsfoot rollers at frequent increments of 2 to 3 feet as the fill is placed, or by other methods producing satisfactory results. At the completion of grading, the relative compaction of the slope out to the slope face shall be at least 90 percent in accordance with ASTM D1557.

Compaction Testing: Field tests to check the fill moisture and degree of compaction will be performed by the consultant. The location and frequency of tests shall be at the consultant's discretion. In general, these tests will be take at an interval not exceeding 2 feet in vertical rise, and/or 1,000 cubic yards of fill placed. In addition, on slope faces, at least one test shall be taken for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope.

SUBDRAIN INSTALLATION

Subdrain systems, if required, shall be installed in approved ground to conform to the approximate alignment and details shown on the plans or herein. The subdrain location or materials shall not be changed or modified without the approval of the Consultant. The Consultant, however, may recommend and, upon approval, direct changes in subdrain line, grade or materials. All subdrains should be surveyed for line and grade after installation and sufficient time shall be allowed for the surveys, prior to commencement of fill over the subdrain.

EXCAVATION

Excavations and cut slopes will be examined during grading. If directed by the Consultant, further excavation or overexcavation and refilling of cut areas, and/or remedial grading of cut slopes shall be performed. Where fill over cut slopes are to be graded, unless otherwise approved, the cut portion of the slope shall be made and approved by the Consultant prior to placement of materials for construction of the fill portion of the slope.