April 29, 2020

Port of Stockton
2201 West Washington St.
Stockton, CA 95203

Attn: Juan Villanueva
Director of Development and Planning

Re: Subject: Geotechnical Review and Recommendations
Project Name: Fyffe Avenue Grade Separation
Project Address: Port of Stockton
Stockton, California
Terracon Project Number: NA205020

Dear Juan:

Our office was requested to review and offer a second opinion regarding geotechnical and foundation recommendations for the subject project. Existing recommendations are contained in the recently completed *Draft Foundation Report Fyffe Avenue Overcrossing Over UPRR Railroad* prepared by Twining, dated January 30, 2020, Project No. 190892.5. This request was made due to the presence on our staff of senior geotechnical engineer, Mr. Ron Heinzen, formerly with Kleinfelder's Stockton office. Mr. Heinzen has a long history and extensive experience with soil conditions and foundation construction practices at the Port of Stockton.

In our initial review and based on Mr. Heinzen experience with two similar past projects (US Gypsum Test Embankment and Expressway Grade Separation) we determined there was a potential for significant costs savings by using a geogrid reinforced cement treated soil section underlying the embankment fills to eliminating the need for stone columns and reduce embankment side slopes. This information was conveyed to HDR, the design engineer, and the owner. Terracon was subsequently authorized to perform a more thorough review of project geotechnical data contained in the Twining report and provide our own geotechnical and foundation recommendations. The Twining report includes data from a preliminary study performed by Wallace Kuhl and Associates (WKA). Four test borings and associated lab tests and two CPT soundings were performed by WKA. Ten additional CPT soundings were performed by Twining. This report presents the results of our review of this data, analysis, and subsequent recommendations.

A description and location of the project is contained in Sections 1.2 and 1.3 of the Twining report. A copy of the report is attached as Appendix E and should be considered part of this report. In addition to the published report, our office was provided with the raw CPT data files for the ten soundings performed by Twining. Due to the volume of existing field and laboratory data we did not feel it was necessary to perform additional field testing as it relates to site characterization. We concur with Twining's description and summary of site geology, seismicity, subsurface soil description, and soil corrosivity contained in their report. Our review, analysis, and recommendations encompassed the following tasks:

1. Perform an embankment settlement analysis
2. Compare settlement analysis to historical embankment performance of Expressway Overcrossing and US Gypsum Test Fill.
3. Re-run CPT liquefaction analysis.
4. Perform slope stability analysis
5. Perform pile foundation analysis and provide pile capacity/design recommendations
6. Provide recommendations for embankment design and construction
7. Provide recommendations for various types of retaining walls
8. Provide recommendations for geotechnical instrumentation monitoring
9. Plan Review and Limitations

An additional item to be performed but not part of this report is the investigation of potential borrow sites for suitable embankment fill and the design for cement treated soil fill overlying a geogrid layer. This is to be performed and results presented in a subsequent report.

1. Embankment Settlement Analysis

As indicated in Section 4.2 of the Twining report the project site is "underlain by interbedded and discrete layers of loose to medium dense sands and soft to very stiff silts, clayey silts, and occasional low to high plastic clays". This highly variable and ultimately discontinuous nature of soil strata exists even between the relatively close spacing of some of the test locations. However, this condition would be expected from the historical meandering depositing of sediments of the adjacent San Joaquin river. What this means is that it is difficult to reasonably quantify the magnitude and location of anticipated settlement. In our opinion the best means would be the past performance of embankment fills in the general area. Two examples are discussed in the next section. However, we did perform a settlement analysis based on the CPT soundings, since these would best account for the variable soil conditions.

The program CPet-IT, v3.0.1.2 by GeoLogismiki was used to compute estimated settlement for the ten Twining CPT soundings under two loading conditions: the full 35-foot embankment height (2.0 tsf) and one-half height (1.0 tsf). A 65-foot-wide embankment was used. The following **Table 1, CPT Settlement Analysis Results** presents a summary of computed settlements for primary (assigned as 1 month) and secondary (assigned as 12 months). Please note the results are only for the depth of the individual CPT sounding, which varied in depth from 56 to 125 feet. However, reviewing the cumulative settlement vs. depth plots show most of the settlement takes place in the upper 40 feet of soil. Program output plots for each of the CPT locations are contained in the attached Appendix A, CPT Settlement Analysis Graphical Plots.

TABLE 1, CPT Settlement Analysis Results							
CPT Location	CPT Depth (ft)	Maximum Embankment Height			1/2 Embankment Height		
		Computed Primary Settlement (in.)	Computed Secondary Settlement (in.)	Total Settlement (in.)	Computed Primary Settlement (in.)	Computed Secondary Settlement (in.)	Total Settlement (in.)
1	60	3.5	0.2	3.7	1.8	0.2	2
2	125	4.9	0.4	5.3	2.2	0.2	2.4
3	100	7.5	0.5	8	3.6	0.4	4
4	125	5.5	0.3	5.8	2.6	0.2	2.8
5	125	6.7	0.4	7.1	3.2	0.3	3.5
6	120	6.1	0.4	6.5	3	0.3	3.3
7	50	5.4	0.2	5.6	2.7	0.2	2.9
8	101	5.7	0.5	6.2	2.4	0.2	2.6
11	66	6.1	0.2	6.3	3.1	0.2	3.3
13	56	6.9	0.3	7.2	3.4	0.3	3.7

The results predict post construction settlement will be between 3 to 8 inches. WKA performed laboratory consolidation tests on two clay samples obtained from test borings R-18-001 and 003 at depths of 26 and 15 feet respectively. Samples were representative of a 19-foot stratum of sandy lean clay and a 17-foot stratum of lean clay respectively. Settlement calculations from these tests produced results similar to those shown in Table 1.

Our results also predicted most of the settlement (primary) will occur during embankment fill placement and within the first month of its topping out. This is due to the numerous interbedded sand lenses allowing for dissipation of excess pore water pressure. There should be some minor (secondary) settlement occurring the next 12 months but this is predicted to be less than ½ inch. We recommended geotechnical instrumentation be installed, as presented in Section 8, to monitor subsurface pore water pressure during embankment fill placement and track the quantity and rate of settlement. If pore pressures become elevated during fill placement, a one to two week break part way through placement may be required to allow pressure to dissipate.

2. Comparison of Settlement Calculations with Past Adjacent Embankment Fill Projects

Considerable engineering judgment is always required when computing and predicting actual site settlement in the field. Fortunately, at the Port of Stockton, two previous embankment projects were completed where instrumentation and settlement plate information are available. The first project was a 21-foot-tall test fill completed for the US Gypsum (USG) project located just north of Fyffe Avenue approximately ½ mile west of the current overcrossing project. The second project was the Port of Stockton Expressway/BNSF Grade Separation Project. The Expressway was previously known as Daggett Road. This project involved 40-foot-tall approach embankments and was completed in 2012. Both the USG and Expressway projects were instrumented with settlement plates, vibrating wire piezometers, and vibrating wire settlement cells. Based on readings from these instruments and actual survey measurements, we know the 21-foot-tall embankment at the USG site settled 6 to 8 inches while the 40-foot-tall embankment for the Expressway project settled slightly less than 12 inches. At the USG site, most of the settlement occurred within 1 to 2 weeks after fill placement. At the Expressway site, high pore pressure readings after approximately half the fill was placed caused the contractor to delay final fill placement for several weeks.

The foregoing instrumented case histories are in reasonably close agreement with the results of our settlement analysis from the CPT data. Although we anticipate a maximum settlement of 8 inches under the highest portion of the project embankment we recommend a value of 3.0 inches per 10 feet of embankment height (10 inches at maximum height) be used in the bid documents since contractors will need to estimate how much additional import may be needed to make final grade.

3. CPT Liquefaction Analysis

The program CLig, v3.0.1.7 by GeoLogismiki was used to estimate post liquefaction settlement at the site. Our theoretical results were similar to those contained in the Twining report of 3 to 8 inches. That said, we are not aware of any confirmed instance of liquefaction in the Central Valley, nor is the site currently located in a liquefaction hazard zone as reported by California Geological Survey. While Mr. Heinzen was at Kleinfelder, a review of historical articles following the 1906 EQ did not mention any damage that could be associated with liquefaction in the Stockton area. However, in our opinion it is still prudent to address the issue of liquefaction for the project embankments but there are alternatives to the stone columns recommended in the Twining report. According to the Twining report, “The larger predicted settlements are within what appears to be a north-south trending band identified by CPT sounding CPT-5, CPT-11, and WKA CPT-180002 and CPT sounding CPT-8 on the south.” They surmise that this band might be an old channel of some kind. This “band” exhibited an estimated 8-inches of post-liquefaction induced settlement while elsewhere the estimated settlement was on the order of 3-inches. In other words, there appears to be some “confinement“ of the area most prone to movement, especially considering the discontinuous and variable nature of the underlying soil strata. In our opinion, another option to minimize this risk is to bridge over this “band”. This can be accomplished by placing a layer of geogrid over the embankment footprint followed with 4-feet of aggregate and cement treated soil. Essentially, this would create a “raft” that would minimize or even eliminate any surface expression caused by liquefaction in these loose, partially confined, sand deposits. Specific recommendations for geogrids and cement treatment are contained in Section 6 of this report.

4. Slope Stability Analysis

The Twining report contained two tables of soil strata/strength parameters, one for the south embankment (Table 5) and one for the north embankment (Table 6). After performing our own analysis of the CPT data as well as review of the test boring logs and laboratory tests it is our opinion the Twining values were overly conservative. Consequently, we produced our own table of soil strata/strength parameters. These are contained in the following **Table 2, Soil Strength Parameters**. Results from our CPT analysis used to help develop these are contained in Appendix B, Select CPT Soil Strength Parameters. It should be understood there is substantial variation in soil stratigraphy throughout the site requiring considerable engineering judgement to determine any values to be used in an analysis.

Table 2, Soil Strength Parameters

Depth		Elevation		Soil	Est. Blowcount	Friction	Cohesion
From	To	From	To	Type	(N_{60})	Angle	(psf)
South Abutment (R-18-003, CPT-4)							
0	17	7.5	-10	Clay	10	-	1000
17	26	-10	-19	Clay	19	-	1500
26	32	-19	-25	Sand	8	32	-
32	45	-25	-38	Clay	20	-	1500
45	50	-38	-43	Sand	16	36	
50	64	-43	-57	Clay	15	-	2000
64	74	-57	-67	Sand	30	38	
74	94	-67	-87	Clay	24	-	3000
94	105	-87	-98	Sand	40	36	
105	125	-98	-118	Clay	37	-	5000
North Abutment (R-18-002, CPT-5)							
0	11	4.5	-7	Clay	4	-	700
11	21	-7	-17	Sand	10	32	-
21	28	-17	-24	Clay	8	-	1000
28	40	-24	-36	Sand	12	33	-
40	46	-36	-42	Clay	20	-	1500
46	57	-42	-53	Sand	16	34	-
57	61	-53	-57	Clay	16	-	2000
61	71	-57	-67	Sand	40	38	-
71	89	-67	-85	Clay	38	-	3000
89	125	-85	-121	Clay	45	-	5000

The program Slope/W v8.16 by GeoStudio was used to perform two dimensional static and dynamic (seismic) loading analysis for the proposed approach embankments. The program was set to use the Bishop method. The analysis was performed twice, once for the soil stratigraphy and strength parameters provided in the Twining report and once for the soil stratigraphy and strength parameters listed in the foregoing Table 2. Seismic values provided in the Twining report were utilized. Side slopes of the embankments were set at 2:1 with an underlying layer of geogrid and 4 feet of aggregate/cement treated soil fill. Results of our analysis, summarized as Factor of Safety (FS) against failure, are presented in the following **Table 3, Slope Stability Analysis Results**. Graphical plots of program output are contained in Appendix C, Slope Stability Analysis Graphical Plots.

Table 3, Slope Stability Analysis Results

Location	Terracon Soil Parameters		Twining Soil Parameters	
	Static FS	Seismic FS	Static FS	Seismic FS
South Embankment	2.09	1.61	1.76	1.37
North Embankment	1.99	1.48	1.57	1.16

The analysis shows the proposed embankments, with 2:1 side slope and an underlying geogrid/cement treated mat, are stable for both Twining and Terracon soil strata/strength parameters. Stability is defined as a FS > 1.5 for static conditions and FS > 1.1 for seismic. The stability analysis should be verified once the borrow investigation for the embankment fill is performed.

5. Pile Capacity/Design Recommendations

Recommended pile foundation designed recommendations are discussed and presented in the following sections:

a) Foundation type
b) Foundation Data Provided by Structural Engineer
c) Pile Group Efficiency Factor
d) Liquefaction Induced Downdrag
e) Pile Design Recommendations
f) Lateral Pile Capacity
g) Passive Resistance on Pile Cap
h) Pile Installation and Construction Considerations

a) Foundation Type

It is proposed to support the bridge abutments on 14-inch square Caltrans Standard Class 200 precast prestressed concrete driven piles.

b) Foundation Data Provided by Structural Engineers

Based on California Amendments to AASHTO LRFD Bridge Design Specifications, LRFD Service-I Limit State, Strength Limit State, and Construction Limit State load combinations should be used for design of the abutment footings.

The foundation design data sheet is presented in **Table 4, Foundation Design Data Sheet**. Foundation factored design loads were provided by the structural designers and are presented in **Table 5, Foundation Factored Design Loads**. The information presented in Tables 4 and 5 follow the latest Caltrans Memo to Designers 3-1 (June 2014).

Table 4, Foundation Design Data Sheet

Location	Pile Type	Finished Grade Elevation (feet)	Cut-Off Elevation (feet)	Pile Cap Size (feet)		Permissible Settlement under Service Load (inch)	Number of Pile per Support
				B	L		
S. Abut	14" sq. PC/PS Concrete Class 200	+5	-3	10	93	1	51
N. Abut		+5	-3	10	93	1	45

Table 5, Foundation Factored Design Loads

Location	Service-I Limit State (kips)		Strength/Construction Limit State (kips)			
			Compression		Tension	
	Total Load Per Support	Perm. Loads per Support	Per Support	Max. Per Pile	Per Support	Max. Per Pile
S Abutment	160	135	245	280	20	140
N Abutment	165	140	255	280	20	140

c) Pile Group Efficiency Factor

According to Section 10.7.3.9 (AASHTO, 2012), a pile-group efficiency factor (GEF) of 0.65 is used for a center-to-center pile spacing of 2.5 times the pile diameter, and a GEF of 1.0 is used for a center-to-center spacing of 4.0 times the pile diameter or greater. Linear interpolation can be used to determine the GEF for intermediate spacing. Based on the layouts provided by the structural designers, GEFs of 1.0 were used for the abutment piles.

d) Liquefaction Induced Downdrag

As discussed in Section 3, there is a potential for seismic induced liquefaction at the project site. If liquefaction was to occur, it could produce potential downdrag on the piles. However, our opinion is the potential for liquefaction around the piles and subsequent downdrag force is negatable for the following reasons:

1. Phase I will be construction of approach embankments. Phase II will be construction of the bridge abutments with pile foundations. Phase II will not commence until after primary settlement of the embankment has occurred. Phase I will leave a 35-foot-high vertical embankment edge located immediately adjacent to the abutment pile footing line. Estimated settlement of the underlying native soil stratum is 10-inches or less. This settlement will induce densification and consolidation of the underlying sand and clay stratum, thus decreasing the sand stratum potential for liquefaction.

2. Significant densification of sand stratum will occur during driving of the proposed displacement piles. It is estimated densification ($\leq$ 10%) will extend a minimum of 2 to 3 feet in radius around each pile. This area of densification will help “column” a non-liquefiable area through the relatively discontinuous and discrete potential liquefiable sand layers encountered in the test holes.

   This conclusion is also based on two past nearby projects with similar soil conditions, the 1) SOHIO project, located ½ mile north on the other side of the deep-water channel and 2) Navy Dr. bridge, located 300-yards SE of site. For the SOHIO project similar 50-foot long, 14-inch square, displacement piles where used. Driving densification was so significant intermediate piles encountered driving refusal and had to be trimmed. For the Navy Dr. bridge project similar pile refusal was also encountered. Furthermore, records show after a 72-hour “setting” period, re-tapping of piles produced driving rated capacities often double the design capacities.

e) Pile Design Recommendations

The nominal resistances for both abutment piles are controlled by the Strength Limit State “Maximum Per Pile Load”. The axial shaft capacities were determined using side friction components of resistance. Furthermore, the pile capacity is also based on soil resistance only and may be further limited by the pile-head connection details and the strength of the pile materials.

AllPile® by CiviTech was used to perform pile analysis and compute axial capacity for abutment foundation design recommendations. These recommendations are presented in **Table 6, Foundation Design Recommendations**. The pile data Table for the contract plans is presented in **Table 7, Pile Data Table**. Appendix D contains program output for both the north and south abutments.

Table 6, Foundation Design Recommendations

Location	Pile Type	Cut-Off Elevation (feet)	Service-I Limit State Load per Support (kips)		Total Perm. Support Settlement (inch)	Required Factored Nominal Resistance (kips) Strength/ Construction		Design Tip Elevation (feet)	Spec. Tip Elevation (feet)	Req. Driving Resistance (kips)
			Total	Perm.		Compression (f =0.7)	Tension (f =0.7)			
S Abut	Class 200 14" Sq PC/PS Con.	-3	160	135	1	280	140	-58 (a) -34 (b) -30 (d)	-58	400
N Abut		-3	165	140	1	280	140	-58 (a) -40 (b) -30 (d)	-58	400

Notes:
1. Design tip elevations are controlled by the following demands: (a) Compression (Strength Limit), (b) Tension (Strength Limit), and (d) Lateral Load.
2. The Specified tip elevations shall not be raised above the design tip elevations for tension, lateral and tolerable settlement.

Table 7, Pile Data Table

Location	Pile Type	Nominal Resistance (kips)		Design Tip Elevation (feet)	Specific Tip Elevation (feet)	Required Driving Resistance (kips)
		Compression	Tension			
S. Abutment	14" sq. PC/PS Concrete Class 200	400	200	-58 (a) -34 (b) -30 (d)	-58	400
N. Abutment	14" sq. PC/PS Concrete Class 200	400	200	-58 (a) -40 (b) -30 (d)	-58	400

Notes:
1. Design tip elevations are controlled by the following demands: (a) Compression, (b) Tension, (c) Settlement and (d) Lateral Load.
2. The Specified tip elevations shall not be raised above the design tip elevations for tension, lateral and tolerable settlement.

f) Lateral Pile Capacity

Recommended soils parameters for lateral load analysis of driven piles have been developed for use in the LPILE computer program and are provided in the following **Table 8, Soil Strength Parameters for Lateral Load Analysis**.

Table 8, Soil Strength Parameters for Lateral Load Analysis							
Depth		Effective Unit Weight (pcf)	Soil Type	Friction Angle	Cohesion (psf)	Coeff. of Static Subgrade Reaction K_s (pci)	Non-default Strain Factor ε_{50}
From	To						
South Abutment (R-18-003, CPT-4)							
0	10	100	Soft Clay	-	1000		0.010
10	17	38	Soft Clay	-	1000	62	0.010
17	26	46	Stiff Clay without Free Water	-	1500	-	0.007
26	32	49	Sand	32	-	62	
32	45	56	Stiff Clay without Free Water	-	1500	-	0.007
45	50	56	Sand	36		70	
50	64	56	Stiff Clay without Free Water	-	2000	-	0.005
64	74	56	Sand	38		75	
74	94	56	Stiff Clay without Free Water	-	3000	-	0.005
94	105	56	Sand	36		70	
105	125	56	Stiff Clay without Free Water	-	5000	-	0.004
North Abutment (R-18-002, CPT-5)							
0	11	100	Soft Clay	-	700	62	0.010
11	21	46	Sand	32	-	-	
21	28	67	Soft Clay	-	1000		0.010
28	40	49	Sand	33	-	65	
40	46	56	Stiff Clay without Free Water	-	1500	-	0.007
46	57	56	Sand	34	-	65	
57	61	56	Stiff Clay without Free Water	-	2000		0.005
61	71	56	Sand	38	-	75	
71	89	56	Stiff Clay without Free Water	-	3000	-	0.005
89	125	56	Stiff Clay without Free Water	-	5000	-	0.004

The depth below ground surface indicated in the soil parameters table is referenced from the existing site surface at the time of the field exploration. The required depths of pile embedment should also be determined for design lateral loads and overturning moments to determine the most critical design condition. Based on the pile layout, p-multiplier should be calculated following methodologies presented in the California Amendments to AASHTO LRFD Bridge Design Specifications.

Lateral load design parameters are valid within the elastic range of the soil. The coefficient of subgrade reaction are ultimate values; therefore, appropriate factors of safety should be applied in the pile design or deflection limits should be applied to the design.

It should be noted that the loaded capacities provided herein are based on the stresses induced in the supporting soils. The structural capacity of the piles should be checked to assure that they can safely accommodate the combined stresses induced by axial and lateral forces. Furthermore, the response of the piles to lateral loads is dependent upon the soil/structure interaction as well as the piles' s actual diameter, length, stiffness and "fixity" (fixed- or free-head conditions).

g) Passive Resistance on Pile Cap

To help resist lateral loads pile cap footings may utilize passive soil pressure acting on the side of the footing. Passive pressure may be computed by utilizing an equivalent fluid pressure of 300 pounds per cubic foot (pcf) above the 6-foot depth and 145 pcf below the 6-depth. The top 1.0 foot of depth should not be considered when computing passive resistance. The 6-foot depth is representative of the maximum height of ground water.

h) Pile Installation and Construction Considerations

We concur with the Twining report that the contractor should submit and perform a pile driveability study as discussed in Section 10 of their report (see Appendix E).

Pile installation should be performed in accordance with Section 49 of the latest Caltrans 2018 Standard Specifications. Although the general area is sparsely occupied, it is suggested as a minimum some initial vibration monitoring be done to document the level of vibrations generated from pile driving operations. If there is any concern for nearby structures a pre-construction survey can be done to record existing conditions. Pre-construction surveys can include measuring and photographing cracks on the exterior and interior of existing buildings, on paved ground surfaces, and on other existing structures or improvements (walls, fences, above-ground utilities, etc.). Surveys should also be performed after construction to evaluate whether or not there are differences compared to pre-construction conditions. The pre- and post-construction surveys can be used to protect against claims from property owners.

Piles should be driven at least to the specified tip elevation and the bearing value should be checked with the pile-driving formula given in Section 49-2.01A(4)(c) of the Caltrans 2018 Standard Specifications. If the specified tip elevation is reached without achieving the nominal resistance, pile driving should continue until bearing is attained. In this case, it may be prudent to allow the pile to "set up" before re-striking to evaluate the pile capacity.

Based on the soil borings and past projects, hard driving could be anticipated below elevation -50 feet at the abutments and a potential for slower production rate. The selected pile-driving hammer should be able to deliver sufficient energy to drive the piles at a penetration rate of not less than 1/8-inch per blow at the required bearing value. Predrilling in native soils and vibratory hammers are not allowed for pile installation.

6. Embankment Design and Construction Recommendations

The one recommended design feature for the project that is different than the previously cited USG or the Expressway projects is the placement of a geogrid followed by 4 feet of aggregate/cement treated soil. This design feature is recommended to mitigate the risk of liquefaction induced settlement and allow for embankment stability with 2:1 side slope. Another benefit of this feature is to reduce settlement by spreading out design loads and mitigate the effect of differential settlement due to varying soil stratigraphy, such as the apparent isolated zones of softer material reflected in the ten CPT probes and four soil borings. With this information considered, we recommend the following design features and procedures for project embankment design and construction:

a) Subgrade preparation
b) Geogrid layer
c) 8-inch aggregate layer
d) 40-inch cement treated soil layer
e) Imported embankment fill
f) Compaction requirements
g) Construction observations and testing

a) Subgrade Preparation

Prior to start of embankment construction the area designated as Limits of Work on the project plans should be cleaned and stripped of all debris, irrigation lines, old pavement, trees, brush, roots, and vegetable ruin. Grub out all large roots (1/2 inch or greater diameter) to a depth of at least two feet. Voids left from the grubbing operations should be filled with engineered fill. The vegetation materials and all materials from the cleaning and grubbing operation shall be removed from the site.

Minor cuts and engineered fills may be needed to present a subgrade that is relatively uniform in profile to allow for placement of geogrid and subsequent mixing and compaction of cement treated soil lifts. Once any necessary cuts have been made and prior to placing any engineered leveling fill, the exposed subgrade soil should be scarified, moisture conditioned, and compacted to the density specified in Table 6. The depth of scarification of subgrade soils and moisture conditioning of the subgrade is highly dependent upon the time of year of construction and the site conditions that exist immediately prior to construction. If construction occurs during the winter or spring, when the subgrade soils are typically already in a moist condition, scarification and compaction may only be 8 inches. If construction occurs during the summer or fall when the subgrade soils have been allowed to dry out deeper, the depth of scarification and moisture conditioning may be as much as 18 inches. A representative of our office should be present to observe the exposed subgrade and specify the depth of scarification and moisture conditioning required prior to placing leveling fill.

On-site soils, free of vegetation, debris, and fragments larger than two inches may be used as engineered leveling fill. All fill should be compacted to the density specified in Table 6. It is estimated the native soil scarified and compacted or used as leveling fill should experience 10% shrinkage.

b) Geogrid

We recommend Tensar TX-8 geogrid be used. Rolls should be placed perpendicular to abutment alignments. Manufacturer recommendations should be followed regarding grid placement and lapping. The grid should extend to the projected limits of embankment slopes.

c) Initial Aggregate Layer

The geogrid should be overlain by 8-inches of an angular aggregate layer with maximum 1.5-inch material and sufficient gradation and fines to produce a compact layer. The intent of this layer is to provide adequate interaction with the geogrid and a non-raveling working platform on which to perform the soil/cement mixing and compacting. Consequently, this layer may consist of pavement grindings, recycled concrete, pit run material, aggregate base, or any other material available that will meet the above stated requirements and intent. This layer should be compacted to the density specified in Table 6. A sample of any proposed material should be submitted to our office for approval prior to importing on-site.

d) Cement Treated Soil

The aggregate layer should be overlain with 40 inches of cement treated soil that has on unconfined compressive strength of at least 500 psi. It is our understanding the Port of Stockton desires material from local dredge spoils be utilized for embankment fill, which will include the cement treated section and should meet the standard Caltans specification for embankment fill. Our office is currently performing an investigation of potential borrow sites and material. Once this investigation is completed we will provide specific recommendations regarding the source and type of fill material and the percentage of cement required. However, for planning purposes 6% cement by a dry soil unit weight of 120 pound per cubic foot should be considered. Based on experience with the potential borrow sites it is anticipated the borrow material will experience 10% shrinkage from pre-excavation density to compacted cement treated section.

With the placement of the aggregate layer, it is anticipated that soil cement mixing and compacting can be performed in-place on top of the geogrid. The soil cement should be compacted to the density specified in Table 6.

e) Embankment Fill

See the above Section 6.d for source of fill. It is anticipated the untreated imported borrow fill material will experience 20% shrinkage from pre-excavation density to compacted embankment fill. Fill should be moisture conditioned and compacted to the density specified in Table 6. Please note required density increases for the top 12 inches of embankment fill that will act as roadway subgrade.

The edges of the embankments adjacent and parallel to the bridge abutments should be constructed vertical as a temporary wall with the use of geogrid and welded wire forms. See the following Section 7.d for design parameters.

f) Compaction Requirements

Compaction requirements are presented in the following **Table 9, Compaction Requirements**.

Table 9, Compaction Requirements

Material Type and Location	Per the Modified Proctor Test (ASTM D 1557)		
	Minimum Compaction Requirement (%)	Range of Moisture Contents for Compaction Above Optimum	
		Minimum	Maximum
Scarified subgrade soil beneath embankment:	90	-2%	+5%
Native leveling fill beneath embankment:	90	-2%	+5%
Angular material over geogrid	90	0%	+3%
Cement treated fill:	90	0%	+5%
Imported embankment fill:	90	-2%	+5%
Upper 12 inches subgrade beneath pavement:	95	-2%	+4%

g) Construction Observation and Testing

The earthwork efforts should be monitored under the direction of the Geotechnical Engineer. Monitoring should include documentation and testing of adequate removal of vegetation and topsoil, scarification and leveling of embankment subgrade, placement of geogrid, and placement and compaction of aggregate layer, soil cement layer, and all embankment fill.

Each lift of compacted fill should be tested, evaluated, and reworked, as necessary, until approved by the Geotechnical Engineer prior to placement of additional lifts. Each lift of fill should be tested for density and water content at a frequency of at least one test for every 5,000 square feet of the lift.

In addition to the documentation of the essential parameters necessary for construction, the continuation of the Geotechnical Engineer into the construction phase of the project provides the continuity to maintain the Geotechnical Engineer's evaluation of subsurface conditions, including assessing variations and associated design changes.

7. Retaining Wall Recommendations

Project retaining structures consist of bridge abutment walls, rigid wing retaining walls (extending roughly perpendicular and 15-feet past the abutments), segmental retaining walls (continuing past the wing walls and tapered up the embankment slope), and temporary/permanent retaining walls constructed with geogrids and welded wire forms (located next to and parallel with the bridge abutments.

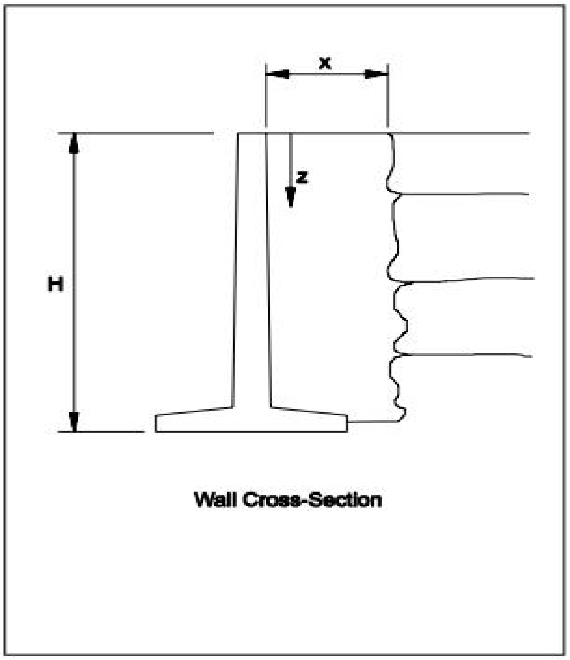
a) Abutment and Wing Retaining Walls

Bridge abutment and wing walls will be a special case of rigid retaining walls since they will be backed by self-supporting abutment fill walls, referred to as Temporary walls (see Section 7.c). The gap between the vertical face of the temporary embankment walls and the rear face of the abutment and wing walls will be backfilled with gravel. This gravel fill, referred to as a "silo" fill, will be the only source of lateral pressure on the walls.

The gap between the two walls will be 2-feet along the wings and 4-feet along the abutments. Estimated height of silo fill on top of the pile cap footing is 41-feet (40' gravel, 1' roadway). We have allowed for an additional 2-feet of height to account for surcharge live load. Gravel backfill should consist of ¾-inch clean washed gravel (ASTM D448, sizes 6, 67, or 68).

Lateral pressure has been computed utilizing the equation published in the Montana Department of Transportation (MDT) Geotechnical Manual, Chapter 17, Earth Retaining Systems (see Figure 1). The following parameters apply:

α_{sh} = horizontal silo pressure	calculated
x = distance between walls	2.0 and 4.0 feet
z = depth at which α_{sh} is calculated, max 43'	43 feet
K = coef. of lateral earth pressure	0.140
ϒ = unit wt. of backfill	110 pcf
δ = angle of friction between wall/fill, 2/3 Φ	32.7
Φ = backfill friction angle	49



$$P_{sh} = \frac{\gamma x}{2\tan\delta}\left[1-\exp\left(-2K\frac{z}{x}\tan\delta\right)\right]$$

where:

P_{sh} = horizontal silo pressure
x = distance between the walls
z = depth at which P_{sh} is calculated
K = coefficient of lateral earth pressure
γ = unit weight of the fill
δ = angle of friction between the wall and fill

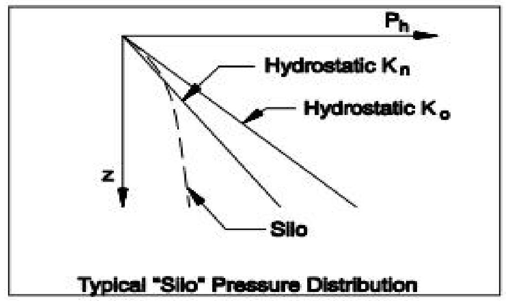


Figure 1, MDT Geotechnical Manual, Earth Retaining Systems, July 2008

Computed horizontal force produces a roughly trapezoidal loading. Therefore, the lateral loading diagram shown in Figure 2 should be used for the walls along with the values presented in Table 10. To account for surcharge load the top of the diagram should start 2-feet above the top of the roadbed.

Table 10, Lateral Wall Pressure

Wall	x, Silo Fill Gap, ft.	α_{sh}, Horizontal Silo Fill Pressure, psf
Wing	2.0	170
Abutment	4.0	300

Due to the limited width of silo backfill it is estimated lateral seismic loading from backfill on the abutment wall will be less than the seismic force on the wall from its self-weight, therefore no additional seismic lateral loading from the backfill needs to be considered.

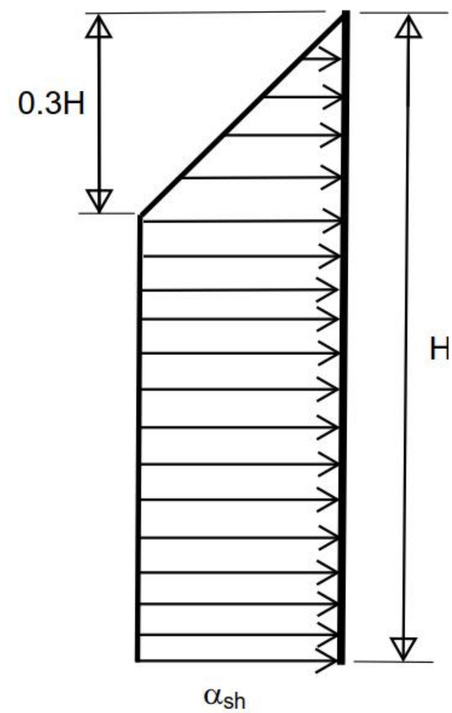


Figure 2, Abutment wall Loading Diagram

b) Segmental Retaining Walls

Segmental retaining walls derive their support by using geogrids or other means buried within the retained backfill with stackable pre-cast concrete members as facing. The following soil strength parameters should be used for the design of walls located within the embankment fills. These values are based on our experience with soil from the proposed borrow site. However, these values may be modified depending on the results of our borrow investigation.

Soil Type:	Clayey sandy silt
Soil friction angle:	$\Phi = 34^{o}$
Soil cohesion:	c = 200 psf
Soil unit weight:	Y_{WD} = 125 pcf
Bearing capacity:	q_{ult} = 4000 psf

c) Temporary Geogrid Retaining Walls

As part of Phase I embankment placement, temporary retaining walls, utilizing the fabric/geogrid supported walls, should be constructed at the edge of each embankment fill adjacent to the abutments and along the wing walls on either side. The temporary walls will create a vertical face parallel and adjacent to the edge of the pile cap foundations. Since the top of pile cap foundations will be 5 feet below existing grade, the first 5 feet of wall should be constructed prior to placement of the embankment geogrids and soil cement mat. These walls are also considered segmental retaining walls. If embankment fill is utilized to construct the walls the soil parameters as indicated in the foregoing Section 7.b should be used for design.

8. Geotechnical Instrumentation Recommendations

It is recommended the underlying native soil's pore water pressure and settlement be monitored by geotechnical sensors during and after construction to verify design recommendations and confirm when primary settlement has occurred. There is the potential for elevated pore water pressures to develop in underlying soil strata during embankment fill placement. If pore pressures become too high, it could weaken the soil to the point where embankment stability could be threatened.

It is recommended 4 vertical sensor arrays be installed in bore holes prior to on-site grading and 15 surface settlement monitoring points installed after Phase I embankment construction is completed. Figure 5 shows approximate instrument locations.

a) Vertical Sensor Array

Each vertical sensor array should consist of two vibrating wire piezometers (Geokon, model 4500) and one vibrating wire settlement system (Geokon, model 4600) installed and grouted into vertical bore holes. The settlement system consists of a vibrating wire piezometer, installed below the depth of anticipated settlement, connected by a liquid-filled tube to a reservoir attached to a settlement plate located at the ground surface. As fill is placed, the reservoir settles and the change in liquid pressure on the sensor is recorded. The advantage of this system is it does not provide any hindrance to construction and grading operations, since all components are buried (see Figure 4).

The piezometers to monitor soil pore water pressure should be located at approximately the 8- and 20-foot depths in the bore holes. These depths are subject to field adjustment since it is desired to locate them within underlying clay strata. The settlement piezometer should be located at the 60-foot depth. Once sensors are installed the borehole should be filled with cement/bentonite grout consisting 2.5-part water, 1.0-part cement, and 0.3-part bentonite (by dry weight). Signal cables from all sensors should be buried in trenches excavated below the depth of anticipated subgrade grading and carried to one of two stations located outside the limits of grading operations. One station will need to be on the north side of the RR track and one on the south side. Station location can best be determined by consultation with the general contractor so as to minimize disruption to their work. Each station should consist of a vibrating wire reader/data logger and telemetry with solar recharge to provide real-time hourly readings posted to the Cloud of all sensors.

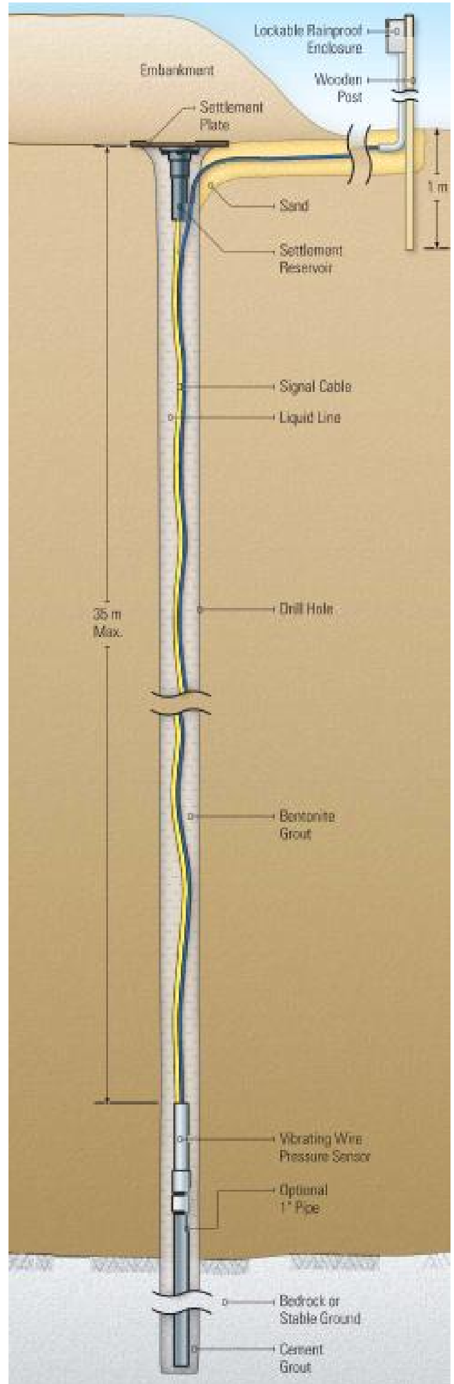


Figure 3, Subsurface Settlement System

b) Surface Settlement Monitoring Points

Immediately after Phase I embankment grading is completed surface settlement monitoring points should be installed. Points should consist of a 3-foot length of #4 rebar driven into the fill with 2-inches protruding. If there is the chance on-going construction activity will disturb them, they should be driven to 3-inches below grade and protected with at-grade Christy boxes. Elevations of each point should be surveyed immediately after installation, every three days for the first two weeks, and weekly after that until movement has stabilized. It is anticipated the weekly readings would extend for 2 months.

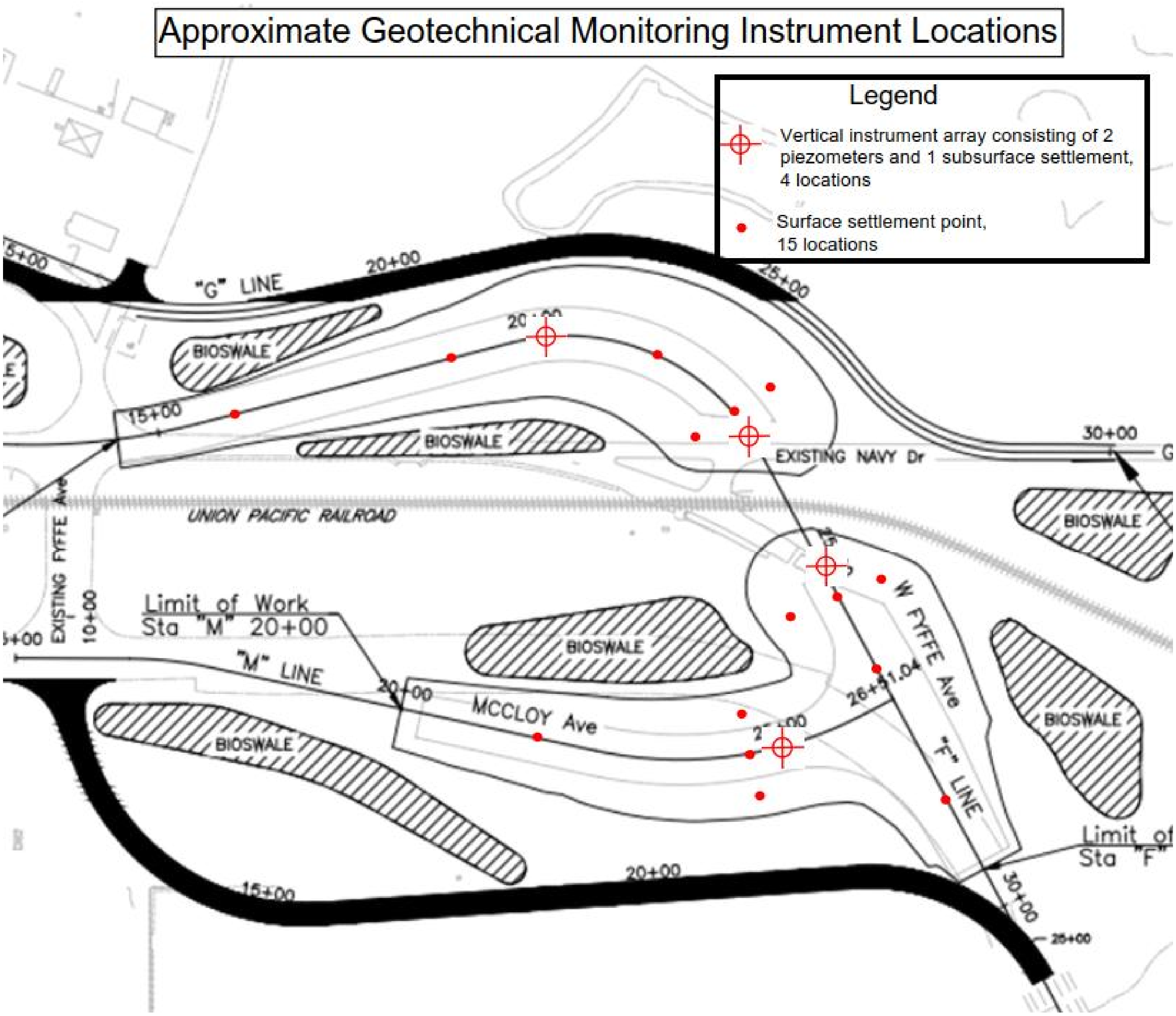


Figure 4, Instrument Locations

9. Plan Review and Limitations

It is recommended that our office be afforded the opportunity to review the completed project plans and specifications to verify our recommendations have been properly interpreted and incorporated. Our office cannot be held responsible for errors that may result from lack of a review process. Results of the review will be submitted in writing.

Our analysis and opinions are based upon our understanding of the project, our experience with geotechnical conditions in the area, and the stated exploration and laboratory data reviewed and used. Natural variations will occur between exploration point locations or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. Terracon should be retained as the Geotechnical Engineer, where noted in this report, to provide observation and testing services during pertinent construction phases. If variations appear, we can provide further evaluation and supplemental recommendations. If variations are noted in the absence of our observation and testing services on-site, we should be immediately notified so that we can provide evaluation and supplemental recommendations.

Our Scope of Services does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

Our services and any correspondence or collaboration through this system are intended for the sole benefit and exclusive use of our client for specific application to the project discussed and are accomplished in accordance with generally accepted geotechnical engineering practices with no third-party beneficiaries intended. Any third-party access to services or correspondence is solely for information purposes to support the services provided by Terracon to our client. Reliance upon the services and any work product is limited to our client and is not intended for third parties. Any use or reliance of the provided information by third parties is done solely at their own risk. No warranties, either express or implied, are intended or made.

Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly impact excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety, and cost estimating including, excavation support, and dewatering requirements/design are the responsibility of others. If changes in the nature, design, or location of the project are planned, our conclusions and recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing.

Sincerely,
Terracon Consultants, Inc.

Neil O. Anderson, Principal
CA Geotechnical Engineer 2245

REGISTERED PROFESSIONAL ENGINEER
NEIL O. ANDERSON
No. 2245
04/29/2020
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Ron Heinzen, Senior Engineering Consultant
CA Geotechnical Engineer 388

Raj Pirathiviraj, Senior Engineer
CA Civil Engineering 71662

Authorized Project Reviewer: Garret S.H. Hubbart, GE2588

Attachments:

Appendix A	CPT Settlement Analysis Graphical Plots, 20 pages
Appendix B	Select CPT Soil Strength Parameters, 10 pages
Appendix C	Slope Stability Analysis Graphical Plots, 8 pages
Appendix D	Pile Axial Analysis, 8 pages
Appendix E	*Draft Foundation Report Fyffe Avenue Overcrossing Over UPRR Railroad* prepared by Twining, dated January 30, 2020, 171 pages