



GEOTECHNICAL & ENVIRONMENTAL ENGINEERING — CONSTRUCTION TESTING & INSPECTION

September 11, 2019

TES No. 190517.001

Mr. Orfil Muniz
A&M Consulting Engineers
204 E. Oak Avenue, Suite 5A
Visalia, CA 93291
Phone: 559.429.4747
Email: orfil@am-engr.com

Project: Industrial Development
Hwy 41 and Idaho Avenue
Lemoore, California

Subject: Supplement to Geotechnical Engineering Investigation

Dear Mr. Muniz,

In accordance with your request, **TECHNICON Engineering Services, Inc. (TECHNICON)** prepared this supplemental letter to address the use of a previously prepared Geotechnical Investigation Report (GIR) for the Industrial Development project (reference file Tes No. 180204.001, dated May 15, 2018). The purpose of the supplemental investigation was to develop recommendations and opinions to aid in project design and construction involving mass grading of the site, grading for two roadways, Road "A" that will run north-south from Enterprise Drive and Road "B" that will run east-west from S 19th Avenue to Road "A", storm water and sewer pipelines (anticipated maximum depths of 20 feet), a sewer and storm drain lift station (anticipated depth of 20 to 40 feet), and a storm water detention basin.

Foundations and Lateral Earth Pressures for Lift Station Wet Well

The proposed industrial buildings may be supported by conventional shallow spread footings and lift station wet well foundations on properly engineered fill and soil. The following recommendations are based on the assumptions that were recommended in the referend report in Section 5, "Earthwork". Recommendations regarding the geotechnical aspects of building design are presented in subsequent sections.

The investigation has revealed a varying range of soils from non-expansive to moderately expansive throughout the area of interest. Based on the potentially expansive nature on the onsite soils, it is recommended interior and exterior footings be designed in accordance with the specifications in Table 1 below. It is recommended that expansion potential testing be performed after site grading has been completed in order to determine the expansive potential of the building pads. Foundations depths and reinforcement should also satisfy structural and constructability considerations.

CORPORATE OFFICE — 4539 N. Brawley Avenue #108, Fresno, CA 93722 — P 559.276.9311 — F 559.276.9344

VISALIA OFFICE — P.O. Box 4093, Visalia, CA 93278 — P 559.732.0200

MERCED OFFICE — 2345 Jetway Drive, Atwater, CA 95301 — P 209.384.9300 — F 209.384.0891

www.technicon.net

TABLE 1
FOUNDATION EMBEDMENT AND REINFORCEMENT

Soils		Minimum Embedment (inches)		Reinforcement
PI	EI	Interior	Exterior	
< 9	< 20	12	12	Per 2016 CBC/CRC
9 to 15	20-40	12	18	1-#4 Bar Top & Bottom
16 to 25	41-80	18	18	1-#4 Bar Top & Bottom

Generally two factors determine the design bearing pressure for conventional spread footing foundations: strength of the foundation soil and tolerable settlement. For lightly loaded structures, design bearing may be dictated by code-required minimum footing geometry or constructability considerations.

Due to the lift station being approximately a ¼ mile away from the closest boring and with the water table being relatively shallow throughout the area of interest, the foundation bearing capacity for the lift station wet well was designed using two (2) cases, shown in Figure 1. Case 1 is based on the assumption that the water table is located above or close to the foundation. Case 2 is based on the assumption that the water table is located well below the foundation. As a result of borings having different water table elevations, soil profiles and properties throughout the site area and the high probability of soil discrepancy between the closest boring and the location of the lift station wet well, it is recommended to not use these values until proper borings are done at the specific location of the lift station wet well.

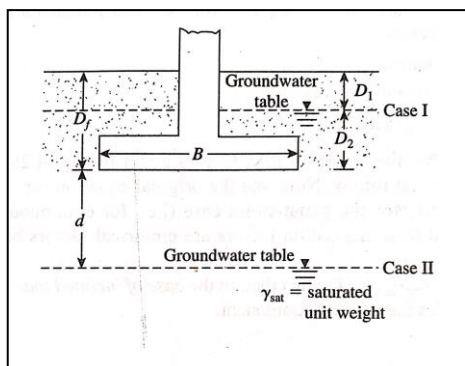


Figure 1: Bearing capacity modification for water table (Das 2012)

The available bearing capacity, based only on the shear strength of the soil, will be dependent upon the footing geometry and different case scenarios. Presented in Table 2 are the allowable bearing capacity (shear strength considerations only) for static loading (D.L + L.L.), total combined loading (D.L + L.L. + transient loading, such as wind or seismic), and unfactored nominal bearing.

**TABLE 2
 BEARING CAPACITY**

Shape	Loading	Bearing Capacity (psf)	
		Case 1 ($0 < D_1 < D_f$)	Case 2 ($d > B$)
Square	Static Loading	$808 D_1 + 503 D_2 + 171 B$	$970 D + 331 B$
	Total Combined Loading	$1211 D_1 + 754 D_2 + 257 B$	$1455 D + 496 B$
	Unfactored Ultimate Bearing	$2423 D_1 + 1509 D_2 + 514 B$	$2911 D + 992 B$
Circular	Static Loading	$808 D_1 + 503 D_2 + 129 B$	$970 D + 248 B$
	Total Combined Loading	$1211 D_1 + 754 D_2 + 193 B$	$1455 D + 371 B$
	Unfactored Ultimate Bearing	$2423 D_1 + 1509 D_2 + 386 B$	$2911 D + 743 B$

Note: For Case 1: D_1 is depth above groundwater table (ft), D_2 is depth below groundwater table to base of footing (ft), B is the footing width (ft). For Case 2: D is the footing depth (ft) and B is the footing width (ft).

The above expressions are appropriate for design using the Basic and Alternative Load Combinations in Section 1605.3 of the 2016 CBC.

Lateral loads applied to foundations can be resisted by a combination of passive lateral bearing and base friction. The allowable and ultimate passive pressures and frictional coefficients for the footings are presented in Table 3.

**TABLE 3
 PASSIVE PRESSURES AND FRICTIONAL COEFFICIENTS**

	Allowable		Ultimate
	Static	Total Combined	
Frictional Resistance	$0.31N + 395 \text{ psf}$	$0.37N + 527 \text{ psf}$	$0.47N + 790 \text{ psf}$
Passive Pressure (psf/ft)	$219 \text{ psf/ft} + 1,473 \text{ psf}$	$292 \text{ psf/ft} + 1,963 \text{ psf}$	$438 \text{ psf/ft} + 2,945 \text{ psf}$
Lateral Translation Needed to Develop Passive Pressure	$0.005 D$	$0.005 D$	$0.007 D$

Note: 1) D is the footing depth (ft), N is the normal load (psf)

Due to the expansive soil conditions, passive resistance should not be used within the top 18 inches of the footing unless abutted by hardscape. 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. The allowable values already incorporate a factor of safety and, would be compared directly to the driving loads. If analytical approaches require the input of a safety factor, the ultimate values should be used.

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

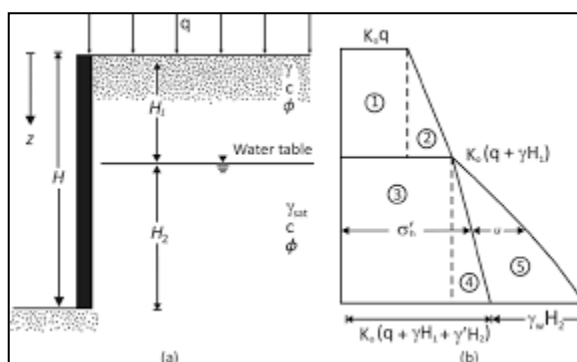


Figure 2: Earth pressure with water table located at depth $z < H$ (Das 2012)

The lateral earth pressure against retaining structures will be dependent upon the ability of the wall to deflect. Presented in Table 4 is the active, at-rest and braced lateral earth pressures for level on-site soil for two (2) cases, similar to the allowable bearing capacities for the foundation. The active pressure is applicable to walls able to translate 0.0005 radians at the top or bottom. The at-rest soil pressure is applicable to retaining structures that are fully fixed against both rotation and translation. Walls restrained from translation at the top and bottom, but able to deflect 0.0005 radian between restrained points should be designed for the braced lateral pressure.

**TABLE 4
 LATERAL EARTH PRESSURES**

Loading Conditions	Lateral Earth Pressures	
	Case I	Case II
Active Pressure (psf/ft of depth)	$18 H_1^2 + 47 H_1H_2 + 42 H_2^2$	43
Braced Pressure (psf)	$11 H_1^2 + 30 H_1H_2 + 27 H_2^2$	28H
At-Rest Pressure (psf/ft of depth)	$27 H_1^2 + 71 H_1H_2 + 48 H_2^2$	65

Note: Case 1: H_1 (ft) is the height above groundwater table, H_2 is the height below groundwater table.
 Case 2: H in the expression represents the retained height in feet (measured from finished grade to bottom of the footing)

Case 1 is based on the assumption that the retaining walls height measured from finished grade to bottom of footings is partially submerged in water due to shallow groundwater table. Therefore, earth pressures in Case 1 include hydrostatic pressures. Case 1 earth pressures do not include hydrostatic pressures; therefore, walls should be adequately drained to prevent the build-up of hydrostatic pressure. As a result of borings having different water table elevations, soil profiles and properties throughout the site area and the high probability of soil discrepancy between the closest boring and the location of the lift station wet well, it is recommended to not use these values until proper borings are done at the specific location of the lift station wet well.

Retaining wall foundation design can utilize the passive pressures and sliding resistance given in Table 3 and the allowable bearing capacity given in Table 2. When utilizing the available allowable bearing capacities of Table 3, the value for static loading would represent the average bearing for the footing and the value for total combined loading would present the allowable maximum toe pressure.

Buoyancy Resistance for Lift Station Wet Well

Buoyant force acting on a submerged object is equal to the weight of fluid which that object displaces, which in this case is applicable due to the high groundwater table around the area of interest. For an installed structure to actually “float”, or exhibit upward movement, the buoyant forces must overcome both the weight of the structure and the shear between the walls and the soil.

Two (2) vertical wet well structures for the Hess Storm water lift station were analyzed for flotation potential. According to the project plans provided by Mr. Orfil Muniz PE, of A&M Consulting Engineering, the shallow wet well structure’s base, which supports the ductile iron resilient wedge gate valves and iron swing check valves, does not extend past the walls of the wet well. This wet well has a smooth-wall design utilizing the weight of the structure itself and the downward friction resistance of the soil surrounding the wet well to resist the upward buoyant force. The deep wet well structure that supports the submersible storm water pump and pump discharge connection utilizes an extended base to provide additional resistance to buoyant forces.

The shallow wet well structure utilizes a square shaped geometry while the deep well utilizes a circular geometry. Presented in Table 5 are the physical parameters of each lift station wet well structure, illustrating their respective weight, sliding resistance, buoyant force, and factor of safety. Both structures were analyzed with the assumption that groundwater existed at the top of the surface. Assumptions also included using a concrete density of 145 lb/ft³, hollow weight and no cover over the structures.

**TABLE 5
 Buoyancy Resistance**

Lift Station	Geometry	Wall Thickness, T _w (ft)	Base Thickness, T _b (ft)	Depth (ft)	Weight of the Structure (lb)	Sliding Resistance (lbs)	Buoyant Force (lbs)	Factor of Safety
Deep (Extended Base – 1ft)	Circular 8' Diameter	0.5	1	32.5	155375	147273	104732	<u>2.89</u>
Shallow (Smooth-Wall)	Square 9' x 9'	0.5	0.5	8	27912	2582	49920	<u>0.61</u>

According to ACI 350, the safety factor against flotation is computed as the total dead weight of the structure divided by the total hydrostatic uplift force. Where maximum ground water or flood levels are not well defined, or where soil friction is included in the flotation resistance, FS of 1.25 can be used. For extended base designs, since friction or cohesion are the primary forces resisting flotation, a minimum safety factor of 2.0 would be appropriate to account for the variability of the soil properties.

Analysis indicates that the deep wet well structure is stable with respect to buoyancy and has a minimum factor of safety greater than 2.0. Therefore, the deep lift station wet well structure doesn't need additional design. Since the shallow wet well structure has a safety factor less than 1, the uplift force is greater than the downward forces, which means that the structure will float without buoyancy countermeasures.

**TABLE 6
 Buoyancy Resistance – Shallow Wet Well with Extended Base**

Lift Station	Geometry	Wall Thickness, T _w (ft)	Base Thickness, T _b (ft)	Depth (ft)	Weight of the Structure (lb)	Sliding Resistance (lbs)	Buoyant Force (lbs)	Factor of Safety
Shallow (Extended Base – 1ft)	Square 9' x 9'	0.5	0.5	8	57732	3099	51293	<u>1.19</u>

Presented in Table 6 are the physical parameters for buoyancy resistance if the shallow lift station wet well had a base extension of 12 inches. Using the extended base design increases the factor of safety, almost doubling in resistance. There are several other methods that can be used to increase the buoyancy resistance of the structure other than the base extension method. Other methods to overcome buoyancy are anchor structures, anchoring the structure to a large concrete mass, increasing member thickness, or simply making the structure deeper and fill with additional concrete.

Figures 3 and 4 illustrates the relationship between buoyant force and factor of safety for the square shaped shallow wet well structure and circular shaped deep wet well structure with their each respective change in depth. This buoyant force is resisted by the weight of the structure's weight and the total sliding resistance.

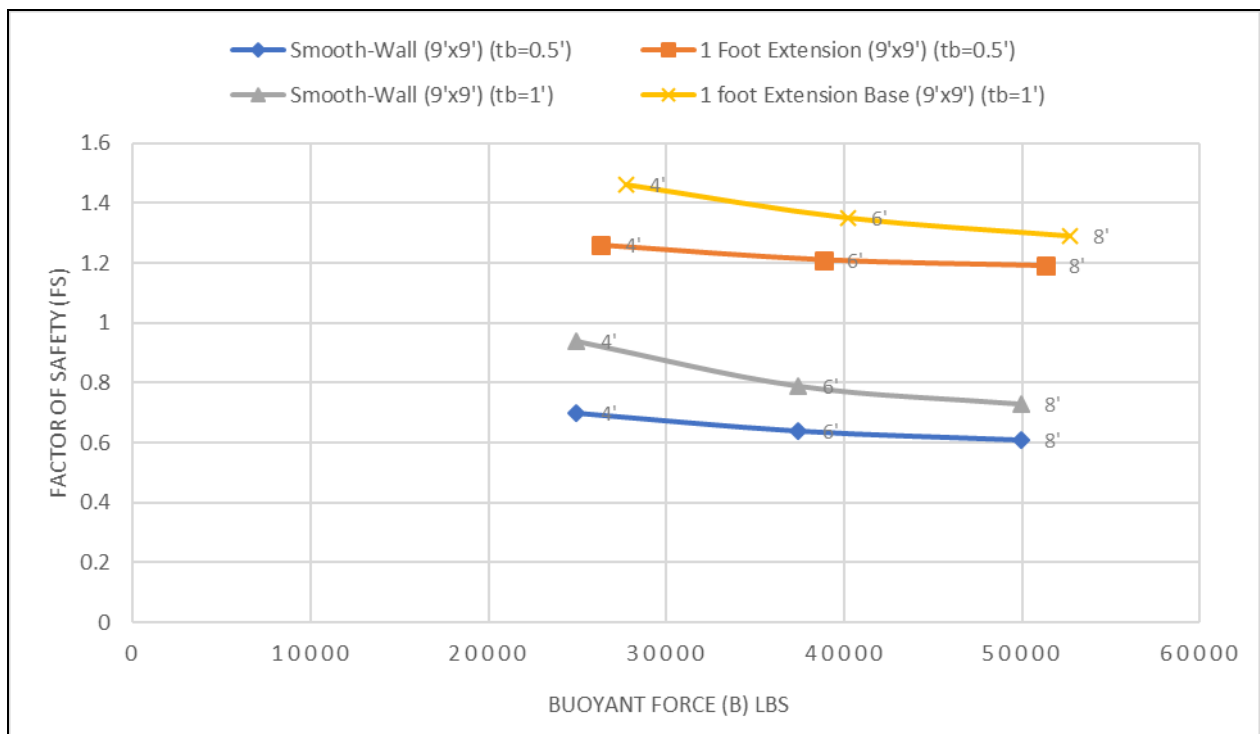


Figure 3: Factor of Safety in respect to buoyant force for different buoyancy countermeasure designs – Square Shaped

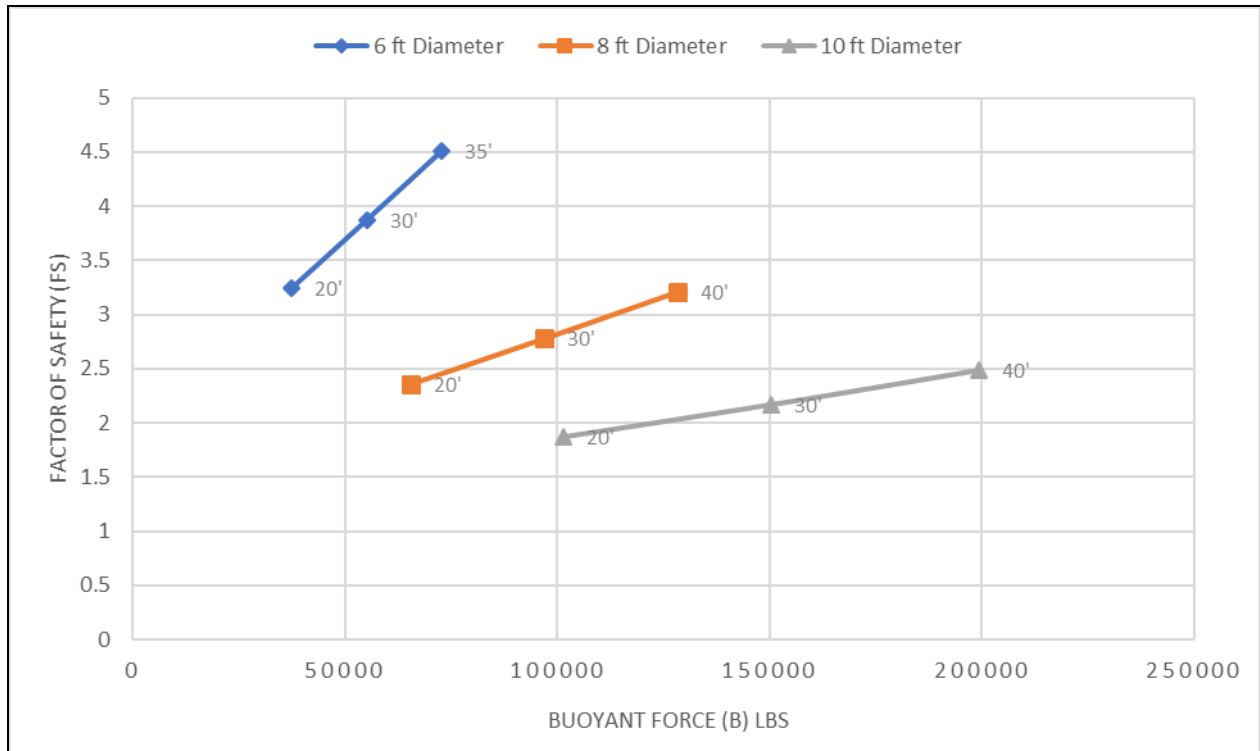


Figure 4: Factor of Safety with respect to buoyant force for circular shaped deep wet well structure – extended base design

The numbers to the right of each point represent the depth of the structure with respect to the diameter or size of the lift station wet well. The buoyancy analysis is defined as the density of water multiplied by the volume of water displaced by the structure, and is expressed as:

$$B_{circular} = \gamma_w \left(\pi \frac{B_d^2}{4} \right) H ; B_{square} = \gamma_w (B^2) H$$

Analysis indicates that an extended base, increased base and wall thickness would be preferable in the design of the lift station wet well regarding buoyancy resistance. However, the designer is advised to use caution in the analysis of the extended base structures because the parameters employed vary greatly with material properties, compaction levels, and friction factors assumed between native and compacted soils.

Preliminary Shoring Design

Safe inclinations of temporary excavations should conform to regulatory requirements and are the contractor’s responsibility. The soil profile and soil properties of boring log 10 from the referenced report were input into the computer program GeoStudio for a deep slope cut analysis. The critical failure surface searching method was specified for analysis using a random technique for generating circular surfaces. It is estimated that a 100 psf surcharged temporary excavations in the silty sand materials steeper than about 1:1 (H:V) will require worker protection and/ or shoring. These cuts in clean sands will slough with time and as they dry out. If clean sand layers slough and undermine overlying competent layers it may be necessary to flatten slopes to maintain stability. Based on the anticipated groundwater levels, it may be necessary to dewater excavations deeper than 5 feet.

Where worker safety or support of adjacent improvements is of concern, excavations should be shored. Heavy construction equipment, construction materials, excavation soil, and vehicular traffic should be kept sufficiently away from the top of any excavation to prevent any unanticipated surcharging. As a general guideline, spoil piles or heavy equipment should be set back (minimum 5 feet) behind the top of unsupported excavation slopes.

Hot Mix Asphalt (HMA) Structural Sections

Flexible pavement design recommendations have been developed for the given T.I.'s based upon the California Department of Transportation (Caltrans) design procedures and a design R-value of 8. The flexible asphalt concrete pavement sections associated with the assumed T.I.'s for on-site asphalt pavements are summarized in Table 7.

**TABLE 7
RECOMMENDED MINIMUM PAVEMENT SECTIONS**

Traffic Index	Asphalt Concrete (inches)	Aggregate Base – Class 2 (inches)
4.5	2.5	8.5
5.0	2.5	10.5
5.5	3.0	11.0
6.0	3.0	13.0
6.5	3.5	14.0
7.0	4.0	14.5
7.5	4.0	16.5
8.0	4.5	17.5

The design criteria assumes a 20-year design period and that normal maintenance (crack sealing, etc.) is performed. The traffic index is a measure of the volume of truck traffic that will be applied to a pavement section in the design life. The allowable average daily truck traffic (ADTT) for the assumed traffic indexes is presented in Table 8.

**TABLE 8
 AVERAGE DAILY TRUCK TRAFFIC**

Traffic Index	2-Axle Vehicle	or	3-Axle Vehicle	or	5-Axle Vehicle
4.5	2.2		0.8		0.2
5.0	5.2		2.0		0.5
5.5	11.6		4.3		1.1
6.0	24.1		9.0		2.4
6.5	47.3		17.7		4.7
7.0	88.1		33.0		8.8
7.5	157.3		59.0		15.8
8.0	270.6		101.5		27.1

The flexible pavement should conform to and be placed in accordance with the Caltrans Standard Specification, 2015. The aggregate base and upper 12 inches of subgrade should be compacted to a minimum of 95 percent relative compaction as determined by Caltrans Test Method 216 (Dry determination) or ASTM D1557 test procedures.

Grading and Backfill

Based on the laboratory data, field exploration, and geotechnical analyses conducted for this study, it is geotechnically feasible to construct the proposed improvements as currently envisioned. Provided that the recommendations presented in this report are incorporated into the project design and construction, use of conventional spread footings bearing on undisturbed native soil or approved engineered fill are considered appropriate for structure support.

Through Expansion Index (EI) testing, the investigation has revealed some moderately expansive foundation soils. These expansive soils are susceptible to volume changes associated with changes in soil moisture content. The potential for future differential movement resulting from these soils can be reduced to normally tolerable levels by following the moisture conditioning and compaction recommendations presented in this report. Expansion characteristics should be determined during grading for each soil type in conjunction with the maximum density determination. Moisture condition and compaction mitigation implemented during grading should be consistent with the expansiveness determined. Careful attention must be paid to future maintenance, including site drainage and irrigation practices.

Note that the moisture content attained during grading and building pad preparation should be maintained between the completion of grading and the placement of the vapor retarder membrane, concrete slabs, and footings. If the moisture content is not maintained between the conclusion of grading and the start of building construction, the moisture content will need to be reestablished prior to building construction.

All surface vegetation and any miscellaneous surface obstructions should be removed from the project area, prior to any site grading. It is anticipated that stripping of vegetation could involve the upper 1 to 3 inches of the site. Surface strippings should not be incorporated into fill unless they can be sufficiently blended to result in an organic content less than 3 percent by weight (ASTM d2974). Stripped topsoil, with an organic content between 3 and 12 percent by weight

may be stockpiled and used as non-structural fill (i.e. landscaped areas). If used in landscape areas soil with an organic content between 3 and 12 percent should be placed within 2 feet of finished grade and at least 5 feet outside of building perimeters. Soil with an organic content greater than 12 percent by weight should be excluded from fill.

Initial site grading should include a reasonable search to locate soil disturbed by previous activity and any undocumented fill soils, or existing utilities that may exist within the area of construction. Any subsurface obstructions should be removed from the project area. Any areas or pockets of soft or loose soils, void spaces made by burrowing animals, undocumented fill, or other disturbed soil that are encountered, should be excavated to expose approved firm native material. Excavations for removal of any unsuitable conditions should be dish-shaped and backfilled with 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 mitigate the effects of 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 and some construction materials (e.g. aluminum).

All engineered fill soils should be nearly free of organic or other deleterious debris and less than 3 inches in maximum dimension. The on-site soil exclusive debris may be used as engineered fill, provided it contains less than 3 percent organics by weight (ASTM D2974).

Should any imported material be used for engineered fill, it should be sampled and tested by a representative of the project Geotechnical Engineer prior to being transported to the site. Table 9 provides general criteria for imported soil.

TABLE 9
IMPORT FILL CRITERIA

<u>Gradation</u> <u>(ASTM C136)</u>			
<u>Sieve Size</u>		<u>Percent Passing</u>	
76 mm (3-inch)		100	
19 mm (¾-inch)		80 – 100	
No. 4		60 – 100	
No. 200		20 – 70	
<u>Expansion Index</u> <u>(ASTM D4829)</u>		<u>Plasticity</u> <u>(ASTM D4318)</u>	
		<u>Liquid Limit</u>	<u>Plasticity Index</u>
< 80		< 50	< 25
<u>Organic Content</u> <u>(ASTM D 2974)</u>			
< 3% by dry weight			
<u>Corrosivity</u>			
<u>pH</u>	<u>Minimum Resistivity</u> <u>(ohm-cm)</u>	<u>Soluble Sulfate</u> <u>(ppm)</u>	<u>Soluble Chloride</u> <u>(ppm)</u>
6 to 8	> 2,000	< 2,000	< 500
<u>Resistance Value</u> <u>California Test Method No. 301</u>			
Minimum R-value = 8			

The import criteria for corrosion are typical threshold limits for non-corrosive soil. Should corrosion concentrations of import soils fall outside of the threshold limits indicated above, revised protection measures will be necessary.

Soils used as engineered fill should be uniformly moisture-conditioned to at least the percentages above optimum indicated in Table 8, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to within the required range of relative compaction indicated in Table 10. Disking and/or blending may be required to uniformly moisture condition soils used for engineered fill. The actual level of moisture conditioning and compaction will be based on Plasticity Index (PI) or Expansion Index (EI) test and moisture density relationships determined during grading. The general intent is to bring the expansive material to about 80 to 85 percent saturation at the time of construction. Based on expansion index tests performed for this study, the following moisture and compaction ranges are recommended for on-site soils.

**TABLE 10
 MOISTURE CONDITIONING AND COMPACTION**

Soils		Relative Compaction (min – max)	Minimum Moisture Conditioning (% Over Optimum)
PI	EI		
< 9	< 20	90%	+ 0%
9 to 15	21-40	90-95%	+ 3%
16 to 25	41-80	88-92%	+ 4%
> 25	> 80	88-92%	+ 5%

The upper 12 inches of pavement subgrade should be moisture conditioned to between 2 to 4 percent above optimum and compacted to between 90 and 93 percent relative compaction. Relative compaction is to be determined by Caltrans No. 216 (dry weight determination) or ASTM D1557 test procedures.

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 specific types of pipes. It is recommended that 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. Randomly excavated near surface soil will likely be Class III or Class IV material per ASTM D2321.

Trench zone backfill (i.e., material placed between the pipe zone backfill and finished subgrade) may consist of native soil which meets the requirements for engineered fill.

It should be noted that the native sandy clay soils may require significant effort to achieve compaction within narrow trenches. If granular import is used for backfill, a native clay soil or lean concrete slurry dike should be provided in the upper 4 feet where the trenches cross beneath the perimeter of the structures. This dike is intended to minimize the lateral migration of subsurface water into clay soil under the buildings. If granular import material is used for pipe or trench zone backfill, it should have a piping ratio compatible with the adjacent soil, or a geofabric separator should be utilized.

All trench backfill should be placed and compacted in accordance with recommendations provided for engineered fill. Reduced compaction (85 percent minimum) could be specified for trench zone backfill in non-structural areas located a distance equal to the depth of the trench from any structure and appurtenant improvements. Mechanical compaction is recommended; ponding or jetting should not be used.

**TABLE 11
 PIPE ZONE BACKFILL PARAMETERS**

Soil Stiffness Modulus (psi)			Backfill Density (pcf)	
E' _n (Trench Sidewall)	E' _b (Backfill)		85% Compaction	90% Compaction
	85% Compaction	90% Compaction		
3,000	700	1,000	118	125

E'_n represents the modulus for the undisturbed natural soil and is based on relative density and data by Howard (1996). E'_b is the modulus for backfill derived from random excavation of on-site soil and is based on data by Hartley and Duncan (1982) and Watkins and Anderson (2000). The design E' will be dependent upon the pipe diameter and trench width, which dictates the relative influence of E'_n and E'_b. Methods by Howard (1996) are suggested for evaluating the design E'. **TECHNICON** can furnish a recommended design E', if provided with pipe diameter and specifications for trench construction.

Stormwater Basin Berm Slopes (2' high) and Sections

The proposed stormwater basin berms are anticipated to consist of cut and fill slopes with an overall depth of up to 2 feet. The gradients of the interior and exterior basin slopes have not yet been determined, however, based on information from the referenced report, preliminary planning calls for gradients of the basin slopes of approximately 2:1 (H:V).

Planning calls for basin slopes of 2:1 (H:V). The stability of the basin slopes was analyzed utilizing the computer program GeoStudio. The analysis indicated that cut and fill slopes constructed at a gradient of 1:1 (H:V), or flatter would be stable with regard to deep-seated failure up to a height of 10 feet. Steeper slopes may be considered and should be evaluated once the final planning/design is complete.

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.

All excavations should be constructed and maintained in conformance with current OSHA requirements (29 CFR Part 1926) for a Type C soil. If excavations encounter saturated soils or groundwater, temporary excavations will have to be laid back or shored and the trench dewatered to maintain stability.

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.

Design Review and Consultation

It is recommended that **TECHNICON** be retained to review those portions of the contract drawings and specifications that pertain to earthwork, foundations, and pavements prior to finalization to determine whether they are consistent with our recommendations.

As a result of borings having different water table elevations, soil profiles and properties throughout the site area and the high probability of soil discrepancy between the closest boring and the location of the area of interest, it is recommended to not use the design values mentioned until proper borings are done at the specific location(s).

CLOSING

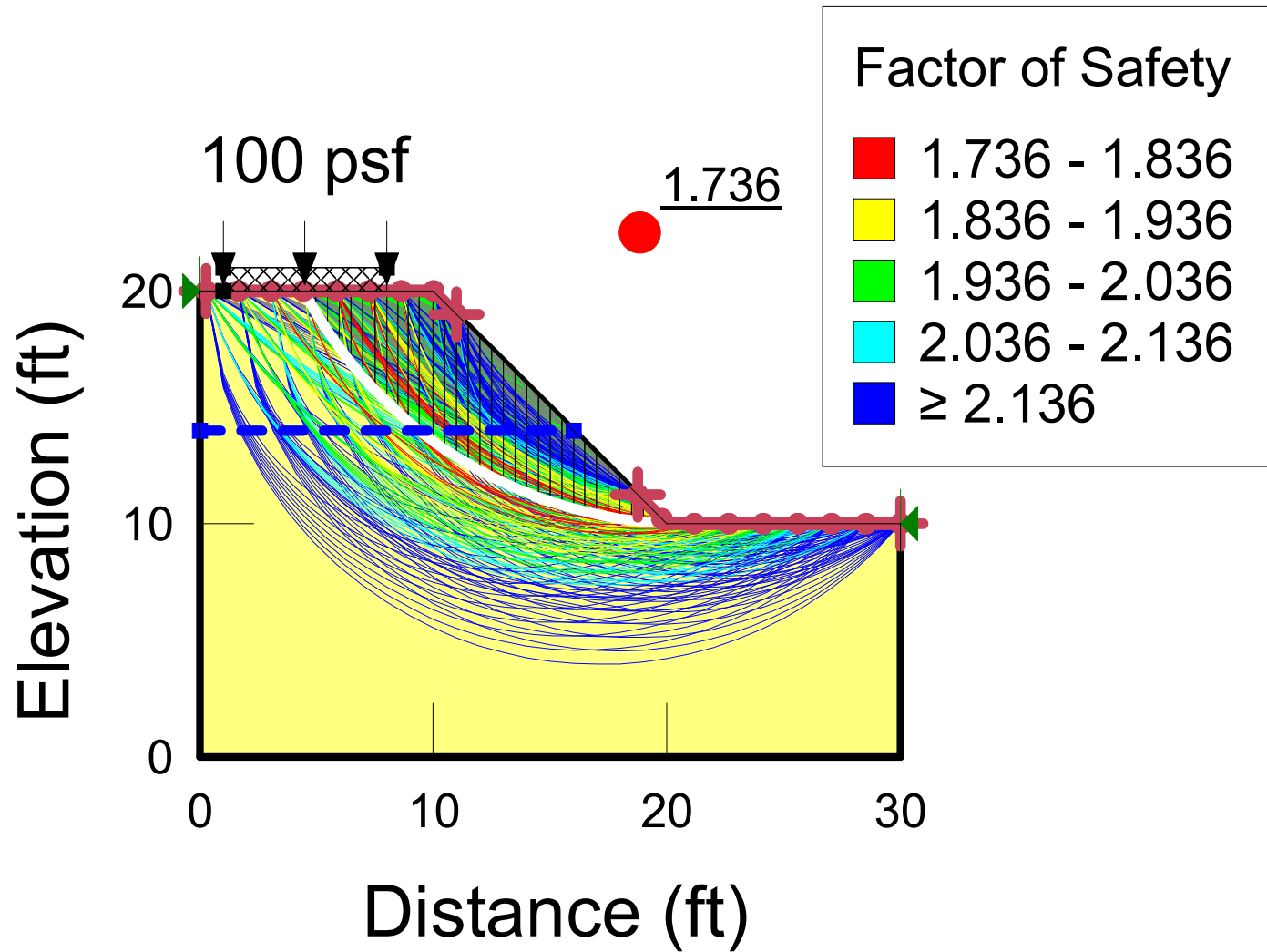
Thank you for your valued business. We trust that this letter/report satisfies your needs. If you have any questions regarding the information presented, please contact us at your convenience.

Respectfully submitted,
TECHNICON Engineering Services, Inc.


Salvador Alvarez, PE
Geotechnical Engineering Manager



MST:SA:vm



SLOPE/W Analysis

Stormwater Basin Berm Slope.gsz

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