Inter Office Memo

DATE: May 1, 2020

TO: Tiana Perez
Architect

FROM: Steve Deis, GE
Construction Engineer

SUBJECT: Preliminary Geotechnical Report
Project No. T90203
Dan Ronquillo Drive and West Avenue
Environmental Compliance Center
Fresno, California

The Construction Division is pleased to provide this preliminary geotechnical report which provides design criteria for foundations, pavement structural section recommendations, and lab testing results for the Environmental Compliance Center. This report has been prepared in general conformance with industry standard methods. We appreciate the opportunity to provide service to you and trust this information meets your current needs. If there are any questions concerning the information presented in this report, please contact the Materials Laboratory at your convenience.

Attachment

PRELIMINARY GEOTECHNICAL REPORT
PROJECT NO. T90203
ENVIRONMENTAL COMPLAINCE CENTER
DAN RONQUILLO DRIVE AND WEST AVENUE
FRESNO, CALIFORNIA

April 22, 2020

A Report Prepared for:

County of Fresno, Department of Public Works and Planning
Capital Projects Division
2220 Tulare Street, 8th Floor
Fresno, California 93721

PRELIMINARY GEOTECHNICAL REPORT
PROJECT NO. T90203
DAN RONQUILLO DRIVE AND WEST AVENUE
ENVIRONMENTAL COMPLIANCE CENTER
FRESNO, CALIFORNIA

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April 22, 2020

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1 INTRODUCTION

1.1 GENERAL

This preliminary report presents the results of the foundation investigation for the proposed new Environmental Compliance Center located on southwest corner of South West Avenue and West Dan Ronquillo Drive (APN: 458-060-72) in Fresno, California. The investigation consisted of a field exploration, laboratory testing, engineering analysis, and preparation of this written report.

1.2 PROJECT DESCRIPTION

The project is a new Environmental Compliance Center facility for County residents to take their household hazardous waste materials for proper disposal. The project involves construction of office buildings, a warehouse, pads for various hazardous material containers, an outdoor amphitheater, driveways, a bus stop, parking stalls, a wrought iron fence, and landscape. The property is quadrilateral shaped parcel which consists of approximately 2.68 acres. It has two 24 foot driveways on Dan Ronquillo Drive. The parking area will be paved with hot mix asphalt and all the building and container pads, driveways, and bus stop will use Portland cement concrete. The adjacent property to the south has a City of Fresno pump station, and the west/southwest boundary is a vacant lot. The site's east boundary is West Avenue and the north boundary is Dan Ronquillo Drive. The entire project is in the County of Fresno. The facility will be operated and maintained by the County Department of Public Works and Planning (PW&P).

No anticipated loads for the building were provided prior to this investigation so a range of loads typical for this type of construction were assumed including wall loads of 1 to 5 kips/foot and column loads of 5 to 20 kips. Fills and cuts ranging from zero to three feet are expected to achieve pad grades and positive site drainage.

1.3 EXISTING FACILITIES

The property is currently vacant. Three sides of the property have a 6 foot temporary chain-link fence, and the side adjacent to the City of Fresno pump station has a 6 foot block wall.

A review of historic photos available on Google Earth shows what appears to be a farmhouse in the northeast corner of the site and some outbuildings spread over the remainder of the site. No historic data was available for these buildings, and no substructures were encountered during drilling. Because of the age of the photos and the nature of the construction, some unknowns could be anticipated in the area during construction.

1.4 PURPOSE AND SCOPE OF WORK

The purpose of this materials report is to provide foundation design parameters, pavement design recommendations, and laboratory test results to aid in project design. Our scope of services consisted of a field investigation program, laboratory testing, and preparation of this written report. The report provides the following:

- A description of the proposed project including a site vicinity map showing the location of the site and a site plan showing the approximate locations of the exploration points for this study
- A summary of the field exploration and laboratory testing programs, including boring logs
- A general description of the surface and subsurface materials, including groundwater conditions
- Recommendations for site preparation and earthwork
- Recommendations for temporary excavations, including temporary slopes and trench backfill
- Recommendations to aid in foundation design including soil bearing pressures, anticipated settlements, and lateral pressures
- Recommended minimum flexible pavement sections for a range of traffic indices.
- Earth pressures for the design of retaining structures
- Recommended seismic design criteria and comments on liquefaction potential

- Recommended subgrade preparation for concrete slabs supported on grade
- Comments on the general corrosion potential of on-site soils to buried metal and concrete
- Comments regarding near-structure site drainage

2 PERTINENT REPORTS AND INVESTIGATIONS

During the preparation of this report, the following reports, drawings and information were used.

- California Department of Water Resources (DWR) Historical Data Map Interface
- United States Department of Agriculture (USDA), Natural Resources Conservation Service, Web Soil Survey using Soil Spatial Data of, Eastern Fresno Area, 2012

3 PHYSICAL SETTING

3.1 SURFACE CONDITIONS AND TOPOGRAPHY

The site lies in the eastern portion of Fresno County. The regional topography is relatively flat. The area immediately surrounding the site is relatively flat, as well.

3.2 REGIONAL GEOLOGY

The project site lies in the central portion of the San Joaquin Valley in the Great Valley geomorphic province in California. This province was formed by the filling of a large structural trough or downwarp in the underlying bedrock. The trough is situated between the Sierra Nevada Range on the east and south and the Coast Range on the west. Both of these mountain ranges were initially formed by uplifts that occurred during the Jurassic and Cretaceous periods of geologic time (greater than 65 million years ago). Renewed uplift began in the Sierra Nevada during the Tertiary time, and is continuing today. The trough that underlies the valley is asymmetrical, with the greatest depths of sediments near the western margin. The sediments that fill the trough originated as erosion material from the adjacent mountains and foothills.

3.3 LOCAL CLIMATOLOGY

The average mean annual precipitation is 9 inches and the average mean annual air temperature is 61˚F. The average annual extremes in temperature range from 23˚F to 107˚F.

3.4 EARTH MATERIALS

The USGS and Soil Conservation Survey (SCS) have mapped the surface materials in the project area. These soils were derived from granitic alluvium. The area of the project is included in the Soil Survey of Eastern Fresno County (Soil Survey) by the USDA. The area of the project includes following soils:

- Greenfield coarse sandy loam - GsA
- Ramona sandy loam - Rb

The soils of the Greenfield series are deep, well drained, and moderately coarse textured and they have a moderately permeable subsoil. These soils are formed in young granitic alluvium that is poorly sorted and contains many coarse particles. The nearly level to moderately sloping soils are on smooth fans of many streams draining into the San Joaquin Valley from the foothills. The profile of this soil is similar to that of Greenfield sandy loam, 0 to 3 percent slopes, except that it has a somewhat coarser textured surface layer. In addition, there is a greater proportion of coarse and very coarse sand particles in the subsoil and in the parent alluvium beneath. This is reflected in low to moderate available water holding capacity.

The Ramona series consists of well drained soils that formed in moderately coarse textured old granitic alluvium. These soils have a dominantly sandy clay loam subsoil that tends to slow, but not seriously impede, penetration by roots and water. The soils are smooth and nearly level and occupy low alluvial terraces. They comprise a large part of the low alluvial terraces from Friant to Orange Cove. Areas of these soils are also located in some lower foothill valleys. The profile of this soil is similar to Ramona sandy loam, but it overlies an unrelated, compact, weakly cemented sandy substratum. The substratum is normally several feet thick and underlies the soil at a depth of 2 to 4 feet. This soil is widely distributed on the low alluvial terraces of the San Joaquin Valley.

The above description provides a general summary of the subsurface conditions encountered during the field exploration and further validated by the laboratory testing program. For a more thorough description of the actual conditions encountered at the specific boring location, refer to the Boring Logs presented in Appendix C. All soils have been classified in general accordance with the Unified Soil Classification System (ASTM D2487). The natural earth materials are mapped as Pleistocene non-marine deposits. The soils encountered in the test borings consisted mainly of silty sand. The consistency of the soil ranges from medium dense to very dense.

3.5 DEPTH TO BEDROCK

Bedrock was not encountered during the exploration program. It is anticipated, based on other collected information, that the depth to bedrock is more than 200 feet below existing grade and will not influence the project.

3.6 GEOLOGIC HAZARDS

Based on the site conditions encountered, it is anticipated that geologic hazards of slope instability, deep subsidence, hydrocompactive soil, expansive soil, ground rupture, or liquefaction will not impact the site.

3.7 WATER

3.7.1 Surface Water

No surface water or waterways impact this site.

3.7.2 Groundwater

Groundwater was not encountered in any of the borings to the depth explored.

Groundwater data from four water wells in areas adjacent to the project alignment was obtained from the California Department of Water Resources (DWR) Historical Data Map Interface website. Data from the wells indicate the groundwater depth is from 58.4 to 72.9 feet.

TABLE 3.7-1
GROUNDWATER DATA

Well No.	Location	Time	Water Elevation Range (feet)	Last Depth Reading (feet)
1	367424N1198288W001	1965-1982	61.1-73.4	72.9
2	367443N1198293W001	1955-1965	45.3-75.0	67.9
3	367338N1198491W001	1958-1962	15.4-58.4	58.4
4	367355N1198127W001	1950-1968	33.7-73.3	65.0

Groundwater conditions at other locations, near the vicinity of the project, may not be apparent, and may or may not differ from the conditions at the wells.

It is possible that groundwater conditions at the site could change at some time in the future due to variations in rainfall, groundwater withdrawal, water banking, or other factors not apparent at the time our test borings were made. Groundwater is not anticipated to have any effect on project design or construction.

4 PREVIOUS EXPLORATION AND INVESTIGATION

No data on previous investigations was found.

5 EXPLORATION AND TESTING PROGRAM

5.1 FIELD INVESTIGATION AND TESTING PROGRAM

Samples were collected at fourteen borings throughout the site. Six bulk samples were taken in the driveways and parking areas, and the remaining samples were taken from eight borings where the new building and warehouse are going to be according to the preliminary site plan. All the samples were taken below the existing topsoil and vegetation. Soils were classified during drilling operations in general accordance with ASTM D2488. The samples were collected by rotary drilling operations, using a Mobile B-32 truck-mounted drill rig with 5" outside diameter (O.D.) continuous flight augers for borings 1 through 10 and the remaining borings using a CME 45B truck-mounted drill rig with 7-5/8" O.D. hollow stem auger drilling techniques with either a 140-pound safety hammer or a 140-pound automatic hammer. Samples retrieved during the exploration were bagged, labeled, and brought to the laboratory for testing. The locations of the borings are indicated in the Site Map in Appendix B of this report.

The earth materials encountered in the test boring were visually classified in the field and a continuous log was recorded. In-place samples of soil units were collected from the test borings at selected depths by driving a 2.5-inch inside diameter (I.D.) split barrel sampler containing brass liners 18 inches into the undisturbed soil with a 140-pound automatic hammer free-falling a distance of 30 inches. In addition, a 1.4-inch I.D. standard penetration sampler without liners was driven 18 inches in the same manner. This latter sampling procedure generally conformed to the ASTM D1586 test procedure. Resistance to sampler penetration over the last 12 inches is noted on the Boring Logs as the "Blows per Foot". The penetration indices listed on the Boring Logs have not been corrected for the effects of overburden pressure, sampler size, rod length, or hammer efficiency. Bulk samples we also obtained at each of the boring locations.

Penetration rates determined in general accordance with ASTM D1586 were used to aid in evaluating the consistency, compression, and strength characteristics of the foundation soils.

5.2 LABORATORY TESTING PROGRAM

Laboratory tests were performed on selected samples to evaluate their physical characteristics and engineering properties. The laboratory testing program was designed with emphasis on evaluation of geotechnical properties of foundation materials as they pertain to the proposed project. The laboratory testing program included the performance of the following laboratory tests:

- Moisture Content (ASTM D2216)
- Sieve Analysis, #200 Wash (ASTM D1140
- Resistance Value (California Test Method No. 301)
- Soluble Sulfate Content (California Test Method No. 417)
- Soluble Chloride Content (California Test Method No. 422)
- Minimum Resistivity and pH (California Test Method No. 643)
- Direct Shear Test of Soils (ASTM D3080)
- Determination of Density (Unit Weight) of Soil Specimens (ASTM D7263)
- One-Dimensional Consolidation (ASTM D2435)
- Plasticity Index (ASTM D4318)

Unit weight result are shown on the Boring Logs in Appendix C. The soluble sulfate, soluble chloride, pH, and minimum resistivity results are presented in Section 6, “Corrosion Potential”. The remaining results of the laboratory testing program are presented in Appendix D.

6 SITE CONDITIONS

6.1 SURFACE CONDITIONS

This site is located on vacant agricultural land. The surface is relatively flat and level. IIt appears to have been recently plowed or turned to prevent plant growth. There is a moderately dense covering of plants that are expected to get denser before the project begins construction. The top few inches of the soil are loose, likely from agricultural work on the site.

There are no structures present on the site.

6.2 SUBSURFACE

The near surface site soils encountered in our test borings generally consisted of a layer of silty sand extending from 10 to 26.5 feet below the existing grade. Alternating discontinuous layers of silty sand, sandy silt, clayey sand, gravelly sand, and lean clay underlie this surface layer. No cemented soils (hardpan) were encountered.

No groundwater was observed in any of the borings.

This is a general summary of the soil and groundwater conditions encountered in this investigation. A more detailed description of the soils encountered in each boring is shown on the boring logs in Appendix C.

7 CORROSION CHARACTERISTICS

Soil samples obtained from test boring B-11 and B-14 at various depths were tested for pH, soluble sulfate content, soluble chloride content, and minimum resistivity. Results are presented below in Table 6-1.

TABLE 7.1-1
CORROSION TEST RESULTS

Boring No.	Depth	Minimum Resistivity (ohm-cm)	pH	Soluble Sulfate (ppm)	Soluble Chloride (ppm)
B-11	3 - 4.5 feet	3542	9.52	45	18
B-14	0 - 3 feet	4856	7.82	34	15

The values obtained are considered inside tolerable limits for buried concrete structures. .

8 LEAD CONTENT

Bulk samples were collected using a hand auger. Eight bulk soil samples were collected for aerially deposited lead testing. Each sample was then placed in an air tight plastic bag and identified with a unique identification number. Only 0 - 6” and 6” -12” samples from each location were sent out for testing. The bulk samples were submitted under a chain of custody to BSK Associates in Fresno, California. The results of their testing are presented in Table 7-1 below:

TABLE 8.1-1
LEAD CONTENT TEST RESULTS

SAMPLE NO.	SAMPLE LOCATION	TEST METHOD	RESULTS
20-0230	120’S-144’W form the NE corner, 0-6”	EPA 6010B	ND
20-0231	120’S-144’W form the NE corner, 6-12”	EPA 6010B	ND
20-0232	100’S-100’W form the NE corner, 6-12”	EPA 6010B	ND
20-0233	100’S-100’W form the NE corner, 6-12”	EPA 6010B	ND

ND - none detected

Since no lead was detected in the on-site soil, any requirement for a lead compliance plan can be waived.

9 SEISMIC RECOMMENDATIONS

9.1 LOCAL FAULTING

There are no known faults which cut through the local soil at the site. The project site is not located in an Alquist-Priolo Earthquake Fault Zone, as defined by Special Publication 42 (Revised 2007) published by the California Geologic Survey (CGS). Based on our current understanding of the geologic framework and tectonic setting of the project, the primary sources of seismic shaking are anticipated to be either the San Andreas Fault (creeping segment), the San Andreas Fault (Parkfield segment), and the Great Valley Fault (Coalinga segment).

9.2 SEISMIC DESIGN CRITERIA

The project site is located in a region with the potential for relativity low to moderate seismic activity. The more significant faults that could influence the project site include the Creeping Segment of the San Andreas Fault (Fault ID No. 182), the Parkfield Section of the San Andreas Fault (Fault ID No. 214), and the Coalinga Section of the Great Valley 13 (Fault ID No. 205).

Based on the data from borings and , the site can be classified as Soil Profile Type D. A Soil Shear Wave Velocity (V_{s30}) of 270 m/s was determined and used for the evaluation.

It is anticipated that the local building official will still classify this project as Risk Category II despite the presence of unknown amount of unknown hazardous materials.

TABLE 9.1-1
SITE CHARACTERISTICS AND
GOVERING DETERMINISTIC FAULTS PARAMETERS

Site Coordinates	Lat = 36.7399 deg, Long = -119.8271 deg
Soil Shear Wave Velocity	270 m/s
Risk Category	II
Site Soil Class	D
MCE_R, Ground Motion (for 0.2 Second Period) S_S	0.622

MCE_R, Ground Motion (for 1.0 Second Period), S_1	0.237
Site-Modified Spectral Acceleration (SA) Value (for 0,2 Second Period), S_{MS}	0.81
Site-Modified Spectral Acceleration (SA) Value (for 1,0 Second Period), S_{M1}	See ASCE 7-16 Section 11.4.8
Numeric Seismic Design Value at 0.2 Second SA, S_{DS}	0.54
Numeric Seismic Design Value at 1.0 Second SA, S_{D1}	See ASCE 7-16 Section 11.4.8
Seismic Design Category, SDC	0.00 See ASCE 7-16 Section 11.4.8km
Site Amplification Factor at 0.2 Second, F_a	1.302
Site Amplification Factor at 1.0 Second, F_v	See ASCE 7-16 Section 11.4.8
MCE_G Peak Ground Acceleration, PGA	0.27
Site Amplification Factor at PGA, F_{PGA}	1.33
Site Modified PGA, PGA_M	0.359
Long Period Transition Period in Seconds, T_L	12
Probabilistic Risk-Targeted Ground Motion (0.2 Seconds), SsRT	0.622
Factored Uniform-Hazard (2% in 50 years) Spectral Acceleration, SsUH	0.673
Factored Deterministic Acceleration Value (0.2 Seconds), SsD	1.5
Probabilistic Risk-Targeted Ground Motion (1.0 Seconds), S1RT	0.237
Factored Uniform-Hazard (2% in 50 years) Spectral Acceleration, S1UH	0.252
Factored Deterministic Acceleration Value (1.0 Seconds), S1D	0.6
Factored Deterministic Acceleration Value (PGA), PGAd	0.5
Mapped Value of the Risk Coefficient at Short Periods, C_{RS}	0.924
Mapped Value of the Risk Coefficient at a Period of 1 .0 Second, C_{R1}	0.94

9.3 LIQUEFACTION POTENTIAL

In order for liquefaction of soils due to ground shaking to occur, it is generally accepted that four conditions will exist:

- ❑ The subsurface soils are in a relatively loose state,
- ❑ The soils are saturated,
- ❑ The soils are non-plastic, and
- ❑ Ground motion is of sufficient intensity to act as a triggering mechanism

Based on the average density, depth to groundwater, and anticipated ground shaking at the site, liquefaction and associated seismically induced settlement is considered unlikely.

10 FOUNDATION RECOMMENDATIONS

10.1 SITE PREPARATION

10.1.1 Stripping and Grubbing

Prior to general site grading, existing vegetation, existing underground utilities, and any debris should be stripped and disposed of outside the construction limits. We estimate the depth of stripping to be less than 4 inches. Stripped topsoil, less any debris, may be stockpiled and reused for landscape purposes. Organics which remain below stripping depth may be incorporated into the fill areas as long as the total amount of organics does not exceed 3 percent, by weight, of the fill material (ASTM D-2974).

10.1.2 Undocumented Fill and Subsurface Obstructions

Historic photos show a residence and outbuildings on the site prior to 19998. During site demolition and prior to actual site grading, a reasonable search should be conducted to locate any undocumented fill soils, wells, trees, or existing utilities that may exist within the area of construction. Any obstructions should be removed from the project area. If any areas or pockets of soft or saturated soils or void spaces made by burrowing animals, undocumented fill, or other disturbed soil are encountered, they should be over-excavated to firm native material and replaced with engineered fill constructed as recommended in this report. Excavations for removal of the above items should be backfilled with engineered fill. Any wells not to remain should be abandoned in accordance with the requirements of the County of Fresno Environmental Health Department.

10.1.3 Scarification and Compaction

After stripping the site and performing any necessary removals indicated above, the exposed surface (in areas of overexcavation or stripped surface in areas to receive fill) should be scarified to a depth of 6 inches, uniformly moisture conditioned to at or near optimum moisture content and compacted to the requirements for engineered fill.

Should site grading be performed during or subsequent to wet weather, near-surface site soils may be significantly above optimum moisture content. These conditions could hamper equipment maneuverability and efforts to compact site soils to the recommended compaction criteria. Disking to aerate, chemical treatment, replacement with drier material, stabilization with a geotextile fabric or grid, or other methods may be required to reduce excessive soil moisture and facilitate earthwork operations. Any consideration of chemical treatment (e.g. lime) to facilitate construction would require additional soil chemistry evaluation and could affect landscape areas.

10.2 ENGINEERED FILL

10.2.1 Materials

All engineered fill soils should be nearly free of organic or other deleterious debris and less than 3 inches in maximum dimension. The native soil materials, exclusive of debris, may be used as Engineered Fill provided, they contain less than 3 percent organics by weight (ASTM D-2974).Any imported fill materials, if any, to be used for engineered fill should be sampled and tested by the project Geotechnical Engineer prior to being transported to the site. Recommended requirements for imported engineered fill, as well as applicable test procedures to verify material suitability are provided on Table 10.2-1.

TABLE 10.2-1
SOIL MATERIALS TEST PROCEDURES FOR IMPORTED FILL

		Test Procedures		
Sieve Size	Percent Passing	ASTM[1]	Caltrans[2]	AASHTO[3]
76 mm (3 inch)	100	C136	202	T 27
19 mm (¾ inch)	80 – 100	C136	202	T 27
No. 4	60 – 100	C136	202	T 27

No. 200	20 – 50	C 36	202	T 27
Plasticity Liquid Limit < 30	Plasticity Index < 8	D4318	204	T 89, T 90
Soluble Sulfates	< 2000		417	
Soluble Chlorides	< 500		422	
Minimum Resistivity	> 1000		643	

Notes:

[1] American Society for Testing and Materials Standards (latest edition)

[2] State of California, Department of Transportation, Standard Test Methods (Latest edition)

[3] American Association of State Highway and Transportation Officials, Standard Specifications for Transportation Materials and Methods of Sampling and Testing (latest edition)

10.2.2 Compaction Criteria

Soils used for engineered fill should be uniformly moisture-conditioned to at least the optimum moisture content, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to at least 90 percent relative compaction.

Disking and/or blending may be required to uniformly moisture-condition soils used for engineered fill.

10.3 TEMPORARY EXCAVATIONS

10.3.1 General

All excavations must comply with applicable local, State, and Federal safety regulations including the current OSHA Excavation and Trench Safety Standards. Construction site safety is the sole responsibility of the Contractor, who shall also be solely responsible for the means, methods, and sequencing of construction operations. We are providing the information below solely as a service to owner (or the owner's representative). Under no circumstances should the information provided be interpreted to mean that the County is assuming responsibility for construction site safety or the Contractor's activities; such responsibility is not being implied and should not be inferred.

10.3.2 Excavations and Slopes

The Contractor should be aware that slope height, slope inclination, or excavation depths (including utility trench excavations) should in no case exceed those specified in local, State, and/or Federal safety regulations (e.g., OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations). Such regulations are strictly enforced and, if they are not followed, the Owner, Contractor, and/or earthwork and utility subcontractors could be liable for substantial penalties.

Near-surface soils encountered during our field investigation consisted predominately of medium desnse to dense silty sands. In general, all excavations should be constructed and maintained in conformance with current OSHA requirements (29 CFR Part 1926) for Type C soils.

10.3.3 Construction Considerations

Heavy construction equipment, building materials, excavated soil, and vehicular traffic should be kept sufficiently away from the top of any excavation to prevent unanticipated surcharging. If it is necessary to encroach upon the top of an excavation, we can provide comments on slope gradients or lateral earth pressures to address surcharging, if provided with the geometry and loading. Where support systems such as shoring or bracing may be required to provide stability and to protect personnel working within the excavation, the system should be designed by a professional engineer registered in the State of California.

During wet weather, earthen berms or other methods should be used to prevent runoff water from entering all excavations. All runoff should be collected and disposed of outside the construction limits.

10.4 TRENCH BACKFILL

10.4.1 Materials

Pipe zone backfill (i.e., material beneath and in the immediate vicinity of the pipe) should consist of soil compatible with design requirements for the pipe. We recommend the project designer or pipe supplier develop the material specifications based on planned pipe types, bedding conditions, and other factors beyond the scope of this study. Trench zone backfill (i.e., material placed between the pipe zone backfill and finished subgrade) may consist of native soil that meets the requirements for engineered fill provided above.

If import material is used for pipe or trench zone backfill, it should consist of soil with a piping ratio compatible with the adjacent soil or a geotextile separator.

10.4.2 Compaction Criteria

All trench backfill should be placed and compacted in accordance with recommendations provided above for engineered fill. Reduced compaction (85% minimum) could be specified for trench zone backfill in non-structural areas. Mechanical compaction is recommended; ponding or jetting should not be used.

In areas of planned pavements, 95% compaction should be specified a minimum of 2.5 feet below the finished paving grade.

10.5 SPREAD FOUNDATIONS

10.5.1 Allowable Bearing Pressures and Settlements

Conventional spread footing foundations can be supported on approved undisturbed native soil or properly compacted fill. Footings for structures should be embedded at least 12 inches below the lowest adjacent soil grade. Foundation depths should also satisfy structural and constructability considerations.

The design bearing for conventional spread footings will be governed by either the shear strength of the soil or tolerable settlement.

The bearing capacity, based only on the shear strength of the soil, will be dependent upon the footing geometry. Table 10.5-1 presents the expressions for the allowable bearing capacity (shear strength considerations only) for static loading (D.L + sustained L.L) and total combined loading (D.L. + L.L. + transient loading, such as wind or seismic).

TABLE 10.5-1
RECOMMENDED VERTICAL BEARING CAPACITY

Loading Condition	Allowable Bearing (psf)	
	Continuous	Square
Static Loading	1290B + 2290D	1030B + 2290D
Total Combined Loading	1940B + 3430D	1550B +3430D

Note: B is footing width in feet and D is footing embedment depth in feet

For other than relatively small footings, settlement considerations may be governed the design pressure for spread footings supporting dead and live loading.

Analysis, based on Scmertmann and Das, determined the following estimated static settlement based on a range of assumed structural loads. These settlements are based on the assumption that static loading (DL + sustained LL) is 100 percent of total dead and live loading.

TABLE 10.5-2
ESTIMATED SETTLEMENTS DUE TO STATIC LOADING

Footing Type	Loading	Design Bearing (psf)	Estimated Settlement (inches)
Wall	Up to 3,5 kip/ft	Up to 3500	0.30
Wall	Up to 5 kip/ft	Up to 3300	0.35
Column	Up to 17 kip	Up to 4250	0.30

Column	Up to 20 kip	Up to 3200	0.30

The design bearing pressures are net values so the weight of embedded concrete does not need to be included in the foundation loading. Reinforcement for structural considerations should be provided by a structural engineer or building designer.

10.5.2 Lateral Loads

Lateral loads can be resisted by friction between the base of the foundation and supporting foundation soil and passive lateral bearing. The allowable and ultimate frictional coefficients and passive pressure are provided below.

TABLE 10.5-3
PASSIVE PRESSURE AND FRICTIONAL COEFFICIENTS

	Allowable	Ultimate
Frictional Coefficient	0.49	0.74
Passive Pressure (psf/ft of depth)	500	1000
Lateral Translation Needed to Develop Passive Pressure	0.010 D	0.056 D
NOTE: D is the footing depth		

If the deflection resulting from the strain necessary to develop the passive pressure is beyond structural tolerance, additional passive pressure values could be provided based on tolerable deflection. The passive pressure and frictional resistance can be used in combination.

10.5.3 Retaining Structures

The lateral earth pressure against retaining structures will be dependent upon the ability of the walls to deflect. Presented in Table 10.5-4 are the active, at-rest, braced, and dynamic increment lateral earth pressures. These pressures consider engineering judgment and experience with expansive soil. The active soil pressure is applicable to walls capable of 0.0005 radian deflection at the top of the wall. The at-rest pressure should be used for walls fully fixed against rotation or translation. Walls restrained from translation at the top and bottom, but able to deflect 0.0005 radian between restrained points (e.g. basement walls) should be designed for the braced lateral pressure. These lateral earth pressures assume a drained backfill condition.

TABLE 10.5-4
LATERAL EARTH PRESSURES

Condition	Lateral Pressure
Active	32 psf/ft
Braced	20 H psf
At-rest	75 psf/ft

Note: H is the height of the wall in feet

The value for at-rest pressure includes the Jaky solution for normally consolidated material and the locked-in pressure associated with soil pre-stressing due to backfill compaction.

10.5.4 Construction Considerations

Prior to placing steel or concrete, foundation excavations should be cleaned of all debris, loose or soft soil, and water. All foundation excavations should be observed by the project Geotechnical Engineer just prior to placing steel or concrete. The purpose of these observations is to check that the bearing soils actually encountered in the foundation excavations are similar to those assumed in analysis and to verify the recommendations contained herein are implemented during construction.

10.5.5 Interior Concrete Floor Systems

Building interior concrete floor systems (e.g. slabs-on-grade or mat foundations) should be supported on recompacted soils or engineered fill as described in this report. Slab thickness and reinforcement must also satisfy structural considerations.

Slab concrete should have good density, a low water/cement ratio, and proper curing to promote a low porosity.

Based on groundwater depth a capillary break (i.e. gravel layer) is not considered necessary.

In areas to receive moisture-sensitive floor coverings, we recommend that a vapor retarding membrane, such as 10-mil PVC, be provided to act as a vapor barrier. The vapor barrier should be sealed at the seams and utility penetrations. Care should be exercised to avoid tearing, ripping, or displacing the membrane during construction. If the membrane becomes torn or disturbed, it should be removed and replaced or properly patched. The membrane should, in turn, be covered with approximately 1 to 2 inches of saturated surface dry (SSD), clean sand to protect it during construction and aid in curing the concrete. Concrete should not be placed if sand overlying the vapor barrier has been allowed to attain a moisture content greater than 5%. Excessive water beneath interior floor systems could result in future significant vapor transmission through the floor, adversely affecting moisture-sensitive floor coverings and could inhibit proper concrete curing.

10.6 EXTERIOR CONCRETE SLABS SUPPORTED-ON-GRADE

Exterior slabs-on-grade should be supported on approved recompacted soil or engineered fill moisture conditioned in accordance with sections 5.1.3 and 5.2. Exterior slabs adjacent to structures should be provided with a gradient away from the structures. Due to differential moisture variations that may occur, isolated exterior slabs may creep or “walk” away from fixed structures. It should be noted that differential slab movement due to heave may also occur. Careful consideration should be made in design details (e.g. smooth dowels) to compensate for this possible movement. Such details may include providing expansion areas between exterior concrete slabs and building elements such as stucco and masonry fascia.

11 SLOPE STABILITY

There are not any slopes anticipated on the site except those required for site drainage. Any slopes greater than 2:1 (H:V) should be analyzed for global stability. Slopes greater than 3:1 (H:V) should be considered for erosion control such as landscape, hydroseeding, etc..

12 PAVEMENT

12.1 FLEXIBLE PAVEMENT

The subgrade R-value for the on-site soil was evaluated in the laboratory on samples taken from borings B-4 and B-8. The on-site material has measured R-values of 70 and 72 as indicated in the laboratory results in Appendix D. According to Section 614.3 of Caltrans Highway Design Manual used for pavement design, the R-Value for subgrade soils used for pavement design should be limited to no more than 50. An R-value of 50 was used for the design of the pavement structure. A Traffic Index (TI) 5.0 was used for parking lot. Pavement structural sections can be selected from Table 12.1-1.

TABLE 12.1-1
RECOMMENDED PAVEMENT STRUCTURAL SECTION

Traffic Index	Dense Graded Asphalt Concrete	Aggregate Base (Class II)
5.0	0.40’	-
5.0	0.25’	0.35’

13 CLOSURE

Recommendations contained in this report are based on our field observations and limited laboratory tests. It is possible that soil conditions could vary.

We have prepared this report in substantial accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our study.

Only the County of Fresno Department of Public Works and Planning may use this report only for the purposes stated, within a reasonable time from its issuance. Land use, on-site conditions, or other factors may change over time.

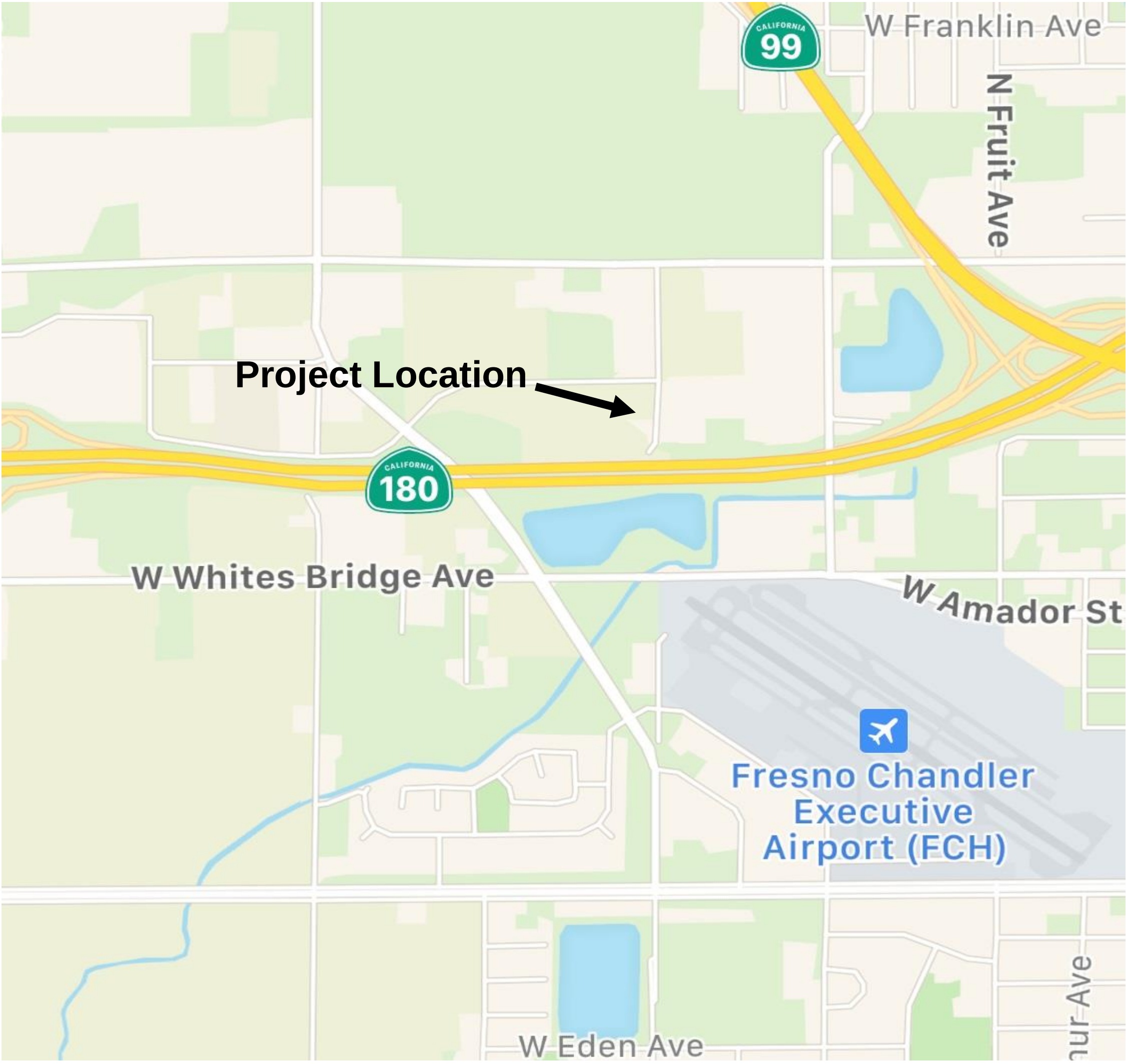
14 GENERAL REFERENCES

AASHTO Guidelines for Traffic Data Programs (2009)

Caltrans HDM, Chapter 600 and Chapter 800

FHWA Traffic Monitoring Guide (April, 2013)

APPENDIX A
VICINITY MAP



APPENDIX B
SITE MAP

APPENDIX C
BORING LOGS

APPENDIX D
LABORATORY TEST RESULTS

Lab No.	Passing #200 (%)	SE	R-Value	PI	USCS
19-0231	39	16	-	-	SM
19-0232	44	15	-	-	SM
19-0233	36	16	-	-	SM
19-0234	36	21	70	-	SM
19-0236	35	18	-	-	SM
19-0237	12	16	-	-	SM
19-0238	35	16	-	-	SM
19-0239	40	15	-	-	SM
19-0240	33	20	-	-	SM
19-0241	24	47	-	-	SM
19-0242	4	75	-	-	SM
19-0243	46	9	-	-	SM
19-0244	60	6	-	-	SM
19-0245	5	70	-	-	MH
19-0250	57	9	-	-	SM
19-0251	17	40	-	-	MH
19-0252	31	-	-	-	SM
19-0258	25	22	72	-	SM
19-0262	78	-	-	-	MH
19-0288	36	20	-	-	SM
19-0299	31				SM
19-0301	-	-	-	10	CL
19-0302	51	-	-		CL
19-0305	36	19	-	-	SM
19-0309	37	-	-	-	SM

Lab No.	Passing #200 (%)	SE	R-Value	PI	USCS
19-0310	45	-	-	-	
19-0311	-	-	-	9	ML
19-0315	36	19	-	-	SM
19-0324	36	19	-	-	
20-0222	39	-	-	-	
20-0225	30	-	-	-	
20-0226	54	-	-	-	
20-0227	75	-	-	-	
20-0228	30	-	-	-	
20-0229	35	-	-	-	
20-0241	28	-	-	-	
20-0242	4	-	-	-	
20-0243	57	-	-	-	
20-0244	41	-	-	NP	
20-0245	31	-	-	-	
20-0246	35	-	-	-	
20-0247	2	-	-	-	
20-0248	71	-	-	-	CL-ML
20-0249	56	-	-	8	CL-ML
20-0250	51	-	-	-	
20-0251	35	-	-	-	
20-0252	37	-	-	-	
20-0256	74	-	-	17	MH
20-0258	70	-	-	5	CL-ML

SM: Silty Sands, poorly-graded sand-gravel-silt mixtures
CL: Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clay
ML: Inorganic silts & very fine sands, silty or clayey fine sands, clayey silts with slight plasticity
MH: Inorganic silts, micaceous or diatomaceous fine sand or silt